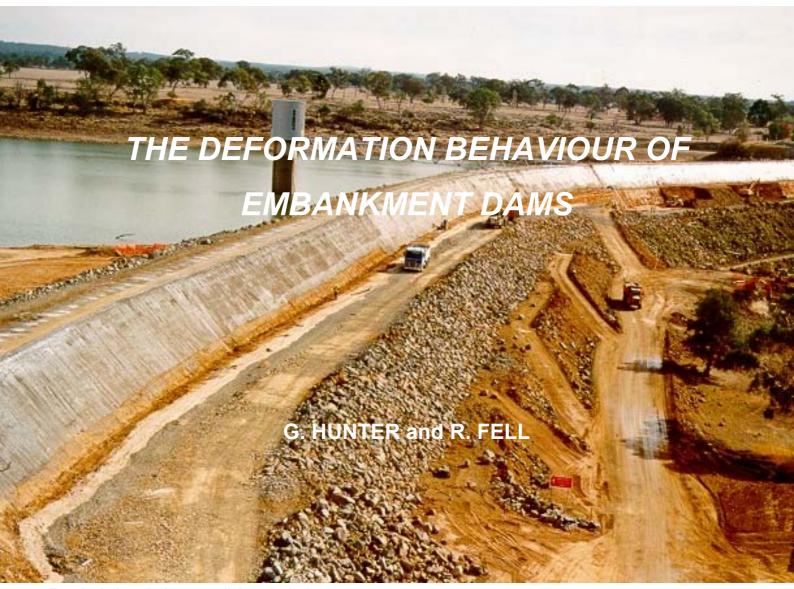


STUDIES FROM SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING



Eppalock dam during remedial works in 1999 photo courtesy of Goulburn Murray Water

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	This 1	report presents the findings of the	e analysis of the deformation behaviour
	of em	bankment dams.	
7. Abstract			
	The a	leformation behaviour during co	nstruction, on first filling and post first
	filling has been analysed from a database of some 134 embankments,		
	including earthfill, zoned earthfill, zoned earth and rockfill and puddle core		
	earthj	fill embankments. Factors influe	ncing the stress conditions, the strength
	and compressibility properties of the embankment materials, and the interaction between embankment zones have been evaluated. Method for identification of potentially "abnormal" deformation behaviour have been developed and mechanism/s for the "abnormal" deformation behaviour assessed on a case by case basis. Guidelines are presented for prediction of		
	the de	eformation behaviour during and	l post construction that incorporate the
	signif	icant factors found to influenc	e the deformation behaviour for the
	vario	us embankment types considered.	
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1.0 INTRODUCTION

1.1 OBJECTIVES OF THE STUDY

Failures by slope instability constitute about 6% of failures and 37% of incidents of embankment dams (Foster et al 1998, 2000). A number of these incidents were prevented from becoming failures by early detection of the impending failure from visual inspection or from deformation monitoring, and remedial action is taken by lowering the reservoir and/or construction of stabilization works.

Those who are responsible for interpreting the monitoring data do not have clear criteria to assess their data and have to rely on their judgement and personal experience. Currently available numerical modelling packages and the sophistication of the constitutive models on which they are based are improving making for useful tools in modelling embankment dam behaviour. However, few reported Type A predictions have been able to accurately model the deformation behaviour during construction and on first filling. Methods based on comparison to historical records of similar embankments are still heavily relied upon and, particularly for the assessment of post construction deformation behaviour, provide the responsible personnel with the best guide to assessment of embankment performance.

Overall, the purpose of the presented data is, from a broad database of well-instrumented earth and earth-rockfill embankments, to provide dam owners and their consultants with methods that:

- Allow comparison of their structures to similar embankment types in terms of the deformation behaviour during construction, on first filling and post first filling.
- Broadly define "normal" deformation behaviour and from this platform to then identify potentially "abnormal" deformation behaviour, either in terms of magnitude, rate or trend.
- Provide some guidance on the trends in deformation behaviour that are potentially indicative of a marginally stable to unstable slope condition, and precursors to slope instability.

The embankment types considered in this report include earthfill, zoned earthfill, zoned earth and rockfill (mainly central core earth and rockfill) and puddle core earthfill embankments. A brief summary of the literature related to slope instability statistics and prediction of deformation behaviour is presented in Section 2.0. The database of case studies is summarised in Section 3.0. Sections 4.0 and 5.0 present the general deformation behaviour for the embankment types considered, and Section 6.0 the methods for identification of "abnormal" deformation behaviour with reference to case studies.

Section 6.6 presents a summary of the methods of identification of "abnormal" deformation behaviour and highlights important aspects evident from the case study analysis. Guidelines are also presented on trends in the deformation behaviour that are potentially indicative of a marginal stability condition. Section 7.0 presents a summary of the outcomes from the analysis for use in prediction of, or comparison of the deformation behaviour of an embankment during and post construction.

Related reports to the study of embankment dams as part of this project include:

- Hunter and Fell (2002a) on the deformation behaviour of rockfill from analysis of the deformation behaviour of rockfill in concrete face rockfill dams.
- Hunter and Fell (2002b) on the post failure deformation behaviour of slides in embankment dams and cut slopes in heavily over-consolidated clays. This report discusses the mechanics of failure and post failure deformation behaviour of failures in embankment dams, and provides guidelines for estimation of slide velocity and prediction of travel distance. Some information is also provided on the precursory signs to slope failure.

1.2 **DEFINITIONS**

1.2.1 Embankment Dam Zoning Classification

The embankment dam zoning classification system used is essentially the system developed by Foster (1999). The system incorporates three components to describe the general zoning of the embankment; embankment type, embankment filters and foundation filters, and consists of a three number system (Figure 1.2). The codes to the numbering used for the embankment zoning category are given in Figure 1.1. Two other codes are used to describe the embankment zoning category that are not shown in Figure 1.1, they are:

- 12 Other embankment types such as timber crib dams
- 13 Unknown embankment type

For the filter classification the numbering code system used for both the embankment and foundation filters is as follows:

- 0 No filter drains present
- 1 One filter present
- 2 Two (or more) filter zones present
- X Unknown

The earthfill materials used in embankment construction have been classified in accordance with the Australian soil classification system (Australian Standard AS 1726-1993 Geotechnical Site Investigations). This classification system is similar to the Unified Soil Classification System (USBR Earth Manual and ASTM D2487-69) except that the particle size limits for the sand and gravel sizes are in metric units and at slightly different sizes.

Foster also developed a subjective classification system for the degree of compaction of earthfill materials, but it has not been used here because for most dams considered in this study the earthfills were either well-compacted rolled fills or puddle core earthfills.

A sub-classification system has been developed to describe the thickness or width of the core. It was developed mainly to distinguish between zoned earthfill or earth and rockfill embankments with thin central cores from those with thick central cores, then broadened to encompass all earthfill and zoned earth-rockfill embankments. The core width classification is as follows:

- Thin earthfill core symbol c-tn. Core width increasing at less than or equal to 0.5 times the vertical distance below the crest; i.e. slopes less than or equal to 0.25H to 1V for central cores.
- Medium thickness earth core symbol *c-tm*. Core width increase in the range 0.5 to 1.0 times the vertical distance below the crest. For central cores it includes cores with slopes greater than 0.25H to 1V and less than or equal to 0.5 H to 1V.
- Thick earthfill core symbol *c-tk*. Core width increase in the range 1.0 to 2.5 times the vertical distance below the crest. For central cores it includes cores with slopes greater than 0.5H to 1V and less than 1.25 H to 1V.
- Very broad earthfill core symbol *c-vb*. This classification includes all homogeneous earthfill embankments and earthfill embankments with filters as well as zoned earthfill and zoned earth and rockfill embankments where the main earthfill water barrier zone has an average width equal to or greater than the width at the crest plus 2.5 times the depth below the crest.

For zoned earthfill embankments the core width classification only considers the width of the main water barrier zone, from its upstream edge to the downstream edge of the filter or permeable transition zones. Therefore, zoned embankments with filters separating similar type earthfills in the downstream shoulder may not be

classified as "very broad", although, earthfill embankments with chimney filters are classified as "very broad". This is somewhat contradictory but the database only includes one earthfill embankment with a chimney filter (Mita Hills) classified as "very broad" that would fail the zoned embankment criteria for the "very broad" classification.

- 3 -

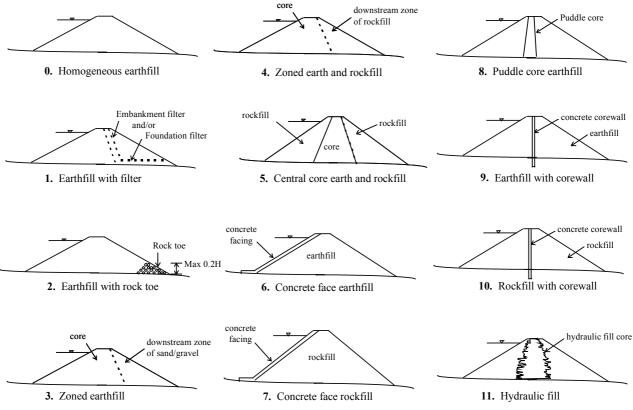


Figure 1.1: Dam zoning categories of embankment types (Foster et al 1998)

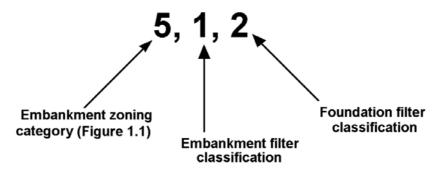


Figure 1.2: Embankment zoning classification system (Foster 1999)

1.2.2 Classification of Rockfill Placement

The method of placement of rockfill has a significant influence on its compressibility during construction and its deformation behaviour post construction. The definitions by Cooke (1993, 1984) for dumped and compacted rockfill have be used as a basis for categorisation of the method of placement. The definitions used are:

• Compacted Rockfill – rockfill placed in layers up to 2 m thickness (generally 0.9 to 2.0 m thick) and compacted by smooth drum vibrating roller. Accepted practice is typically 4 to 6 passes of a minimum 10 tonne (possibly up to 15 tonne) deadweight vibrating roller, with variation in layer thickness, added water and number of passes depending on the quality and type of the rockfill, amount of fines and location within the embankment. Three classifications for compacted rockfill have been used:

- Well-compacted layer thickness typically less than about 1.0 m (depending on the compressive strength of the intact rock) and compacted with a minimum of four passes of a 10 to 15 tonne deadweight smooth drum vibrating roller (SDVR).
- Reasonable Compaction layer thickness typically 1.5 to 2.0 m and compacted with typically four passes of a 10 tonne SDVR.
- Reasonably to Well Compacted layer thickness typically 1.2 to 1.6 m (depending on the compressive strength of the intact rock) and compacted with typically 4 to 6 passes of a 10 to 15 tonne SDVR.
- Rockfill not formally compacted or "poorly compacted". Several methods of rockfill placement have been included under the definition "poorly compacted", these include:
 - Dumped rockfill rockfill placed in lifts ranging from several to tens of metres thickness, with or without sluicing, and without formal compaction.
 - Rockfill placed in lifts less than about 2 to 3 m thickness and not formally compacted (i.e. without the use of rollers for compaction). Specified track rolling by bulldozer or other plant, or rockfill indicated as being trafficking by trucks or other haulage equipment has been classified under "not formally compacted".
 - Rockfill placed in lifts greater than 2 to 3 m and formally compacted. For these rockfills the lift thickness is considered too great for compaction to have any significant influence at depth.

Watering is an important component for placement of rockfills, particularly in cases where the compressive strength of the rock used in the rockfill is susceptible to reduction on wetting, breaks down under the action of the roller, or if the rockfill contains large quantities of fines. Cooke (1993) comments that watering is not overly important for compaction of very high strength rockfills that are not susceptible to weakening on wetting. However, these rockfills can still show collapse type settlements on wetting.

For dumped rockfills, sluicing had a significant influence on the deformation behaviour of the rockfill as evidenced by the large collapse deformations of dry dumped or poorly sluiced rockfills when wetted (Cogswell dam (Bauman 1958), Strawberry and Dix River dams (Howson 1939)).

1.3 DEFORMATION BEHAVIOUR OF ROCKFILL

Hunter and Fell (2002a) analysed the deformation behaviour of rockfill from a database of thirty-six case studies of mainly concrete face rockfill dams (CFRD). Predictive methods were developed for estimation of the deformation of rockfill based on historical records of embankment performance. Methods were developed or current methods enhanced for estimation/prediction of the vertical deformation during construction, deformation of the face slab and post-construction crest settlement for CFRD. The methods were derived from a good quality data set and consideration of the significant factors that influenced the deformation behaviour.

The accuracy of prediction of rockfill deformation during construction (for both numerical and empirical methods) is dependent on the quality of representation of the stress-strain relationship of the rockfill, which is strongly dependent on the rockfill properties, in particular the intact strength of the rock, method of placement and particle size distribution after compaction. The embankment height is a significant factor as this predicates the level of applied stress within the embankment. Valley shape is a significant factor for embankments constructed in narrow valleys due to the effects of cross-valley arching and resultant reduction in applied stresses. Typical stress-strain relationships of rockfill from field measurements during construction (under the embankment centreline) show that the relationship is non-linear and has a general trend of decreasing secant (and tangent) modulus with increasing applied stress.

For dumped rockfill there was not sufficient data from which to give guidelines on moduli during construction.

The predictive method of post-construction crest settlement is based on historical records. The significant factors incorporated into the deformation prediction include the degree of compaction of the rockfill (i.e. dumped versus compacted), intact rock strength and embankment height. The long-term crest settlement rate values provide a useful guide for comparison to rockfills used in other embankment types, such as central core earth and rockfill dams.

2.0 LITERATURE REVIEW

2.1 FAILURE AND ACCIDENT STATISTICS

This review of slope instability incidents in embankment dams is a summary of that by Hunter and Fell (2002b) as part of the post-failure deformation analysis of instability incidents in embankment dams. Most of the data and findings on slope stability incidents is a summary of the work by Foster et al (1998) on dam failure and accident incidents, derived from a database of some 11192 large embankment dams from around the world constructed up to 1986.

Foster et al (1998) reported 124 incidents of slope instability. Of these, 12 were failures that involved a breach and uncontrolled release of water from the reservoir and the remaining 112 were accidents (no breach). They concluded that the major factors influencing the likelihood of slope instability in embankment dams were:

- Slope instability is more likely in earthfill embankments than earth and rockfill embankments. Homogeneous dam embankments (without filters) and hydraulic fill dam embankments are statistically the most susceptible to slope instability.
- Slope instability is more likely for dam embankments on soil foundations than for dam embankments on rock foundations. Of the slides passing through the embankment and foundation, the incidence of slope instability is much greater in foundations of high plasticity clays.
- Slope instability is much more likely in dam embankments constructed of high plasticity clays and silts, particularly for slides in the downstream slope. Dam embankments constructed of sands and gravels have a significantly lower incidence of slope instability.
- Compaction, or lack there of, of the earthfill materials is also a significant factor. The incidence of slope instability is much more likely for earthfills constructed by placement with no formal compaction, and the least likely for rolled, well compacted earthfills.

Further analysis by Hunter and Fell (2002b), which excluded hydraulic fill embankments, found that:

- Earthfill embankments (homogeneous earthfill, earthfill with filters and earthfill with rock toe) made up a very high percentage (80%) of the slope stability incidents that passed through the embankment only.
- For embankments that incorporate rockfill zones, it is very to extremely unlikely that the surface of rupture will pass through the rockfill zone itself, it will preferentially pass through the foundation.

2.2 HISTORICAL DEVELOPMENT OF EMBANKMENT DAMS AS THIS AFFECTS DEFORMATION BEHAVIOUR

2.2.1 Earth and Earth-Rockfill Dams

An historical summary of the use of rockfill in embankment design and construction (Galloway 1939; Cooke 1984; Cooke 1993) is given in Table 2.1.

Table 2.1: Historical summary of rockfill usage in embankment design (Galloway 1939; Cooke 1984; Cooke 1993).

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Approximate	Method of Placement and	Comments							
Time Period	Characteristics of Rockfill								
	Concrete Face Rockfill Dams								
Mid to late 1800's to early 1900's	Dumped rockfill with timber facing	Early embankments constructed with timber facing. Typically of very steep slopes (up to 0.5 to 0.75H to 1V). First usage of concrete facing in the 1890's. Height limited to about 25 m.							
1920's to 1930's	Main rockfill zone dumped in high lifts (up to 20 to 50 m) and sluiced, although the sluicing was relatively ineffective. A hand or derrick placed rockfill zone was used upstream.	Rockfill typically sound and not subject to disintegration. Dam heights reaching 80 to 100 m. For high dams, cracking of the facing slab and joint openings resulted in high leakage rates (2700 l/sec Dix River, 3600 l/sec Cogswell, 570 l/sec Salt Springs).							
Late 1930's to 1960's	High pressure sluicing used for the main rockfill zone. Rockfill still very coarse.	Cracking of face slab, particularly at the perimeter joint, and high leakage rates a significant issue with higher dams (3100 l/sec at Wishon, 1300 l/sec at Courtright).							
From late 1960's	Rockfill placed in 1 to 2 m lifts, watered and compacted. Reduction in particle size. Usage of gravels and lower strength rock.	Significant reduction in post-construction deformations due to low compressibility of compacted rockfill. Significant reduction in leakage rates; maximum rates typically less than 50 to 100 l/sec. Continued improvement in plinth design and facing details to reduce cracking and leakage.							
	Earth and Ro	ockfill Dams							
1900 to 1930	Dumped rockfill	Use of concrete cores with dumped rockfill shoulders at angle of repose. Limited use of earth cores. Dam heights up to 50 to 70m.							
1930's to 1960's	Earth core (sloping and central) with dumped rockfill shoulders.	Use of earth cores significant from the 1940's due to the difficulties with leakage of CFRD. Increasing dam heights up to 150 m.							
From 1960's	Use of compacted rockfill. Typically placed in 1 to 2 m lifts, watered and compacted with rollers.	Improvements in compaction techniques. Early dams compacted in relatively thick layers with small rollers. Gradual increase in roller size and reduction in layer thickness reduced the compressibility of the rockfill. Significant increase in dam heights in the mid to late 1970's, up to 250 to 300 m.							

From the 1930's to the mid 1960's the height of construction of CFRDs was limited to about 80 to 100 m due to problems with face slab cracking and high leakage rates as a result of excessive deformation of the dumped and sluiced rockfill. In terms of stability though the embankment performance was excellent (Cooke 1984). The transition from dumped and sluiced to compacted rockfill in embankment design (Cooke 1984, 1993), occurring in the late 1950's to 1960's, resulted in proliferation in the use of CFRD from the late 1960's.

For earthfills, the significant changes affecting deformation behaviour were developments in placement methods. The use of rollers from about the 1850's was significant, although their usage was more widespread from about the late 1890's into the early 20th century. But, the most significant development was after the 1930's with the application of soil mechanics principles (Wilson and Squier 1969; Penman 1986) and consideration of moisture content control, compacted layer thickness and level of compaction. Consideration of these principles was evident in embankment construction earlier in the 20th century (Schuyler 1912), such as at Belle Fourche dam in California.

2.2.2 Puddle Core Earthfill Dams

Skempton (1990) referred to the puddle core earthfill dam as the preferred method of dam construction in the United Kingdom (UK) in the 19th and first half of the 20th century. It was also the preferred method of construction in India and Australia up to the 1930's.

The significant historical events in the design and construction of puddle dams in the UK (Skempton 1990; Foster 1999) are summarised as follows:

- The first puddle dams were constructed in the late 18th century.
- Puddle Core improvements in preparation procedures resulting in greater uniformity of the core material
 - Late 18th century to mid 19th century the puddle core was watered on the embankment, allowed to soak and then heeled (by foot) or rammed (using hand held rammers) in 150 to 200 mm layers.
 - In the second half of the 19th century the clays for core material were generally wetted up in the borrow area before being brought onto the embankment.
 - 20th century, preparation of core materials in pugmills and placement in 100 to 150 mm layers.
- Shoulder Fill significant improvements in design and method of placement
 - Initially, no zoning in shoulder fill and no formal compaction. Compaction was typically using loaded wagons and carts in relatively thick layers (generally less than 1 m but up to 1.5 2 m).
 - From the 1850's/60's, following the failures of Bilberry Dam in 1852 (by internal erosion) and Dale
 Dyke in 1864 (by a combination of internal erosion and over-topping), embankment designs generally
 included a select zone of the more cohesive earthfills adjacent to the core.
 - From the 1900's improved compaction of shoulder fill using thinner layers and steam driven rollers.
 - From the 1940's further improvements in compaction procedures following improvements in earthmoving plant and the application of soil mechanics.
- The use of rolled clay cores took over from puddle clay cores in the UK in the 1960's.

In Australia and India the use of rolled clay cores took over from puddle clay cores in the 1930's. The last large puddle dam constructed in Australia was in the 1930's. Improvements in the preparation of puddle cores and alteration in design and placement methods of the shoulder fill occurred in Australia at relatively similar time periods to those in the UK. There is evidence of the use of horse or bullock drawn sheepsfoot rollers (up to 3.5 tonne dead-weight) for compaction of the shoulders and bullocks for compaction of the puddle core in Australia from the 1870's.

2.3 FACTORS AFFECTING THE DEFORMATION OF EMBANKMENT DAMS

2.3.1 Deformation Properties of Rockfill

A more detailed assessment of the factors affecting the compressibility of rockfill are given in Hunter and Fell (2002a). In summary, field observations (Mori and Pinto 1988; and others) and the results of large scale laboratory tests (Marsal 1973; Marachi et al 1969; and others) indicate the compressibility properties of rockfill are affected by:

- Degree of compaction of the rockfill
- Applied stress conditions and stress path.
- Particle shape and particle size distribution.
- Intact strength of the rock used as rockfill

An important aspect in the evaluation of the deformation behaviour of rockfill in embankment dams is its susceptibility to collapse compression on wetting. This is discussed in Sections 4.2.2 and 6.3.1.

The time dependent or creep type deformation of rockfill is an important aspect for evaluation of the post construction deformation behaviour in embankment dams. The best guide from field records is Hunter and Fell (2002a).

2.3.2 Deformation Properties of Earthfill

The general features of the stress-strain relationship of soils are well described in text books on soil mechanics and are applicable to describe the deformation behaviour of rolled earthfills. Factors such as the degree of overconsolidation, permeability, soil structure, matric suction and the rate, magnitude and direction of loading affect the strength and compressibility properties of an earthfill. Some of these factors are affected by:

- The soil type, including its mineralogy, gradation and plasticity.
- The degree of compaction
- The moisture content at placement relative to Standard optimum

Gould (1953,1954) found that a reasonable prediction of the drained compressibility of a well compacted, rolled earthfill could be estimated from its classification and plasticity of the fines fraction. For more plastic soil types, he found that the moisture content relative to the Standard Proctor optimum influenced the shape and magnitude of the compressibility curve. Most of Gould's data was of earthfills placed on the dry side of Standard Proctor optimum.

The compacted earthfill is a partially saturated soil with an amount of over-consolidation that is related to the compactive effort of the roller and the pore water suction. The pore water suction is dependent on the moisture content at placement, the degree of saturation of the compacted earthfill and the material type. For a dry placed, well compacted earthfill, the amount of settlement increases with the increase in effective stress as the embankment is constructed. The settlement occurs due to compression of pore air in the partially saturated soil and is a relatively rapid process, as demonstrated by the fact that most of the earthfill's settlement occurs during construction. As the pore air is compressed the degree of saturation increases and pore water suction decreases. At some point, positive pore water pressures may be developed. Laboratory data presented by Fredlund and Rahardjo (1993) and Holtz and Kovacs (1981) indicate that the value of the pore pressure parameter, B, is low for degree of saturation up to 85 to 90% and possibly higher.

In compacted earthfills placed close to or wet of Standard optimum moisture content, the degree of saturation is relatively high at compaction (generally greater than 85 to 90%) and the matric suction relatively low compared with dry placed earthfills. Positive pore water pressures are generally developed at low levels of confining stress and the pore pressure parameter, B, approaches high values, from about 0.6 to almost 1.0. Assuming the earthfill to be of low permeability, conditions during conditions will be close to undrained. For these earthfill types deformations largely occur as plastic type deformations due to yielding in undrained loading.

On filling, seepage through the earthfill begins. It may take many years to reach a steady seepage condition, depending on the permeability of the partially saturated and/or saturated earthfill (refer Section 4.3.3, item f). As the degree of saturation of the partially saturated earthfill increases on saturation, deformations will occur. Deformations due to collapse compression on wetting can occur if the rolled earthfill is placed well dry of Standard optimum (even if heavily compacted) or dry placed and poorly compacted (refer Sections 4.2.2 and 6.6). In the long term, there will be a small time dependent creep component to the deformation behaviour. This is generally greater for poorly compacted soils than for well-compacted soils.

2.3.3 Summary of Research on Puddle Core Earthfill Dams

Building Research Establishment (BRE) and Imperial College have, since the 1980s, published the findings of research into the performance of puddle core earthfill dams. In the area of deformation behaviour the research has been mainly targeted at the long-term performance of these embankments more than 50 to 100 years after construction. Instrumentation installed in a number of puddle core embankments from about the late 1970's/early 1980's comprised monitoring of:

- Pore pressures, horizontal stresses and internal deformations (both vertical and horizontal) in the puddle core and in some cases in the puddle clay cut-off.
- Pore pressures in the downstream fill.
- Surface deformations of the crest, upper part of the upstream slope and downstream slope.

A set of puddle dams owned by Yorkshire Water, including Ramsden, Walshaw Dean Lower, Ogden and Holmestyes dams, have been reasonably well monitored and analysed by BRE and Imperial College in conjunction with the owners. Imperial College has undertaken finite element analysis of these and other puddle core earthfill embankments (Tedd et al 1997b; Dounias et al 1996; Kovacevic et al 1997).

Table 2.2 summarises the mechanisms identified by Tedd et al (1997a) affecting the long-term deformation behaviour of puddle core earthfill embankment dams. They comment that an understanding of the likely deformation behaviour due to processes not detrimental to dam safety allows for easier identification of deformations that may be indicative of a "hazardous" situation.

Tedd et al (1997a) defined the factors affecting the magnitude and/or direction of "normal" long-term deformation behaviour of puddle core earthfill dams as:

- The age of the dam
- The height of the dam
- The position of the element controlling the phreatic surface within the dam. For most puddle dams the central puddle core itself is the element controlling the phreatic surface within the dam, but there are several embankments where an upstream lining is the controlling element (e.g. Holmestyes Dam).
- The compressibility properties of the various embankment zones and foundation
- The permeability of the earthfill/s in the upstream shoulder
- The reservoir operation, in particular the past history of operation, the reservoir level at the time of measurement and the duration and depth of drawdown.

Table 2.2: Mechanisms affecting the long-term deformation behaviour of old puddle core earthfill embankments (Tedd et al 1997a).

Mechanisms Causing "Normal" Deformation Behaviour	Deleterious Mechanisms / Processes
Shrink-swell related deformations due to seasonal	Slope instability
fluctuations in moisture content.	Internal erosion processes
Secondary consolidation of the core and creep of the	
shoulder fill under "steady state" conditions.	
Deformations due to stress changes associated with the	
fluctuation in reservoir level during normal operation.	
Consolidation of the foundation.	

The case study data and some of the findings from the published literature by BRE and Imperial College relating to the deformation behaviour of puddle core earthfill dams is included with the analysis and discussion in this report (refer Sections 5.0, 6.5, 6.6 and 7.2). Some of the findings of the research by BRE, other than from the monitored field deformation behaviour, are:

- Laboratory oedometer tests on samples of the core and upstream fill (Holton 1992; Tedd et al 1994; Tedd et al 1997b) showed:
 - Measured c_{α} , coefficient of secondary compression, were in the range 0.0001 to 0.0032. The general trend was for c_{α} to increase with increasing effective vertical stress.

- Non-linear stress-strain behaviour on repeated unloading and reloading on remoulded samples (at stress levels below the initially applied stress). The stress-strain behaviour followed hysteresis loops, with the constrained modulus value decreasing as the number of reload cycles increased, and each load cycle resulted in a non-recovered strain, the magnitude of which was dependent on the magnitude of the load.
- Where the shoulder fill has been sourced from weathered sedimentary deposits it is relatively permeable, although the permeability is quite variable (Tedd et al 1993). These fills generally comprised a mixture of silty and sandy clays with varying proportions of gravel to boulder size fragments of mudstone, siltstone and sandstone. Tedd and Holton (1987) report the permeability from constant head tests to vary from 2 × 10⁻⁵ m/s to 10⁻⁷ m/s in the downstream shoulder fill at Ramsden Dam. For Cwnwernderi Dam, Charles and Watt (1987) reported a relatively rapid response in horizontal earth pressure on reservoir filling indicating the upstream filling to be quite permeable.
- Charles and Watt (1987) found that the horizontal stresses in the core of puddle dams some 100 years post construction were relatively low, significantly lower than nominal stresses for a homogeneous type embankment, and Charles and Tedd (1991) comment that measured vertical stresses were not significantly greater than the horizontal stresses. These findings confirm the low stresses in the core present at the end of construction due to arching are still present some 100 years after construction.

2.4 Predictive Methods of Deformation Behaviour

For embankment dams, the components of deformation are those that occur as a result of changes in effective stress conditions (during construction, on impoundment and due to reservoir fluctuation), changes in total stress (for wet placed earthfill cores in zoned embankments), on saturation or wetting (e.g. collapse compression) and the on-going time dependent or creep type deformations. Predictive methods are typically divided into the three components of deformation during construction, on first filling and long-term post construction (or post first filling).

Most predictive methods cover the deformation behaviour of one or two of these components. Finite element analyses have been used (Saboya and Byrne 1993; Kovacevic 1994; Naylor et al 1997; Dounias et al 1996, amongst others) for analysis of deformation due to changes in stress conditions, mostly for deformation during construction and first filling, but also for deformation under large drawdown. Empirical predictive methods, usually based on historical performance of embankments, are more generally available for post construction deformation some of which incorporate the deformation on first filling, although methods are available for deformation during construction.

2.4.1 Empirical Predictive Methods

2.4.1.1 Predictive Methods of Deformation During Construction

There are methods available for prediction of deformations during construction, but they are limited in number. Development of these methods has probably been curtailed by the availability of finite element methods in the last 30 to 40 years, their relative ease of use to model embankment construction and improvements in constitutive relationships to model the material compressibility and deformation behaviour. In addition, the complexity of stress and deformation behaviour during construction, particularly for zoned embankments, makes assessment by simple empirical methods difficult. Several of the methods developed are:

 Poulos et al (1972) developed charts for estimation of stresses and vertical and horizontal deformations during construction based on elastic solutions for a homogeneous embankment on a rigid foundation. The methods provide a quick and simple means of estimating deformations, which the authors comment are suitable as a preliminary estimate.

- Penman et al (1971) developed a method for estimation of the settlement during construction under the dam axis based on elastic solutions, but that allows for material stress-strain non-linearity by using an equivalent compressibility. It is based on the assumption of confined conditions (i.e. zero lateral strain) under the dam axis and is therefore suitable where this assumption is reasonable (e.g. homogeneous embankment).
- ICOLD (1993) derived a general equation for the vertical strain of rockfill in terms of vertical stress, rockfill modulus and creep strain parameters (Equation 2.1) based on the assumption of a linear relationship between stress and strain of coarse rockfill and non-linear relationship between strain and time. Derivations are given (refer ICOLD 1993) for the settlement at a specific elevation and period of time after start of construction, settlement at end of construction and settlement post construction.
- Several other methods have been developed for prediction of the lateral deformation of the shoulder during construction, including:
 - Resendiz and Romo (1972) and Walker and Duncan (1984) for earthfill or essentially homogenous dams.
 - Penman (1986) provides guidelines for assessment of "acceptable" and "excessive" rates of horizontal displacement, mainly as guide for evaluation of a possible impending failure condition. It is based on a limited number of case studies of differing embankment type and should be used with caution.

$$e = \frac{\sigma}{E_M} + \sigma \frac{t}{\theta + \lambda t} \tag{2.1}$$

where e = relative strain, σ = vertical stress (in MPa), E_M = modulus of instantaneous strain of the rockfill (in MPa), t = time (in years), and θ and λ are the empirical parameters describing creep strain (in MPa/year and MPa respectively).

One of the major issues with deformation during construction relates to material compressibility properties under the imposed stresses from the stress path during construction. The stress path and stress conditions from field instrumentation (Charles 1976) and finite element analysis (Saboya and Byrne 1993; Kovacevic 1994; Charles 1976; and implicit in the analysis by others) are shown to vary depending on the embankment zoning geometry, material properties and location within the embankment. Hence, the dependency on finite element analyses to model the stress path, stress conditions and deformation behaviour in all but the simplest problems. The assumption of confined conditions under the central embankment region for homogeneous embankments or embankments with broad central zones is shown by field instrumentation and finite element analysis to be a reasonable assumption. For these conditions published data is available for estimation of material moduli:

- Hunter and Fell (2002a, in press) for estimation of rockfill moduli from analysis of concrete face rockfill dams (refer Section 1.3 for a summary). They also provide a summary of several other predictive methods from the published literature.
- Gould (1953, 1954) analysed the compressibility characteristics of rolled earthfill from the internal deformation records of some 22 rolled earthfill and earth-rockfill embankments. All case studies were USBR embankments and predominantly of embankments with thick to very broad and well compacted rolled earthfill cores. Gould found that the classification of the earthfill and plasticity of the fines fraction provided a reasonable prediction of compressibility. For the more plastic soil types, he found moisture content relative to the Standard Proctor optimum influenced the shape and magnitude of the compressibility curve. Table 2.3 presents a summary of Gould's data (adapted from Sherard et al 1963).

Table 2.3: Vertical compression of rolled, well-compacted earthfills measured during construction (adapted
from Sherard et al 1963)

Embankment Soil Type	Approximate Range of Measured Vertical Compression (%) *1				
Embankment Son Type	At 10 psi (approx. 70 kPa) *2	At 100 psi (approx. 700 kPa) *2			
Silty gravels and coarse silty sands (GM and SM)	0.2 to 0.3	0.9 to 1.4			
Fine silty sands and silts of low plasticity (SM-ML)	0.2 to 0.5	1.2 to 2.1			
Clayey sands and gravels (GC-SC)	$0.3 \text{ to } 0.8 *^3$	1.9 to 3.3			
Clays of low to medium plasticity (CL and CL-ML)	0.2 to 1.1 * ³	2.8 to 4.6 * ⁴			

- *1 vertical compression measured from internal settlement gauges within the embankment
- *2 pressures are vertical effective stresses
- * variation directly with placement moisture content relative to Standard Proctor optimum, low value for soils placed well dry of optimum and high value for soils placed wet of optimum (Gould 1954).
- *4 spread influenced by moisture content at placement and plasticity of the fines fraction (Gould 1954).

2.4.1.2 Predictive Methods for Deformation Behaviour Post Construction

Empirical predictive methods of the post-construction deformation from the literature are typically based on historical deformation curves for similar embankment types or empirical relationships derived from historical deformation performance. Several of the methods incorporate the deformation during first filling whilst others only consider the prediction of deformation post first filling.

Table 2.4 provides a summary the range of post construction deformations of earthfill and earth and rockfill dams from the published literature. The data indicates that the post construction settlements and displacements are generally relatively small for well-compacted earthfills and rockfills, and are much greater for embankments incorporating dumped rockfill.

Predictive methods of post construction deformation behaviour have been proposed by Sowers et al (1965), Soydemir and Kjærnsli (1975), Clements (1984), Dascal (1987), ICOLD (1993) and Charles (1986), amongst others. All have been derived from the historical performance of a database of embankments, in most cases from dams predominantly of rockfill (concrete or membrane face, and sloping and central core earth and rockfill dams). The method proposed by Charles (1986) is for puddle core earthfill embankments.

The methods by Clements (1984), Dascal (1987) and the data summarised in Table 2.4, provide a range or bounds of likely deformation behaviour, with Clements having the largest database of case studies. Both Dascal and Clements assume zero time is the time of the first deformation reading after construction. Clements (1984) suggested the bounds be used to provide an expected range of deformation, and for better prediction he suggested estimating from the deformation curves of dams with similar characteristics. Pinto and Marques Filho (1985) commented that these methods are useful, however, their application is not so straight-forward as the deformation behaviour is strongly dependent on the initial time of measurement in relation to the end of construction. And, as pointed out by Parkin (1977), the shape of the deformation versus time curve will vary depending on the established zero time.

The other methods have developed empirical relationships for estimation of settlement and/or displacement, and are summarised below. Most of the proposed equations are simple logarithmic or power type relationships based on one or two variables. In comparison, the Russian method for estimation of settlement referenced in ICOLD (1993), which is based on time after the end of construction, stress level and the creep strain empirical parameters, is quite complex (Equation 2.1 gives the basic form of the equation). ICOLD (1993) provides further details on this method.

dams

Reference Range of Deformation Comments Dam Type/s Deformation (% of dam height) *1 Parameter "rockfill" *2 ICOLD (1993) 0.2 to 1.0% Crest displacement up to 50% of crest settlement the crest settlement. crest displacement 0.1 to 0.5% 0.1 to 0.2% shoulder settlement Sowers et al "rockfill" *2 crest settlement 0.25 to 1.0% 14 dams, settlements up to 10 years after construction. (1965)(upper range for dumped RF) **CFRD** Database of 68 dams. Clements crest settlement Up to 2.5% (dumped RF) (1984)0 to 0.25% (compacted RF) 0 to 2.5% crest displacement **CCER** crest settlement 0.05 to 1.25% crest displacement -0.75% to 0.5% Sloping core 0.06 to 1.1% crest settlement crest displacement 0 to 0.6% Bernell (1958) CCER with 0 to 0.2% (silty and sandy 6 dams with moraine cores placed crest settlement using the wet compaction method. moraine core during first filling moraines) Fine fractions less than 20%. 0.05 to 0.3% (clayey moraines) "rockfill" dams Dascal (1987) crest settlement < 0.35% (compacted RF) 15 Hydro Quebec dams and dikes (*2) with 0.3 to 0.55% (dumped RF) on rock foundations moraine cores up to 0.7 to 0.8% downstream shoulder settlement crest displacement ≥ crest settlement for compacted RF < crest settlement for dumped RF Sherard et al "rockfill" *2 Greater settlement for dumped crest settlement 0.1 to 0.4% (for well constructed (1963)wetted RF) Greater displacements for dams well constructed crest displacement on < 25 to 50 mm with dumped RF. dams first filling Gould (1954) rolled earthfill crest settlement < 0.2% in first 3 years Typical range of settlement for USBR dams.

Table 2.4: Published ranges of post construction deformation of embankment dams

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<0.4% up to 14 years

Sowers et al (1965) proposed a logarithmic relationship between crest settlement and time (Equation 2.2) to describe the post construction crest settlement for "rockfill" dams. It was derived from a database of 14 rockfill dams (mix of concrete face rockfill, central core earth and rockfill, and sloping core rockfill dams) with rockfill ranging from sluiced and compacted to dumped and poorly sluiced. Sowers et al found that the coefficient α was dependent on the method of placement of rockfill and ranged from 0.2 %/log cycle of time for compacted and well sluiced rockfills up to 0.7 %/log cycle of time. They commented that a straight line could not be used to approximate the post construction settlement at Dix River dam where the rockfill was dumped and poorly sluiced.

$$\Delta H = \alpha (\log t_2 - \log t_1) \tag{2.2}$$

where ΔH = crest settlement as a percentage of dam height, t_1 and t_2 are time in months from the date when construction was half completed, and α = slope of the crest settlement-time curve (in units of settlement as a percentage of dam height per log cycle of time, time in months).

Soydemir and Kjærnsli (1975) proposed a power law relationship between crest deformation (both settlement and displacement) and height (Equation 2.3), derived from a database of 48 membrane face, central and sloping earth core rockfill dams. Values of the coefficients β and δ were determined from best fit analysis taking into consideration the embankment type, deformation type (e.g. crest settlement or crest displacement) and time after end of construction (e.g. on initial impounding, 10 years after construction). Clements (1984) found a high degree of variability applying the coefficients to his data.

displacement is horizontal deformation, downstream displacement is positive and upstream is negative.

^{*2 &}quot;rockfill" dams including membrane face rockfill dams, and central and sloping earth core rockfill dams CFRD = concrete face rockfill dam, CCER = central core earth and rockfill dam, RF = rockfill

$$s = \beta H^{\delta} \tag{2.3}$$

where s = deformation in metres, H = embankment height in metres, and β and δ are constants.

Parkin (1977) commented that a rate analysis, based on the power law relationship derived from rate process theory (Equation 2.4), allows for more accurate prediction of post construction settlement of rockfill and is a more powerful tool for evaluation of performance because it encapsulates the fundamentals of creep deformation. For a power coefficient of -1 (i.e. m = 1), integration of Equation 2.4 converts to the logarithmic form of Equation 2.2, and Parkin comments that for many purposes this approximation may be adequate for predictive purposes. He further indicates that problems with interpretation or prediction of settlement versus time records using Equation 2.2 is in the incorrect establishment of the initial time (t_o) or where m is not equal to 1 from the rate process equation, which will result in curvature in the settlement versus log time plot.

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$$\varepsilon = a(t - t_o)^{-m} \tag{2.4}$$

where $\dot{\varepsilon}$ = settlement rate in percent/month, a = constant, t_o = initial or origin of time, t = time in months after t_o and m = slope of the settlement rate versus time plot.

Charles (1986) proposed a settlement index, S_l , for assessment of the long-term crest settlement behaviour of puddle core earthfill embankments (Equation 2.5). Charles and Tedd (1991) point out that the settlement index is analogous to the coefficient of secondary consolidation for a clay soil. They provide values of the settlement index for a number of puddle core embankments during the period of normal reservoir operation. Tedd et al (1997b) suggest that values of settlement index "greater than 0.02 could indicate some mechanism other than creep was causing the settlement and that the situation should be seriously examined".

$$S_I = \frac{s}{1000 \cdot H \cdot \log(t_2/t_1)} \tag{2.5}$$

where s is the crest settlement in millimetres measured between times t_2 and t_1 after the completion of embankment construction, and H is the height of the dam in metres. The equation is of the same form as Equation 2.2.

2.4.2 Numerical Methods

Duncan (1996) and Kovacevic (1994) are recent references on the state of the art in finite element analyses applicable to the deformation behaviour of embankments, mainly zoned earth and rockfill dams. They discuss the methods of analysis, their limitations, available constitutive models of the stress-strain relationship and areas of uncertainty. As Duncan points out, most analyses from the literature are Class C1 (Lambe 1973), i.e. after the event, which may account for the generally good agreement between predicted and observed deformation behaviour.

Duncan (1996) comments that the choice of stress-strain constitutive model used in the analysis is a balance between simplicity and accuracy, and that the choice of constitutive model will depend on the purpose of the modelling. He commented that if the purpose of the analysis is to analyse stresses and/or the trend of deformations then more simplistic models could be suitable, depending on the relative stiffness between different embankment materials. More accurate modelling of deformations requires a more complex stress-strain model that more realistically approximates the real behaviour of the rockfill and earthfill.

Several other important aspects of finite element analysis with respect to modelling dam embankments raised by Duncan (1996) and Kovacevic (1994) include:

- The importance of modelling the construction as a series incremental layers
- The type of stress-strain relationship used for modelling materials. Duncan (1996) comments that elastic stress-strain models (e.g. linear elastic, multi-linear elastic or hyperbolic models) are appropriate where the soils are not stressed to failure, but, that these models are not suitable for modelling undrained behaviour or problems where local failure occurs. He recommends the use of models that incorporate plasticity theory for these situations (e.g. for wet placed earthfill cores of thin to medium width in zoned embankments).
- For situations where the earthfills are not stressed to failure, linear elastic models are generally not suitable for accurate modelling of deformation behaviour due to the non-linear stress-strain relationship of earthfill and rockfill (Figure 2.1). For these situations, the use hyperbolic or multi-linear elastic models provide improved predictions of deformation and have been successfully used, particularly for deformation prediction during embankment construction.
- A limitation of hyperbolic and multi-linear elastic models is that the model parameters are typically derived from triaxial compression or oedometer laboratory tests and are therefore limited by the narrow range of stress paths covered in comparison to the broader range of stress paths imposed in the field. Kovacevic (1994) highlighted the limitations of multi-linear elastic models to accurately model the deformation of the face slab of concrete face rockfill dams (CFRD) on reservoir impoundment. This would also apply to zones in other embankment types (e.g. the downstream shoulder in central core earth and rockfill dams) where the stress path on impoundment is markedly different to that in the triaxial compression or oedometer test.
- Kovacevic (1994) found that elasto-plastic models that model pre-peak plasticity were more suited for
 modelling the deformation behaviour of the upstream face of CFRD during reservoir impoundment due to
 the ability of these constitutive models to more realistically account for the rockfill deformation under the
 stress path conditions imposed.
- A significant factor of uncertainty associated with Class A type predictions based on laboratory testing is
 the difference in material properties between the laboratory test results and those in the field due to
 limitations on the maximum particle size that can be tested and variations in stiffness due to differences in
 material quality, compacted density and moisture content.

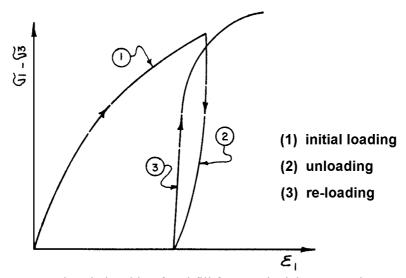


Figure 2.1: Typical stress-strain relationship of rockfill from a triaxial compression test (Mori and Pinto 1988)

A further problem with laboratory testing for determining the compressibility parameters for rockfills is the limitation of representing the layering that occurs within the layer. The method of field placement and compaction of rockfills generally results in variations in density, modulus and grading with depth within the compacted layer.

An important component of the modelling of embankment dams is the consideration of collapse compression of susceptible rockfills and earthfills on wetting. The effects of collapse compression are most noted for the upstream shoulder on initial impoundment, but collapse compression has also been observed in the downstream shoulder following wetting due to rainfall, leakage or tail-water impoundment. Incorporating collapse compression into constitutive models adds further complexity and greater uncertainty in the estimation of material parameters between laboratory and field conditions due to the dependency of collapse compression on compaction moisture content, compacted density, applied stress conditions and material properties. Justo (1991) and Naylor et al (1989) propose methods for incorporation of collapse compression of rockfill into constitutive models. The analysis of Beliche Dam, a central core earth and rockfill dam, by Naylor et al (1997) is an example where collapse compression of the upstream rockfill was considered in the modelling.

The effect of pore water pressure dissipation in earthfills during construction has been considered by a number of authors including Eisenstein and Law (1977), in modelling Mica dam, and by Cavounidis and Höeg (1977), amongst others. For these cases, the incremental embankment construction was modelled as a two stage process, the first stage modelling the new layer construction using undrained properties for the core and the second stage modelling pore water pressure dissipation. In most cases though, pore water pressure development in the core is ignored. For wet placed earthfills, where high pore water pressures are developed during construction, the core is often modelled using undrained strength and compressibility parameters, and the permeability is assumed as sufficiently low such that pore water pressures will not dissipate during the period of construction. For dry placed earthfills, the earthfills are often modelled using elastic models (e.g. multi-linear elastic or hyperbolic elastic). In some cases, the earthfill is analysed in terms of effective stresses and using elasto-plastic models with pore water pressures input into the model (e.g. Naylor et al (1997) for the core of Beliche dam). In this case, the analysis was undertaken after the embankment had been constructed and actual pore pressures were used to derive the input to the model.

3.0 DATABASE OF CASE STUDIES

The case study database comprises some 134 embankments; 63 central core earth and rockfill dams, 21 zoned earth and rockfill dams, 23 zoned earthfill dams, 10 earthfill dams and 17 puddle core earthfill embankments. A summary of the case studies in the database is presented in Table 3.1 to Table 3.5.

3.1 EARTHFILL AND ZONED EARTH AND EARTH-ROCKFILL EMBANKMENTS

The type of rolled earthfill and zoned earth and earth-rockfill embankments (Figure 1.1) in the case study database include:

- Central core earth and rockfill embankments (Table 3.1), this is the dominant embankment type
- Zoned earth and rockfill embankments (Table 3.2)
- Zoned earthfill embankments (Table 3.3)
- Earthfill embankments (Table 3.4), including homogeneous earthfill, earthfill with filters and earthfill with rock toe.

Embankments with unusual design or of design with limited numbers of case studies available from the literature were excluded. Sloping core zoned earth and rockfill embankments have not been considered.

Further details on most of the case studies are given in Appendix A (Tables A1.1 to A1.4), including a summary of the embankment details, material types and their method of placement, reservoir operation, hydrogeology, monitoring (during and post construction) and references for each dam. In most embankments the main earthfill zone acting as the water barrier has generally been placed in thin layers and well compacted using rollers suitable for the earthfill type.

Approximately half of the earthfill and zoned earth and earth-rockfill case studies are embankments owned or supervised by sponsors and in-kind sponsors of the research project. The authors are most grateful to these organizations for their assistance and making available information on the materials, construction procedures and instrumentation records. For most of these embankments the deformation records are of good quality. These case studies have been supplemented by case studies from the published literature for which the deformation records were of reasonably to good quality, and a reasonable level of detail was available on the embankment design, material types and construction methods.

3.2 PUDDLE CORE EARTHFILL EMBANKMENTS

A database of seventeen case studies of puddle core earthfill dams (3 from Australian and 14 from the UK) has been gathered for analysis of deformation behaviour (Table 3.5). Further details on the case studies are given in Appendix D, including a summary of the embankment details, reservoir operation, hydrogeology and monitoring for each dam. Most of the available deformation records are from surface measurement points (SMP) established many years after construction, in the 1970's to 1980's for a number of the case studies. Some monitoring data and anecdotal information is available on the early performance of several embankments. Compared to the rolled earthfill and zoned earth and earth-rockfill embankment case studies, the monitoring records for the puddle core earthfill dam data set is fairly limited, but to be expected given the age of most of the dams.

Table 3.1: Central core earth and rockfill embankments in the database

Dore Nove	1	Height, H	Embank	ment Classif	ication	Rockfill		
Dam Name	Country *1	(m)	Classification Core Size Core Type			Source Compaction Rating		
Agigawa	Japan	102	5,1,1	c-tm	CL	-	good (?)	
Ajuare	Sweden	46	5,2,2	c-tn	SM	schists & gneisses	poor	
Ataturk	Turkey	184	5,2,0	c-tm	CH	basalt & limestone	reasonable (?)	
Bath County Upper Dam	USA, Virginia	134	5,2,0	c-tm	SC (?)	siltstone & sandstone	good (?)	
Beliche	Portugal	55	5,2,2	c-tn	GC	schists & greywacke	poor (?)	
Bellfield	Australia, Vic.	40	5,2,0	c-tm	CL/SC	sandstone	poor, dry placed	
Bjelki-Peterson	Australia, Qld	41.5	5,2,1	c-tn	CL	phyllite	good	
Blowering	Australia, NSW	112	5,2,0	c-tm	SC - CL	quartzite & phyllite	good	
Chaffey	Australia, NSW	54	5,2,0	c-tm	CH/GC	jasper	reasonable to good	
Cherry Valley	USA, San Francisco	100	5,2,0	c-tk	SM/ML	granite & granodiorite	dumped and sluiced	
Chicoasen	Mexico	261	5,2,0	c-tn	GC	limestone	good	
Copeton	Australia, NSW	113	5,2,0	c-tm	SC	granite	reasonable to good, dry place	
Corin	Australia, ACT	74	5,2,0	c-tm	SM	quartzite, sandstone & siltstone	good	
Cougar	USA, Oregon	159	5,2,0	c-tn (u)	GM	basalt and andesite	good (downstream), reasonab (upstream)	
Dalesice	Checkoslovakia	90	5,2,0	c-tn (u)	CL	_	good	
Dartmouth	Australia, Vic.	180	5,2,0	c-tm	SC/SM	granitic gneiss	good, dry placed	
Djatiluhur	Indonesia	105	5,2,0	c-tm (u)	CH	andesite	poor	
Eildon	Australia, Vic.	79	5,1,0	c-tk	CL	quartzitic sandstone	poor, dry placed (?)	
Liidon	Australia, Vic.	7.5	3,1,0	C-tK		diorite & silicified	poor, dry placed (!)	
El Infiernillo	Mexico	148	5,2,0	c-tn	CL-CH	conglomerate	poor to reasonable, dry place	
Eppalock	Australia, Vic.	47	5,2,0	c-tk	CL	basalt	poor, dry placed	
Frauenau	Germany	80	5,2,0	c-tn	SM	-	-	
Fukada	Japan	55.5	4,1,2	c-tm	GC (?)	sandstone & granite.	good (?)	
					,		reasonable to good (inner), po	
Geehi	Australia, NSW	91	5,2,0	c-tn	SM	granitic gneiss	(outer)	
Gepatsch	Austria	153	5,2,0	c-tn	GM/GC	gneiss	reasonable, dry placed	
Glenbawn Saddle A	Australia, NSW	35	5,2,0	c-tm	-	limestone & sandstone	good (?)	
Glenbawn - main dam (original, prior to raising)	Australia, NSW	76.5	5,1,0	c-tk	CL	limestone	poor, dry placed	
Googong (before raising)	Australia, ACT	62	5,2,0	c-tm	SC	dacite & granite	good, dry placed	
	Adoliana, Ato i							
Kisenyama	Japan	88	5,2,0	c-tm	-	slate, chert & sandstone	good (?)	
Kurokawa	Japan	98	5,2,0	c-tn (u)	-	-	good (?)	
La Grande 2 (LG-2)	Canada, James Bay	160	5,2,0	c-tn (u)	SM	granite ?	reasonable to good, dry place	
Maroon	Australia, Qld	52	5,2,2	c-tk	SC/CL	porphery	reasonable to good	
Matahina	New Zealand	85	5,1,0	c-tm	CL/ML	ignimbrite	poor, dry placed	
Mud Mountain	USA, Washington	128	5,1,0	c-tm	SM/GM	andesite & tuff	dumped and sluiced	
Naramata	Japan	158	5,1,0	c-tn	GM/GC	rhyolite & granite	good	
Netzahualcoyotl	Mexico	132	5,2,0	c-tn	MH	conglomerate	poor to reasonable.	
Nillahcootie	Australia, Vic.	35	5,2,0	c-tm	CL	sandstone & mudstone	poor (?)	
Nottely	USA, Tennessee	56	5,1,1	c-tk	CL	quartzite & mica schist	dumped and sluiced	
Parangana	Australia, Tas.	53	5,2,2	c-tm	SM	quartzite & schist	reasonable to good, dry place	
						hornfelsed andesite &		
Peter Faust	Australia, Qld	51	5,2,0	c-tn	CL	andesitic tuff	reasonable to good	
Round Butte	USA, Oregon	134	5,2,0	c-tn (u)	SM	basalt	reasonable to good, dry place	
Rowallan	Australia, Tas.	43	5,1,0	c-tn	GM	quartzite	reasonable to good, dry place	
Sagae	Japan	112	5,2,0	c-tn	_	-	good (?)	
Seto	Japan	102	5,1,0	c-tn	-	-	good (?)	
Shimokotori	Japan	119	5,1,0	c-tn	-	-	good (?)	
	Australia, Qld.					brecciated quartzites &		
Split Yard Creek		76	5,2,1	c-tm	CL	greenstone	reasonable	
South Holson	USA, Tennessee	87	5,2,1	c-tk	CL	sandstone, some shale	dumped and sluiced	
Srinagarind	Thailand	140	5,2,0	c-tn	GC/SC	limestone	good	
Svartevann	Norway	129	5,2,0	c-tn (u)	GM	granitic gneiss	reasonable, dry placed	
Taisetsu	Japan, Hokkaido	86.5	5,1,0	c-tn	-	-	good (?)	
Talbingo	Australia, NSW	162	5,2,0	c-tm (u)	CL	rhyolite & porphyry	good	
Tedorigawa	Japan, Honshu Island	153	5,2,0	c-tn	GM/GC	-	good (?)	
Terauchi	Japan, Fukuada	83	5,2,0	c-tm	SC/SM	lutaceous schist	good	
Thomson	Australia, Vic.	166	5,2,0	c-tm	SC	sandstone & siltstone	good, dry placed	
Tokachi	Japan, Hokkaido	84.3	5,1,0	c-tm	GC	keratophyre, 30% slate	good (?)	
Upper Dam	Korea	88.5	5,2,0	c-tn	GC	tuff & rhyolite	poor (?)	
Upper Yarra	Australia, Vic.	90	5,0,0	c-tk	CL	sandstone & siltstone	poor, dry placed	
	Norway	121	5,2,0	c-tn	SM	quartz, granite & gneiss	reasonable, dry placed	
Vatnedalsvatn	New Zealand	36	5,1,1	c-tk	CH/SC	ignimbrite	poor	
Vatnedalsvatn Waipapa				c-tk	CL/SC	quartzite	dumped and sluiced	
Waipapa		94			JLIJU	quai iziic	aarripea aria sialeed	
Waipapa Watuaga	USA, Tennessee	94 34	5,2,1 5,2,0		MI	rhyodacite	reasonable to good	
Waipapa Watuaga William Hovell	USA, Tennessee Australia, Vic.	34	5,2,0	c-tm	ML	rhyodacite	reasonable to good (inner)	
Waipapa Watuaga	USA, Tennessee	34 67	5,2,0 5,2,1		CL	rhyodacite andesite	•	
Waipapa Watuaga William Hovell	USA, Tennessee Australia, Vic.	34	5,2,0	c-tm		·	reasonable to good (inner),	

Note: *1 For Australian dams; Qld = Queensland, NSW = New South Wales, Vic. = Victoria, Tas. = Tasmania, ACT = Australian Capital Territory

Table 3.2: Zoned earth and rockfill embankment case studies

Dam Name	0		Embank	ment Classif	Comment	
Dam Name	Country	Height (m)	Classification	Core Size	Core Type	Comment
Andong	Korea	83	4,1,1	c-tn	SM	Rockfill used in outer shoulders
Bradbury	USA, California	85	4,1,1	c-tk	SC/GC	Weathered shale in downstream shoulder
Burrendong	Australia, NSW	76	4,2,1	c-tk	CL	Rockfill in up and downstream shoulder, poorly compacted.
Canales	Spain	156	4,1,0	c-tn	СН	Limestone rockfill in up and downstream shoulder
Cardinia	Australia, Vic.	86	4,2,1	c-tm (u)	SC/CL	granodiorite in upstream shoulder
Dixon Canyon	USA, Colorado	74	4,1,0	c-vb	CL	thin outer rockfill zones. Virtually homogeneous.
Eucembene	Australia, NSW	116	4,1,0	c-vb	SC	Quartzite rockfill as thin outer zones.
Greenvale	Australia, Vic.	52	4,2,2	c-tk	SM	granodiorite rockfill in upstream shoulder
Jackson Gulch	USA, Colorado	56	4,0,1	c-tm (u)	CL	Broad GC earthfill zones up and downstream of the core, and thin outer rockfill shoulders
La Agnostura	Mexico	146	4,0,0	c-tn	CL	poor quality limestone used in up and downstream shoulders
La Grande - LG4	Canada	125	4,1,0	c-tm	SM	granite and gneiss used in up and downstream shoulders
Long Lake	USA	39	4,0,0	c-tk	CL/ML	Broad earthfill zones up and downstream of core and thin outer rockfill zones.
Nurek	Russia	289	4,2,0	c-tn	GC/GM	thin outer rockfill zone
Peublo (right abutment)	USA, Colorado	51	4,0,1	c-vb	CL	limited use of rockfill
Rector Creek	USA, California	61	4,0,0	c-vb	SC/SM	limited rockfill used, outer shoulders. Close to homogeneous classification.
San Justo	USA, California	41	4,2,2	c-tk	CL	limestone rockfill used upstream shoulder
San Luis - Main Dam	USA, California	116	4,2,1	c-vb	CL	limited rockfill used, outer shoulders
San Luis - Slide Area	USA, California	30 to 45	4,1,1	c-vb	CL	limited rockfill used, outer shoulders
Soyang	Korea	123	4,1,0	c-tm	GC	poor ly compacted rockfill used in outer shoulders
Spring Canyon	USA, Colorado	68	4,1,0	c-vb	CL	limited rockfill used, outer shoulders
Tooma	Australia, NSW	67	4,1,2	c-tk	SM	dumped and sluiced granite rockfill in upstream shoulder
Tullaroop	Australia, Vic.	41	4,1,0	c-tk	CL	limited rockfill used, at up and downstream toe regions. Could also classify as 2,1,0.

Table 3.3: Zoned earthfill embankment case studies

		Height	Embank	ment Classif	ication	
Dam Name	Country	(m)	Classification	Core Size	Core Type	Comment
Benmore	New Zealand	110	3,2,1	c-tk	GM/GC	thick core with compacted gravel shoulders
Cairn Curran	Australia, Victoria	44	3,0,0	c-vb	CL/ML	embankment with very broad core
Carsington	England	36	3,0,1	c-tm	СН	wet placed core supported by shoulders of well- compacted weathered mudstones. Failed during construction.
Cobb	New Zealand	35	3,0,1	c-tk	GM/GP	thick core with gravel upstream shoulder and talus downstream shoulder
Davis	USA, Arizona	60	3,0,1	c-tm (u)	CL	upstream sloping medium width core supported by broad silty sand to silty gravel earthfill zones.
Deer Creek	USA, Utah	71	3,0,0	c-tk	CL-GC	central core of thick width with earthfill shoulders
Granby	USA, Colorado	88	3,0,0	c-vb	SC-GC	embankment with very broad core
Hirakud	India	59	3,0,1	c-vb	SC-GC	embankment with very broad core
Horsetooth	USA, Colorado	48	3,0,0	c-vb	CL	embankment with very broad core
Kastraki	Greece	95	3,1,0	c-tn	CL	thin clay core with well compacted gravel shoulders
Khancoban	Australia, New South Wales	18	3,0,2	c-vb	SM	embankment with very broad core, upstream gravel shoulder.
Kremasta	Greece	165	3,1,0	c-tn	CL	thin clay core with well compacted gravel shoulders
Mammoth Pool	USA, California	113	3,2,2	c-tk	SM	thick core with compacted earthfill shoulders
Medicine Creek	USA, Nebraska	48	3,0,0	c-vb	CL	embankment with very broad core
Meeks Cabin	USA, Wyoming	57.5	3,1,1	c-tk	CL	thick core with gravel shoulders
Navajo	USA, New Mexico	123	3,1,1	c-tk	CL	thick core with compacted gravel and earthfill shoulders
O'Sullivan	USA, Washington	64	3,1,0	c-tk	SM	central core of thick width with filter / transition zones up and downstream of the core
Pukaki	New Zealand	76	3,2,1	c-tk	GM	thick core with compacted gravel shoulders
Rosshaupten	Germany	41	3,1,1	c-tm	GM/ML	medium width core with compacted gravel shoulders
Ruataniwha	New Zealand	50	3,2,1	c-tk	GM	thick core with well compacted gravel shoulders
Soldier Canyon	USA, Colorado	70	3,0,0	c-vb	CL	embankment with very broad core
Steinaker	USA, Utah	49	3,0,0	c-vb	CL	embankment with very broad core
Trinity	USA, California	164	3,0,0	c-tk	SM/GM	thick core with gravel shoulders

Table 3.4: Earthfill embankment case studies

- 20 -

			Embankm	ent Classificatio		
Name	Name Country H		Dam Type	Classification	Core Type	Comment
Belle Fourche	USA, South Dakota	35	Homogeneous Earthfill	0,0,0	CL-CH	concrete facing on upstream slope
Biet Netufa	Israel	11	Earthfill with rock toe	2,0,0	CH	
Bonny	USA, Colorado	44	Earthfill with foundation filter	1,0,1	ML-SM	Foundation filter under downstream shoulder
Enders	USA, Nebraska	31.5	Earthfill with thin rockfill shell	2,0,0	SM/ML	virtually a homogeneous embankment
Fresno	USA, Montana	24	Earthfill with rock toe	2,0,0	ML-CL	
Heart Butte	USA, North Dakota	42	Earthfill with rock/gravel toe	2,0,0	SC-SM	
Mita Hills	Africa, Zambia	49	Earthfill with filters	1,2,1	CL-ML (?)	upstream sloping dual chimney filter
Peublo (left abutment embankment)	USA, Colorado	37	Earthfill, rockfill toe	1,0,1 (or 3,0,1)	CL	thin downstream outer shoulder zone of gravels. Stability berm on downstream shoulder, placed 1981.
Roxo Dam	Portugal	34	Earthfill, rockfill toe	2,0,0	CL/SC	Part earthfill part concrete gravity & buttress dam.
Sasumua	Kenya	35	Earthfill with filters	1,2,0	МН	central chimney filter (dual filters) and large rockfill zones at up and downstream toe.

Table 3.5: Puddle core earthfill embankment case studies

				Core Din	nensions	Puddle I	Material	Select	
Name	Country	Year Completed	Height (m)	Top Width (m)	Core Slope (H to V)	Source	Class ⁿ	Zone Yes/No	General Earthfill
Burnhope	UK	1935	41	nk	nk	Boulder Clay	CL	Yes	hard mudstone - 450 mm layers, compacted by steam rollers
Challocombe	UK	1944	15	1.2	1 to 15	nk	ML	Yes	nk
Cwnwernderi	UK	1901	22	1.5	1 to 10	nk	CL	No?	nk
Dean Head	UK	1840	19	nk	nk	nk	nk	nk	nk
Happy Valley	Australia	1896	25	2.44	1 to 8.25	residual	CL	Yes	Residual clays (CL) - 150 mm layers compacted by wagons, carts and grooved rollers
Hollowell	UK	1937	13.1	2	1 to 10	Upper Lias - blue clay	nk	Yes	Sand and sand-rock, placed in thin layers by dozer
Holmestyes	UK	1840	25	2	1 to 20	Boulder Clay	nk	No	Weathered mudstones & sandstones, mix sandy silty clay with gravel to boulder size fragments
Hope Valley	Australia	1872	21	1.8	1 to 12	alluvial / fluvial	СН	Yes (u/s fill and d/s inner)	alluvial / fluvial (downstream only), SC/SM, placed in 1200 mm layers compacted by 3.5t sheepsfoot roller
Ladybower	UK	1945	43	nk	nk	nk	nk	nk	nk
Langsett	UK	1904	33	nk	nk	nk	Clay	Yes	Not sure, possibly weathered shale, placed in 900 mm lifts and compacted by slewing of the rails
Ogden	UK	1858	25	2	1 to 10	Boulder Clay	СН	No	Weathered mudstones & sandstones, mix sandy silty clay with gravel to boulder size fragments
Ramsden	UK	1883	25	3	1 to 12	Boulder Clay	ML/MH	Yes	Weathered mudstones & sandstones, mix sandy silty clay with gravel to boulder size fragments
Selset	UK	1959	39	1.5	1 to 20	Boulder Clay	CL	Yes	Boulder Clay (CL) sandy clay, placed in 225 mm layers by dozers and 13.6 t rollers
Walshaw Dean	UK	1907	22	2.6	1 to 12	Boulder Clay	CL	No	Weathered mudstones & sandstones, mix sandy silty clay with gravel to boulder size fragments
Widdop	UK	1878	20	nk	nk	nk	nk	No	nk
Yan Yean	Australia	1857	9.6	3.05	1 to 3.3	alluvium	CL/CH	No	alluvium, CL/CH clays upstream and ML/CL sandy silts and silty clays downstream, placed in 600 to 1200 mm lifts
Yateholme	UK	1872	17	nk	nk	Boulder Clay	CL	No	Weathered mudstones & sandstones, mix sandy silty clay with gravel to boulder size fragments

Note: Classification of soil types is to Australian Standard AS 1276 - 1993: Geotechnical Site Investigations.

nk = not known

H to V = horizontal to vertical

4.0 GENERAL DEFORMATION BEHAVIOUR OF EARTHFILL AND ZONED EARTH AND EARTH-ROCKFILL EMBANKMENTS

The type and number of earthfill and zoned earth and earth-rockfill embankments used for the deformation analysis are summarised in Section 3.1. As indicated, embankments of unusual design or designs with limited numbers of case studies were excluded and sloping core zoned earth and rockfill embankments have not been considered.

For virtually all of the embankments considered the main earthfill zone/s have generally been placed in thin layers and well compacted in line with modern practice. In some of the older embankments the moisture content control of the earthfill was possibly not as stringent as it would have been with modern practice, but this has not necessarily been detrimental to the embankment performance.

The deformation behaviour has been sub-grouped into deformation during construction and deformation post construction. The deformation behaviour during first filling is discussed as a separate section but the plotted deformation behaviour has, in most cases, been incorporated into plots of the post construction deformation behaviour.

4.1 DEFORMATION DURING CONSTRUCTION OF EARTH AND EARTH-ROCKFILL EMBANKMENTS

The analysis of the deformation behaviour during construction of earthfill and earth-rockfill embankments is mostly concentrated on the central earthfill core zone of the embankment because this is where instruments are located. Most of the deformation data collected and analysed is vertical deformations, usually from internal vertical settlement gauges (IVM) installed in the core as construction proceeds. Data on horizontal deformations, usually from internal horizontal movement gauges (IHM) or inclinometers, is less frequently monitored, but has been collected and analysed for a number of embankments.

Analysis of deformation requires consideration of the stress conditions imposed during construction and the stress-strain relationship of the materials used in embankment construction. In zoned embankment construction it is also important to consider both the total and effective stress conditions, and the interaction between the different material zones in the analysis. While finite element analysis can be used for individual dams, this was impracticable for this study due to the number of case studies analysed. Therefore, simplified approaches have been used with reference to finite difference or finite element analysis to indicate the generalised stress conditions and deformation behaviour.

4.1.1 Stresses During Construction

4.1.1.1 Stresses in a Homogeneous Embankment on a Rigid Foundation

For the simplified case of a homogeneous embankment on a rigid foundation and assuming elastic behaviour, the vertical and lateral stresses within the embankment are dependent on the slope geometry, and the total unit weight and Poisson's ratio of the earthfill.

During the initial stages of embankment construction the embankment width is much greater than the height of earthfill. Under these conditions, assuming plane strain conditions along the dam axis, the stresses below the dam axis can be modelled assuming zero lateral strain normal to the dam axis (Figure 4.1a). The elastic stress-

strain relationships for a soil element under the embankment axis following placement of an incremental layer of distributed load p kPa are approximated as:

$$\varepsilon_x = \varepsilon_y = 0$$

$$\Delta \sigma_z = p \; ; \qquad \Delta \sigma_x = \Delta \sigma_y = \frac{v \, \Delta \sigma_z}{(1 - v)}$$

constrained tangent modulus, $\Delta D = \Delta \sigma_z / \Delta \varepsilon_z$

Young's tangent modulus,
$$\Delta E = \frac{\Delta D (1 + v) (1 - 2v)}{(1 - v)}$$

where ε = strain, σ = stress, x, y and z are axis co-ordinates, v is Poisson's ratio and Δ represents the incremental or tangential component of the parameter.

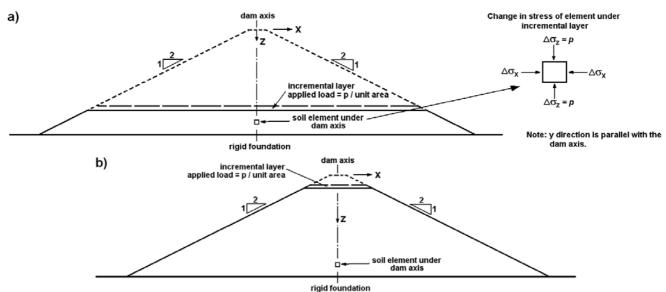


Figure 4.1: Embankment construction indicating (a) broad layer width to embankment depth ratio in the early stages of construction and (b) narrow layer width to embankment depth ratio in the latter stages of construction.

Because of geometry effects, this approximation is not sufficiently accurate for estimating the change in stresses in a soil element low in the dam in the latter stages of construction where the layer width is small compared to the constructed embankment height (Figure 4.1b).

To facilitate analysis of the data, total vertical stress profiles during and at the end of construction were estimated for a variety of symmetrical embankment shapes by elastic finite difference analysis. Figure 4.2a presents a typical total vertical stress profile at end of construction for an embankment with slopes of 1.8H to 1V and Figure 4.2b the ratio of total vertical stress at the end of construction for various embankment slopes compared to the total vertical stress calculated simply by the depth below the ground surface and the unit weight (i.e. γH). Lines of best fit were obtained using a second order polynomial equation according to Equation 4.1. Values of the coefficients A_1 and A_2 for the various embankment slopes analysed are given in Table 4.1.

$$\sigma_{z,EOC} = \gamma \cdot H \left(A_1 \left(h/H \right)^2 + A_2 \left(h/H \right) \right)$$

$$= Z_1 \cdot \gamma \cdot H$$
(4.1)

where $\sigma_{z,EOC}$ = total vertical stress at end of construction (at depth h/H), γ = bulk density, H = embankment height, h = depth below crest, A_1 and A_2 are coefficients and Z_1 is the vertical stress ratio.

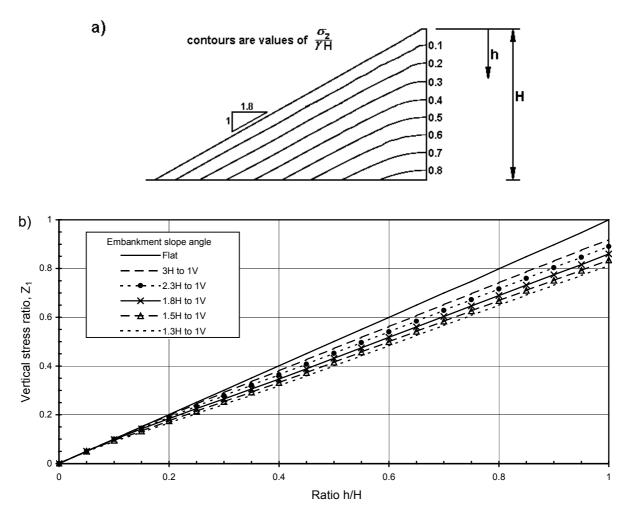


Figure 4.2: (a) Total vertical stress ($\sigma_{z,EOC}$) contours at end of construction for 1.8H to 1V embankment slopes, and (b) vertical stress ratio, Z_1 , versus h/H under the embankment axis for various embankment slopes.

Table 4.1: Multiplying coefficients for the calculation of the total vertical stress at the embankment centreline at end of construction to the embankment height ratio (h/H) according to Equation 4.1.

Embankment Slope Angle	\mathbf{A}_1	A_2
1.3H to 1V	0.0084	0.803
1.5H to 1V	-0.0098	0.843
1.8H to 1V	-0.029	0.886
2.3H to 1V	-0.044	0.931
3.0H to 1V	-0.062	0.978

For estimation of stresses within the embankment during construction finite element or finite difference methods can be used. Alternatively, published charts such as those by Clough and Woodward (1967) or Poulos et al (1972) can be used.

4.1.1.2 Stresses in Zoned Embankments

Zoned embankments present a greater degree of complexity for analysis than the homogeneous embankment case because of the potential for stress transfer or shedding of stresses from the core to the shoulder or vice

versa. Simplification to a homogeneous analysis is only acceptable where arching has a negligible influence on the vertical stress profile. As demonstrated in Section 4.1.1.3 this is a reasonable assumption for:

- Zoned embankments with broad central earthfill zones, e.g., for central core widths with a combined core slope (upstream and downstream core slope combined) greater than about 1.5 to 2H to 1V. It may also be a reasonable simplification for combined central core widths down to 1H to 1V where the well-compacted core has been placed on the dry side of Standard optimum moisture content, giving a core of high undrained strength so that the stress conditions in the core are maintained in the elastic region.
- Zoned embankments where the compressibility properties of the central earthfill zone and gravelly or
 rockfill shoulders are similar; i.e., compacted sandy and gravelly soils with non-plastic fines or low (less
 than about 20% finer than 75 micron) plastic fines contents, with shoulders of compacted gravels or rockfill.
 This is on the proviso that pore pressures generated in the core during construction are small and potential
 plastic type core deformations due to lateral spreading of the core are therefore negligible.

Where the core is of thin to medium width and the compressibility properties of the core and shoulders are dissimilar it is not possible to use the simple methods described in Section 4.1.1.1 to analyse the deformation of the core. The problem is no longer one-dimensional because the vertical strain of the core under the embankment centreline comprises two components, one due to the vertical strain under the imposed vertical stress and a second component due to the lateral strain of the core. In addition, stress transfer effects cannot be ignored. Meaningful analysis of stresses and strains during construction can only be undertaken using finite element modelling based on constitutive models that reasonably approximate the as placed properties of the various materials used in construction.

However, to facilitate the analysis of the case study deformation data, the analysis of a simple plane strain finite difference model of a zoned embankment was undertaken to assess the general stress conditions and trend of deformation behaviour during construction. The modelled embankment (Figure 4.3) consisted of a 100 m high central core earth and rockfill dam, of thin core width, on a rigid foundation. Construction was modelled in ten lifts each of 10 m height. A Mohr-Coulomb linear-elastic perfectly plastic model was used for both the central core and rockfill. Three analyses were carried out each using the same properties for the rockfill (Figure 4.3) and varying the properties of the core material (Table 4.2) as follows:

- Case 1 core with similar shear strength properties to the rockfill. Poisson's ratio was assumed to be the same for the core and rockfill shoulders. This case is considered typical of a well-compacted silty gravel core with non-plastic fines.
- Case 2 A very stiff core with Poisson's ratio much greater than that of the rockfill (0.40 compared to 0.22) and strength indicative of the undrained strength envelope.
- Case 3 A wet placed core. The core is assumed to behave nearly fully undrained with a Poisson's ratio of 0.48 and a small equivalent undrained friction angle of 5 degrees.

The same modulus has been used for the core (100 MPa) and the rockfill (80 MPa) in all three cases to minimise where possible the number of variants. A friction angle of 45 degrees is used to model the rockfill.

The results, shown in Figure 4.4 and Figure 4.5, highlight the influence of the difference in shear strength and Poisson's ratio of the core on the stresses within the embankment and on the deformation behaviour. Where the properties of the core are similar to those of the rockfill (Case 1) the lateral stress distribution is similar to the homogeneous case and lateral displacements in the core and rockfill are small (Figure 4.4 and Figure 4.5c). The vertical stress distribution on the embankment centreline for this case (Case 1) is similar to the homogeneous case, except that the higher modulus core attracts higher stresses during construction. The assumption of homogeneous conditions reasonably models the lateral stresses but will under-estimate the vertical stresses in the core where the core is less compressible than the rockfill, and vice versa if the core is more compressible

than the rockfill. For broader core widths the vertical stress profile on the centreline approaches the homogeneous case.

Where the stress-strain properties of the core are different to those of the rockfill, such as for Cases 2 and 3, the stress profiles (Figure 4.4) differ significantly from the homogeneous case with much higher lateral stresses developed during construction as well as arching across the core. Outward lateral displacements of the core and rockfill are observed that are much larger than for Case 1 or for the homogeneous case (Figure 4.4 and Figure 4.5c). The displacements are caused by the differential lateral stress conditions that develop between the core and shoulders. For Case 3, yield conditions are reached in a large portion of the core and in this case the rockfill is in effect acting as support for the core.

The results also show:

- Lateral stresses and strains increase with increasing Poisson's ratio of the core, reflecting yielding and plastic deformation of the core, particularly for Case 3.
- Maximum lateral strains are observed at a depth below the crest of approximately 65% of the embankment height (h/H = 0.65).
- The lateral strains, which occur largely as plastic deformation of the core, contribute significantly to the vertical strain observed in the core.
- Arching occurs in the narrow core as a result of the large differential vertical deformation between the core and the shoulders. Figure 4.5a shows the vertical stress profile at end of construction for Case 3 is much less than for the homogeneous condition, and Cases 1 and 2.

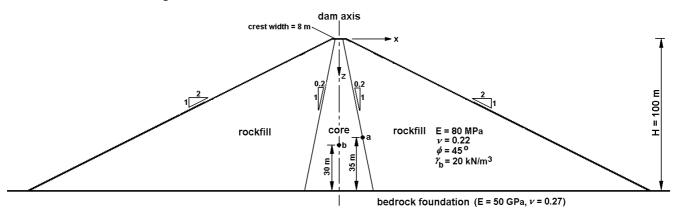


Figure 4.3: Embankment model for finite difference analysis during construction.

Table 4.2: Properties of central core used in finite difference analyses

	Modulus, E	Poisson's	Bulk Unit	Cohesion, c	Angle of Internal
Analysed Case	(MPa)	ratio, ν	Weight, γ	(kPa)	Friction, ϕ
			(kN/m^3)		(degrees)
Case 1 – zoned embankment with core similar to rockfill	100	0.22	20	0	45
Case 2 – zoned embankment with very stiff core	100	0.4	20	75	20
Case 3 - zoned embankment with stiff, wet core	100	0.48	20	30	5
Homogeneous (same as rockfill)	80	0.22	20	0	45
Cases 4 to 7 – zoned embankment, core slope 0.2H to 1V to 0.75H to 1V (refer Section 4.1.1.3)	40	0.33	20	2	32

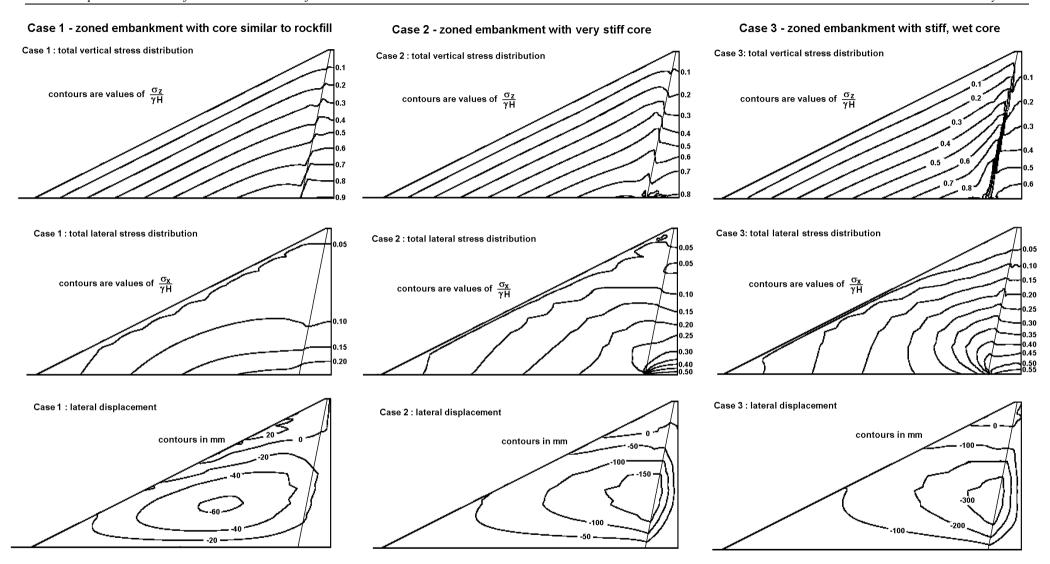


Figure 4.4: Total vertical stress, total lateral stress, and lateral (horizontal) displacement distributions at end of construction for zoned earthfill finite difference analysis.

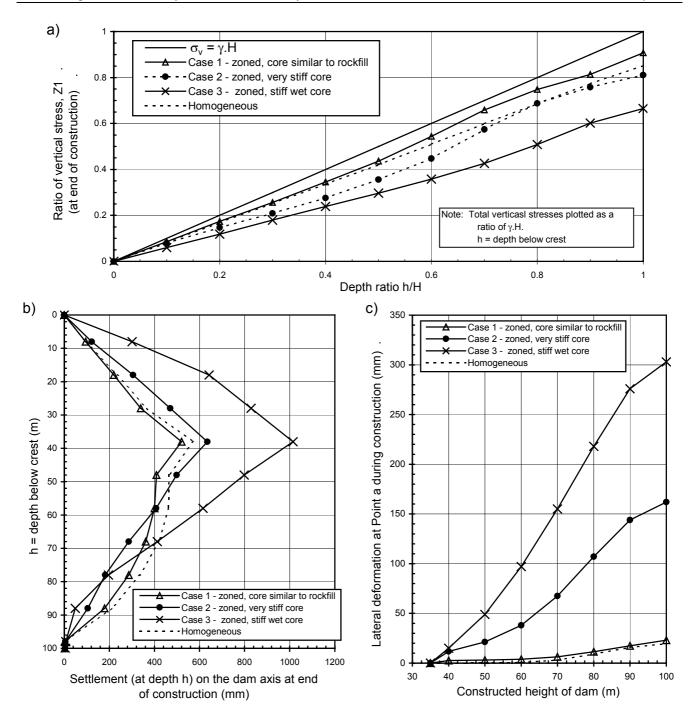


Figure 4.5: Finite difference modelling results; (a) total vertical stress on the dam centreline at end of construction, (b) settlement profile at dam axis at end of construction, and (c) lateral deformation at Point a (Figure 4.3) during construction.

The finite difference analysis was based on the assumption of plane strain conditions. This assumption is valid for embankments that are long compared to their height because strains in the direction parallel to the dam axis (i.e. y-direction in Figure 4.1) are negligible compared to those measured vertically and perpendicular to the dam axis. Consideration should be given to the potential for cross-valley arching and its effect on the stresses within the embankment for embankments constructed in steep valleys with narrow river sections.

4.1.1.3 Arching or Stress Transfer in Zoned Embankments

Arching across the core of zoned embankments, including puddle core earthfill embankments, has important implications on the deformation behaviour of the core during construction as well as the potential for hydraulic fracture post construction due to the reduced vertical stresses in the core. Bishop (1952), for undrained conditions in puddled core dams, and Nonveiller and Anagnosti (1961), for fully drained conditions in zoned

embankments, have developed methods for assessment of arching for embankments with narrow cores. The progression to finite element analysis and improvements in constitutive models to more accurately model the stress-strain relationship of materials allows for realistic assessment of arching in zoned embankments, as shown by the analysis of Beliche dam (Naylor et al 1997) (Figure 4.6a) and the modelling of puddle core earthfill dams (Figure 4.6b) by Dounias et al (1996).

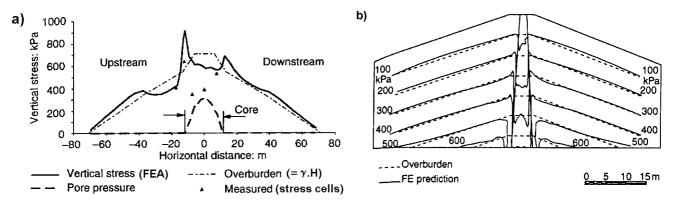


Figure 4.6: Finite element analysis of zoned embankments highlighting arching of core zone from (a) Beliche dam at about mid-height, elevation 22.5 m (Naylor et al 1997), and (b) a puddle core embankment (Dounias et al 1996).

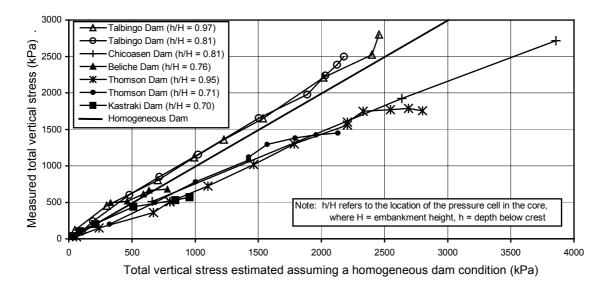
As the published analyses and those from Section 4.1.1.2 show, arching is developed in zoned embankments with narrow cores not only due to the core being more compressible than the supporting shoulders but also due to lateral spreading of the core. Bishop and Vaughan (1962) comment that for the Selset puddle core earthfill embankment the vertical deformations of the core were largely a result of plastic deformations due to lateral spreading and not consolidation type settlements due to pore pressure dissipation.

Factors contributing to the potential for arching are the difference in strength and compressibility properties between the core, filter/transition and outer earth or rockfill zones, the stress path and the embankment geometry, particularly the width of the core.

Some examples of vertical pressures measured during construction in the core of zoned embankments using pressure cells are shown in Figure 4.7, and are compared to the vertical stress estimated for a homogeneous embankment allowing for the embankment shape. The core width to depth ratio is 0.4 to 0.5 for most of the cases (i.e. thin cores) and is 0.7 for Talbingo dam. The near symmetrical thin cores comprised sandy clays to clayey sands and clayey gravels placed close to or on the wet side of Standard optimum moisture content and were supported by reasonably to well compacted filters, and gravel or rockfill shoulders.

For Kastraki, Thomson, Beliche and Chicoasen dams the measured vertical stresses are less than the homogeneous case. All four cases are considered to represent arching across the core. At the higher stress levels associated with the latter stages of construction the measured stresses tend toward the horizontal (i.e. minimal measured increase in total stress with increasing embankment height) at Kastraki, Thomson and Beliche dams. At Chicoasen dam (Alberro and Moreno 1982) the pressure cell measurements do not show the curvature in the latter stages of construction as do the other three cases.

For Talbingo dam the pressure cells in the mid region of the slightly upstream oriented core show a linear stress response with the stresses plotting close to the homogeneous case indicating arching is not a factor. However, the pore pressure response in the core and settlement during construction (refer Appendix B, Section 1.14) indicate that arching occurred in the vicinity of the near vertical downstream core / filter interface but not in the central region or the upstream region next to the core / filter interface sloped at 0.9H to 1V.



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Figure 4.7: Estimated versus measured (from pressure cells) total vertical stresses during construction.

Further numerical analysis was undertaken to analyse the effect of core width. The same procedure and embankment geometry was used as described in Section 4.1.1.2 analysed with symmetrical core slopes of 0.2H, 0.3H, 0.5H and 0.75H to 1V (Cases 4 to 7 respectively). Properties for the core are given in Table 4.2 and those for the rockfill were as per the earlier analysis (Figure 4.3). For these cases the core was modelled with a Young's modulus of 40 MPa, half that of the rockfill. The results are presented in Figure 4.8.

The total vertical stresses on the embankment centreline at Point b (Figure 4.3, h/H = 0.70) for each stage of construction are presented in Figure 4.8c, and include those from Cases 2 and 3. The total vertical stress for the zoned cases initially approximates that for the homogeneous case at low stress levels and then curves away at higher stress levels, similar to that observed for the thin core case studies (Figure 4.7) and is indicative of arching of the core.

The results of the analysis also show that at end of construction the total vertical stresses on the embankment axis increase with increasing core width, and for this example, approached stresses close to the homogeneous case for core slopes greater than about 0.5H to 1V (i.e. thick cores). However, there is still a significant differential in stress at the interface between the core and rockfill as can be seen in Figure 4.8b.

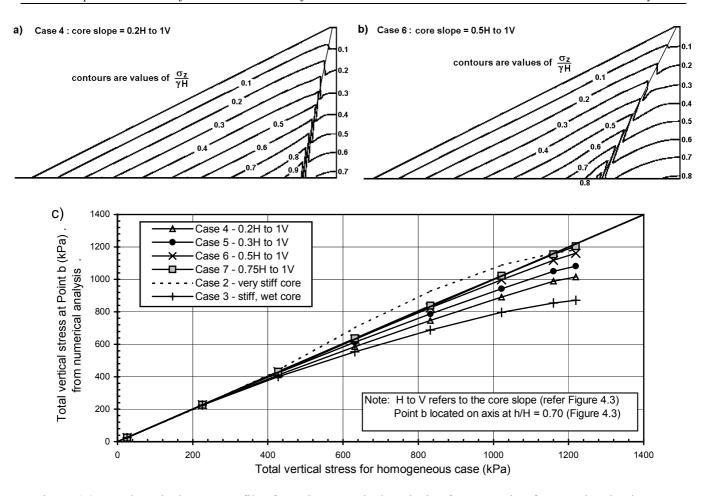


Figure 4.8: Total vertical stress profiles from the numerical analysis of construction for zoned embankments with different core slopes; (a) 0.2H to 1V case, (b) 0.5H to 1V case, and (c) total vertical stress at h/H = 0.70 with increasing embankment height.

4.1.1.4 Pore Water Pressures Developed During Construction

So far the discussion on stresses during construction has been on total stresses. However, for a more complete assessment of the deformation behaviour it is important to consider effective stresses and the effective stress paths during construction. For permeable embankment materials such as clean or free draining rockfills, gravels and sands the assumption of drained conditions during construction is valid. Therefore effective stresses and the effective stress state in these materials are readily assessable from the total stress conditions and consideration of partial impoundment, if this occurs during construction.

For low permeability embankment materials effective stresses are more difficult to predict prior to construction, but their assessment can be critical in terms of embankment stability. Therefore, instrumentation is often installed and monitored during construction in the form of piezometers, pressure cells, and internal and external deformation monitoring gauges. However, due to the unreliability of pressure cells, total stresses are generally assessed from finite element analysis, which can also be unreliable.

At placement, earthfill materials are partially saturated, with the degree of saturation dependent on the moisture content, material type, and the method and degree of compaction of the earthfill. The pore water pressures that are developed in the earthfill during construction, assuming it is well-compacted, are dependent on a number of factors including the initial degree of saturation, material compressibility properties, permeability, time of construction and applied stress levels. It is convenient to consider the positive pore water pressures in terms of r_u , the pore water pressure coefficient, as defined in Equation 4.2.

$$r_u = u/\sigma_z \text{ or } \Delta r_u = \Delta u/\Delta \sigma_z$$
 (4.2)

where r_u = pore water pressure coefficient, u = pore water pressure, σ_z = total vertical stress and Δ represents the incremental or tangential component of the parameter. The term r_u is used for the pore water pressure coefficient determined from case study data rather than \overline{B} , as defined in Equation 4.3, because the lateral stresses (σ_x and σ_y) parallel and perpendicular to the dam axis may not be equal.

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Three main types of pore water pressure response are generally observed (Figure 4.9):

- Negligible or limited positive pore water pressure response (r_u generally less than 0.1 to 0.2). Gould (1953, 1954) observed and the case study data indicates that for earthfills placed drier than about 0.5% to 1% of Standard optimum limited positive pore water pressures are developed during construction for most soil types.
- Moderate to very high sustained positive pore water pressure response (Δr_u greater than 0.5) with negligible to very limited drainage. This type of response is generally observed for clayey sands, clayey gravels and dominantly clayey earthfill types placed wetter than about 0.2 to 0.5% dry of Standard optimum (i.e. close to or above Standard optimum). Often the positive pore water pressure response is preceded by a period of negative pore water pressure response at low stress levels (Figure 4.9).
- Drainage during construction is significant. This is generally observed for earthfills of silty sands to silty gravels placed close to or wet of Standard optimum (i.e. wetter than about 0.2 to 0.5% dry of Standard optimum), but in some cases it is also observed for clayey sands and clayey gravels with low plasticity and possibly low content fines (less than about 15 to 25% finer than 75 micron). The typical pore water pressure response when placed wet of optimum is for an initially high pore pressure coefficient, which then decreases with increasing embankment height above the piezometer. Partial drainage during shutdown is also evident.

There are, however, case studies that fall outside or between these general types of pore water pressure response.

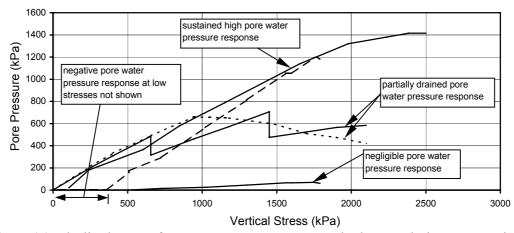


Figure 4.9: Idealised types of pore water pressure response in the core during construction.

Hilf (1948) provides a method for prediction of the pore water pressure response for partially saturated soils derived from one-dimensional consolidation theory and applied to field prediction using oedometer tests on laboratory samples prepared at proposed field placement specifications. The method assumes undrained conditions, and field compression is one-dimensional (i.e. zero lateral strain) and due to compression of the air voids. The method gives predictions indicating that the rate of pore pressure increase becomes very rapid as the air voids volume approaches zero and a saturated condition is reached. Once saturated the pore pressure increase is assumed to be equal to the increase in total vertical stress (i.e. no change in effective stress). Fredlund and Rahardjo (1993) comment that whilst the Hilf method provides a reasonable estimate of pore

water pressure, it will over estimate it because the derivation assumes matric suction equals

water pressure, it will over-estimate it because the derivation assumes matric suction equals zero (i.e. the pore water pressure equals the pore air pressure).

Khalili (2002) comments that for partially saturated soils of very low permeability (i.e. where undrained conditions may be a reasonable assumption) the rate of pore pressure increase in confined compression will always be less than the rate of vertical stress increase due to the presence of a small volume of air voids that remain under the magnitude of effective stresses likely in embankment dams. The field data for clayey earthfills of very low permeability placed close to or above Standard Optimum moisture content tend to support this (Figure 4.10), with maximum rates of pore water pressure increase equivalent to 60 to 90% of the change in total vertical stress (i.e. $\Delta r_u = 0.6$ to 0.9).

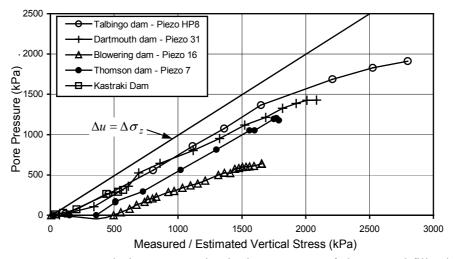


Figure 4.10: Pore pressure response during construction in the core zone of clayey earthfills placed at close to or wet of Standard Optimum moisture content.

It is noticeable from Figure 4.10 that the pore water pressure coefficient, Δr_w may decrease toward the end of construction. The reason for this is possibly partly due to pore water pressure dissipation. A more likely explanation is that for piezometers in the mid to lower region of the core the total stress ratio $(\Delta \sigma_3 / \Delta \sigma_1)$ in Skempton's equation for pore water pressure (Equation 4.3) is not constant but decreases in the latter stages of construction where the layer width is small compared to the height above the gauge (Figure 4.1b). Assuming the coefficients A and B are constant, the effect of a reduction in total stress ratio would be for a reduction in the ratio $\Delta u / \Delta \sigma_1$.

In contrast, the pore water pressure observations in the narrow medium to high plasticity wet placed clay core at Yonki dam indicated that the pore water pressure actually increased during non-construction periods (Fell et al 1992). This behaviour is possibly due to the yielding as a result of the strain rate dependency of the limit state surface. Whilst not common, an increasing pore water pressure response during non-construction periods has been observed in the soft foundations of fills constructed on soft ground, several examples of which are referred to by Hunter et al (2000).

For wet placed earthfills of permeability and construction staging such that partial drainage during construction occurs, the pore water pressure response displays an interesting feature that is replicated in laboratory testing. Triaxial isotropic compression tests on wet placed silty sands and silty gravels (Bishop 1957; Bernell 1982) undertaken in stages of undrained loading with a period of drainage between loading stages (Figure 4.11) show a reduction in Skempton's pore pressure parameter, \overline{B} (Equation 4.3), during subsequent stages of undrained loading.

A similar pore pressure response is observed in wet placed dominantly sandy and gravelly soils (silty sands and gravels, and some clayey sands and gravels) where partial dissipation of pore water pressures occurred in shutdown periods during construction (Bishop 1957, Eisenstein and Law 1977, Nakagawa et al 1985). Bishop (1957) considers the response is due to a decrease in the compressibility of the earthfill under increased effective stress conditions that result from partial drainage.

A decrease in the degree of saturation during dissipation of pore water pressures at shutdown periods could also influence the pore water pressure response afterward. The relatively high degree of saturation at placement is likely to result in the migration of water rather than air from the soil structure during the drainage stage, and as pressures dissipate previously dissolved air in water is likely to return to the air phase. The net effect could result in an overall decrease in the degree of saturation below that at initial placement.

$$\frac{\Delta u}{\Delta \sigma_1} = \overline{B} = B \left[1 - (1 - A) \left(1 - \frac{\Delta \sigma_3}{\Delta \sigma_1} \right) \right] \tag{4.3}$$

where u = pore water pressure, $\sigma_l = \text{major principal stress}$, $\sigma_3 = \text{minor principal stress}$, Δ represents incremental or tangential component of the parameter and \overline{B} , B and A are pore water pressure parameters defined by Skempton (1954).

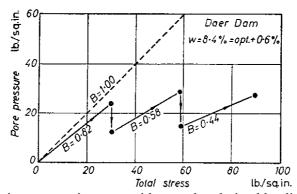


Figure 4.11: Triaxial isotropic compression tests with staged undrained loading and partial drainage on a partially saturated silty sand (Bishop 1957).

4.1.1.5 Summary of Stress Conditions During Construction

The total and effective stress conditions established in an embankment during construction are shown to be dependent on the embankment geometry, the embankment zoning geometry, and the strength and compressibility properties of the embankment materials.

For the simplest case of a homogeneous embankment on a rigid foundation, elastic solutions show that under the embankment centreline the vertical stress profile is dependent on the embankment geometry and the lateral stresses are dependent on the Poisson's ratio of the earthfill and embankment geometry. During the initial stages of construction when the embankment width is large in comparison to its height the assumption that the increase in stress equals the depth of soil times its weight is reasonable in the central region of the embankment. However, in the latter stages of construction this assumption is no longer valid due to the decreasing layer width in relation to the embankment height.

The homogeneous model is also a reasonable assumption for the stress conditions under the embankment axis for:

• Zoned embankments with broad central earthfill zones where the earthfill has been well compacted and placed drier than about 0.5% of Standard Optimum moisture content. Under these placement conditions the core has a high undrained strength and the stress conditions in the core are likely to be maintained in the

elastic range. Analysis and the case study data suggests central core widths with a combined core slope (i.e. upstream and downstream core slope combined) greater than about 1.5 to 2H to 1V are sufficiently broad, but it may also be a reasonable simplification for combined central core widths down to 1H to 1V.

and.

• Zoned embankments where the compressibility properties of the central earthfill zone and gravelly or rockfill shoulders are similar; i.e., compacted sandy and gravelly soils with non-plastic fines or low (less than about 20% finer than 75 micron) plastic fines contents, with shoulders of compacted gravels or rockfill. This is on the proviso that pore pressures generated in the core during construction are small and potential plastic type core deformations due to lateral spreading of the core are therefore negligible.

For zoned embankments the stress conditions during construction become more complex and are affected by the differential strength and compressibility properties of the embankment materials and the embankment zoning geometry. Total vertical stresses are affected by stress transfer or arching not only from differential compressibility but also from differential lateral stresses and the resulting lateral deformations to reach equilibrium stress conditions. Lateral stresses are affected by differences in Young's modulus and Poisson's ratio between the core and shoulders.

Finite element analysis is required for analysis of stresses and deformations in zoned earthfill embankments due to the complexity of the problem. Whilst finite element analysis is a standard tool for assessment of stresses and deformations during construction of embankment dams there is often a trade off between accuracy and simplicity (Duncan 1996) due to the complexity of the constitutive model to accurately model the strength and compressibility properties of partially saturated earthfills and the free draining gravel and rockfill zones.

For assessment of the effective stress conditions it is necessary to consider the pore water pressure response in the partially saturated embankment materials that are not free draining. Advances in the theory of partially saturated soils have developed constitutive models and methods that allow better estimation of pore water pressures than the simplistic Hilf (1948) method, but they have not been studied in detail here. Partial drainage during construction should also be considered.

4.1.2 Lateral Deformation of the Core During Construction

Table 4.3 summarises the lateral deformation of the core for 12 central core earth and rockfill embankments. In all cases the central, well-compacted core is of thin to medium width and is supported by filters/transition zones and shoulders of rockfill and/or gravels. Further information on the material and geometrical properties of the core, filter / transition zones and outer shoulders is given in Table 4.4 and the tables in Appendix A. No data is given for Beliche dam but Naylor et al (1997) comment that the deformations during construction distorted the inclinometer tubes such that lateral deformation measurements could not be taken, indicating the lateral deformation of the core was likely to have been quite large.

Lateral deformations were, in most cases, measured by internal horizontal movement gauge (IHM) installed in the filter and rockfill zones downstream of the core, and in some cases upstream of the core. The displacements quoted in Table 4.3 are from the IHM gauge closest to the core, usually from within the downstream filter or transition zone. The "lateral displacement ratio" is defined as the lateral displacement divided by the core width. Where only deformations on the downstream side of the core were available the "lateral displacement ratio" of the core has been estimated by dividing the displacement by the half width of the core and it is assumed that this displacement ratio is representative of the core at this location.

Figure 4.12 presents the estimated lateral displacement ratio versus fill height above the measuring gauge at the end of embankment construction. Figure 4.13 presents the estimated lateral displacement ratio versus fill height above the measuring gauge during embankment construction. It would be preferable to present these plots of the lateral displacement ratio versus vertical stress, but for several of the cases no pressure cell records were available and the likelihood of arching meant that vertical stresses could not be accurately estimated.

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The data shows that the lateral displacement ratios during construction are significant for zoned embankments with thin to medium width cores. Estimated lateral displacement ratios range from less than 0.5% up to more than 3% from the case study data.

The earlier finite difference analyses of zoned embankments on a rigid foundation (Section 4.1.1.2, Figure 4.4 and Figure 4.5) showed that the lateral deformation of the core was dependent on the development of differential lateral stress conditions between the core and supporting shoulder zones. The greater the stress difference the greater the lateral deformation of the shoulders to equilibrate the greater lateral stresses in the core. The rigid foundation also influenced the lateral stress and displacement profiles, the maximum lateral displacement being observed at approximately one third of the dam height above foundation level.

Table 4.3: Lateral deformations of the central core at end of construction for central core earth and rockfill embankments.

		Core Summary		Displacement Details				
Dam Name Height (m)		(refer Table 4.4 for mor	re details)	Displacement Details				
		Material Type / Placement	Width	Depth*1	h/H	Displacement	LDR * ³	
			(H to V)	(m)		(mm) * ²	(%)	
Copeton	113	SC – medium plasticity fines,	0.8H to 1V	40	0.35	140	-0.77	
		spec. = 1% dry to 1% wet of		59	0.52	150	-0.58	
		OMC (likely dry of OMC)		89	0.79	285	-0.76	
Wyangala	86	SC/SM – 1% dry of OMC	0.8H to 1V	51	0.59	67	-0.24	
				66	0.77	158	-0.70	
Fukada	56	GC – no details on moisture	0.6H to 1V	18.5	0.33	-6	0	
		spec., suspect dry of OMC.		36.5	0.65	42	-0.31	
Dartmouth	180	SC/SM – medium plasticity	0.65H to 1V	64	0.36	385 (dn), -450 (up)	-1.71	
		fines, spec. 0.5% dry to 2%	(average)	124	0.69	1220 (dn)	-2.69	
		wet.						
Beliche	55	GC – low plasticity fines, spec. ± OMC	0.44H to 1V	-	-	-	large	
La Grande, LG4	125	SM - non-plastic fines, spec. 1% dry to 2% wet	0.5H to 1V	70	0.56	300 (dn), -200 (up)	-1.3	
Nurek	289	GC/GM – no details on moisture spec., suspect at or above OMC given high PWP.	0.46H to 1V	145	0.50	425	-0.7 to -1.2	
Thomson	166	SC – medium plasticity fines,	0.5H to 1V	88	0.53	770 (dn), -750 (up)	-3.3	
		2% wet of OMC		118	0.71	445	-1.45	
Gepatsch	153	GM/GC - 0.5 to 2% wet of	0.25H to 1V	82	0.54	300 to 400 (dn - up)	-1.25 to -1.7	
_		OMC		122	0.80	750 (dn - up)	-2.2	
Talbingo	162	CL – medium plasticity, at OMC	0.7H to 1V	95	0.59	410 (dn), -1310 (up)	-2.80	
Blowering	112	CL/SC – medium plasticity fines, 0.3% dry to 0.3% wet	0.9H to 1V	73	0.65	735	-2.26	
El Infiernillo	148	CL/CH - medium to high plasticity, 3.7% wet of OMC	0.18H to 1V	74	0.59	-	-1.60	

^{*1} Depth = depth below crest.

^{*2} Lateral displacements mainly from IHM gauges and from the gauge closest to the core (usually within the (downstream filter/transition zone). Downstream (dn) is the default. Denoted (up) if from upstream of the core.

^{*&}lt;sup>3</sup> LDR = lateral displacement ratio and is calculated from the measured lateral core displacement divided by the core width. Values are negative in accordance with the convention that compressive strains are positive.

In the analyses the compressibility properties of the supporting rockfill zone were the same for each case. But, the compressibility properties of the supporting rockfill zone will also influence the amount of lateral deformation. Under similar differential stress conditions greater deformations will occur for more compressible supporting shoulders. The case study data tends to support this as discussed below.

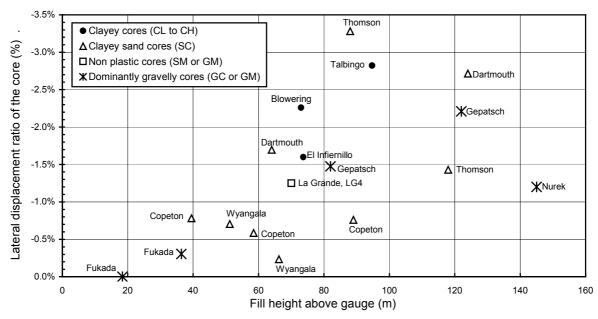


Figure 4.12: Estimated lateral displacement ratio of the core at end of construction.

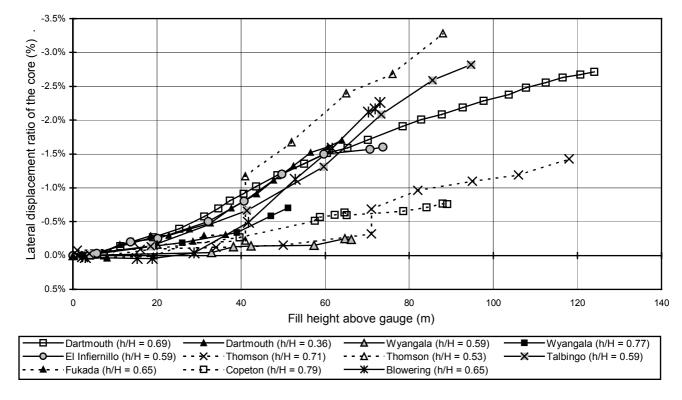


Figure 4.13: Estimated lateral displacement ratio of the core versus fill height above gauge during construction.

Table 4.4: Summary of embankment and earthfill properties for cases used in the analysis of lateral core deformation during construction.

	Height	Zoning			Co	re Details			Fil	lters / Transiti	ion	Should	lers	
Dam Name	(m)	Class ⁿ * ¹	Width * ² (H to V)	ASCS *3	Fines * ⁴ (%)	Plasticity *4	FMC to OMC *5	Pore *6 Pressure	Material	Width *7	Compaction Rating *8	Material	Compaction Rating *9	Comment
Fukada	56	5,1,2 c-tm	0.6 to 1	GC (?)	-	-	No details	-	-	thin	good (?)	weathered sandstone & granite "dirty"	good (?)	relatively high moduli for weathered rockfill
La Grande, LG4	125	4,1,0 c-tm	0.5 to 1	SM	30	non-plastic	1% dry to 2% wet, suspect avg. is wet.	negl to low	sandy gravels	mod to thin	good	gravels	good (inner)	outer rockfill shoulders (granite & gneiss), reasonable to good
Nurek	289	4,2,0 c-tn	0.46 to 1	GC/GM	-	-	No details	med to high	sands & gravels	mod to thin	good	gravels	good	Suspect core placed close to OMC given high PWP response.
Copeton	113	5,2,0 c-tm	0.8 to 1	SC	25 to 60	medium	Spec. 1% dry to 1% wet	negl to low	crushed rock	thin	good	granite (very high strength)	reasonable to good	Likely dry core placement given low PWP.
Wyangala	86	5,2,0 c-tm	0.8 to 1	SC-SM	-	-	1% dry	negligible	sands & crushed rock	mod to thin	good	porphyritic gneiss (high to very high strength)	reasonable to good (inner)	outer rockfill of reasonable to poor compaction.
Thomson	166	5,2,0 c-tm	0.5 to 1	SC	44	medium	2% wet	med to high	crushed sandstone	thin	reasonable	sandstone and siltstone	good (inner)	High to very high strength rock. Outer rockfill of lesser quality.
Gepatsch	153	5,2,0 c-tn	0.25 to 1	GM/GC	30	-	0.5 to 2% wet	low to high (rapid dissipation)	gravels	broad	reasonable to good	gneiss	reasonable	clean to "dirty" coarse rockfill.
Beliche	55	5,2,2 c-tn	0.44 to 1	GC	20 to 36	low	Spec. ± OMC	med to high	sandy gravels	thin	reasonable (?)	rockfill (med to high strength)	poor to reasonable	inner rockfill high fines and low moduli
Dartmouth	180	, ,	0.6-0.7 to 1	SC/SM	24	medium	Spec. 0.5% dry to 2% wet	high to very high	crushed rock	thin	good	granitic gneiss	good (inner)	reasonable compaction of outer rockfill
Talbingo	162	5,2,0 c-tm (u)	0.7 to 1	CL	18 to 70	medium	OMC	very high	crushed rock	broad	good	rhyolite & porphry	good (inner)	reasonable compaction of outer rockfill
Blowering	112	5,2,0 c-tm	0.9 to 1	CL/SC	30 to 80		0.3% dry to 0.3% wet	med to high	sandy gravels	thin	good	quartzite & phyllite	good (inner), reasonable (outer)	relatively low moduli of rockfill
El Infiernillo	148	5,2,0 c-tn	0.18 to 1	CL/CH	54 to 70	med to high	3.7% wet	suspect very high	sands & gravels	moderate	good	diorite & silicified conglomerate	reasonable (inner) to poor (outer)	very narrow core.

Notes: *1 Zoning classification system defined in Section 1.2. Based on the system developed by Foster (1999).

*2 core width refers to the width of the core compared to the height (H = horizontal, V = vertical)

*3 ASCS = soil classification to Australian Soil Classification System (Australian Standard AS 1726 – 1993)

*4 Fines = % finer than 75 micron, Plasticity: low = low plasticity, med = medium plasticity, high = high plasticity

^{*5} FMC to OMC = field or placement moisture content relative to Standard Optimum moisture content

^{*6} Pore pressures during construction; negl = negligible $(r_u < 0.1)$, low = r_u of 0.1 to 0.25, med = medium $(r_u = 0.25 \text{ to } 0.5)$, high = r_u of 0.5 to 0.75, very high = r_u of > 0.75.

^{*7} Qualitative assessment of width of filters / transition zone; thin = 3 to 6 m width, broad = more than about 20 m but dependent on dam height

^{*8} Compaction rating for filters / transition based on placement methods, layer thickness and relative density.

^{*9} Compaction rating for rockfill summarised in Section 1.2.

Several trends are evident from the lateral displacement case study data:

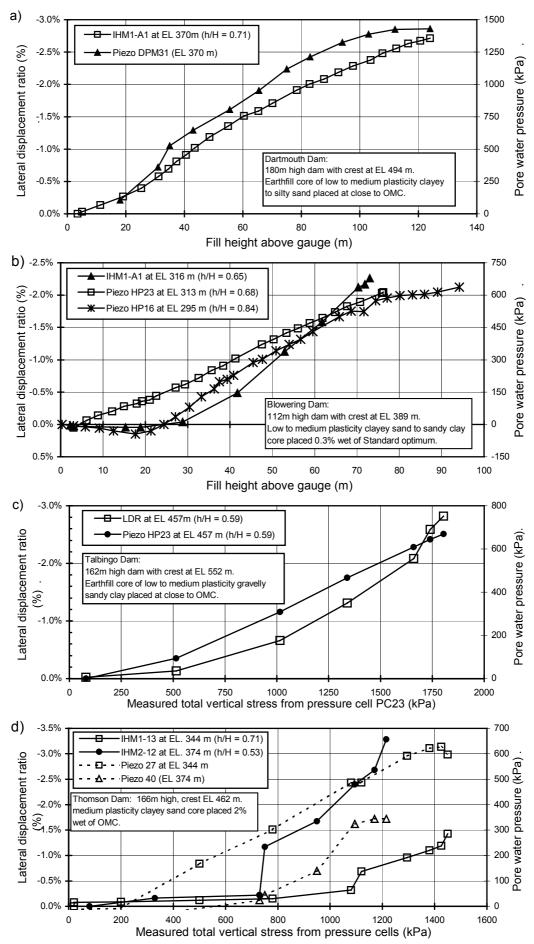
- Placement moisture content of the core relative to Standard optimum moisture content is significant. Low lateral displacement ratios (< 1% at fill heights of 35 to 90 m) are observed for earthfills placed on the dry side of Standard optimum (Wyangala, Copeton and probably Fukada). Higher lateral displacement ratios (1.0 to 2.2% at 60 m fill height) are generally observed for earthfills placed at or on the wet side of Standard optimum.
- Low lateral displacement ratios are observed for embankments with broad well-compacted gravelly zones of likely very high moduli adjacent to the central core (Nurek, La Grande LG4 and Gepatsch dams). For these cases the cores are silty to clayey sands and gravels placed at or wet of Standard optimum.
- Large lateral displacement ratios are observed for clayey earthfills placed at or wet of Standard optimum supported by shoulders of relatively highly compressible rockfills (Beliche and Blowering dams). At Beliche dam, Naylor et al (1997) describe the inner rockfill zone of weathered, medium strength schists and greywackes as "relatively lightly compacted" and of "relatively high compressibility". At Blowering dam the rockfill, although wetted and reasonably (outer Zone 3B) to well compacted (inner Zone 3A), the vertical secant moduli of the rockfill was calculated from hydrostatic gauges to be relatively low (20 to 40 MPa for Zone 3A placed in 0.9 m lifts and 10 to 20 MPa for the outer Zone 3B). Further details on the material properties at Blowering dam are given in Appendix C.
- Arching effects in the very thin wet placed clay core at El Infiernillo dam are possibly significant in the relatively low lateral strain measured.
- At Talbingo dam the lateral deformation was significantly greater on the upstream side of the core (1310 mm) compared to the downstream side (410 mm) at elevation 457 m (h/H = 0.59). Possible reasons for this are the slight upstream orientation of the core and the much broader width of the filter and transition zones (of well-compacted finer sized rockfill) on the downstream side.

For a number of case studies the lateral displacement ratio versus fill height plot (Figure 4.13) shows a marked increase in the rate of lateral displacement ratio at fill heights in the range 20 to 50 m above the gauge. The cases that display this behaviour are typically clayey earthfills placed at or wet of Standard optimum. It is possible that this response may be due to yielding or may reflect an increase in the undrained Poisson's ratio as the initially over-consolidated earthfill approaches or becomes normally consolidated.

For several case studies the lateral displacement ratio of the core was compared to the pore water pressure response at similar elevations to ascertain if the incremental increase in lateral displacement ratio observed correlated to a change in the pore water pressure response (Figure 4.14). The results indicate:

- At Thomson dam (h/H = 0.53) and Talbingo dam (h/H = 0.59) the data indicates that the increase in incremental lateral displacement ratio occurs at about the same level of total vertical stress level as an incremental increase in the pore water pressure response.
- At Dartmouth dam (Figure 4.14a) there is some indication of an incremental increase in the lateral displacement ratio and the pore water pressure response at 20 to 25 m fill height, although it is not clearly evident for the lateral displacement ratio.
- At Blowering dam the pore water pressure response at Piezometer 16, at a lower elevation than the deformation gauge, shows a similar response at 22 m fill height, but the piezometer at the same elevation (Piezometer 23) shows no incremental increase in the pore water pressure response.

Overall, there is some data, although it is not conclusive, that the observed incremental increase in lateral displacement ratio for several of the wet placed clayey earthfill case studies occurs concurrently with an incremental increase in the pore water pressure response.



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Figure 4.14: Lateral displacement ratio and pore water pressure versus fill height or measured total vertical stress at (a) Dartmouth dam; (b) Blowering dam; (c) Talbingo dam and (d) Thomson dam.

Several case studies show a flattening in the curve of lateral displacement ratio versus fill height near the end of construction (Figure 4.13). This is potentially due to arching (El Infiernillo) and the effect of embankment shape on the distribution of vertical and lateral stresses, which is not properly allowed for by considering depth of fill as the variable.

4.1.3 Vertical Deformation of the Core During Construction

Internal settlements during construction were collected for 57 embankments of earthfill, zoned earthfill and zoned earth and rockfill types. The settlement records were mostly from internal vertical settlement gauges (IVMs) installed in the central core of the earthfill (near the dam axis) as construction proceeded.

The purpose of the analysis is to define "normal" deformation behaviour so case studies showing "abnormal" deformation can be identified and then further analysed. The data is grouped into those case studies where a "homogeneous" type analysis (i.e. small lateral core strain) is appropriate, and those where lateral deformations are likely to be significant.

The vertical deformation in the core under the embankment axis (or within close proximity to the axis) is presented in the following formats:

- Vertical strain versus effective vertical stress (and secant modulus versus effective vertical stress) for those case studies where a "homogeneous" type analysis is appropriate.
- Vertical strain versus fill height above the cross-arm for case studies where lateral deformations are likely to be significant.
- Total vertical settlement (in millimetres) of the core during the period of construction versus embankment height. The total vertical settlements are estimated from the cumulative settlement between the cross-arm gauges.

Vertical strains in the core are calculated between the individual cross-arm intervals of the IVM by dividing the measured settlement (between the cross-arm interval) by the original distance between the cross-arms. The vertical stresses or fill heights are representative of the midpoint between the nominated cross-arm intervals. Corrections have been made to the strain readings so that the zero strain reading is equivalent to zero stress (or zero fill height). The corrections are based on the strain measurements at low stress levels.

The vertical strain (and secant modulus) plots are presented in two forms; (i) vertical strain at the end of construction, and (ii) vertical strain during construction. For the plots at the end of construction the vertical strains (or secant moduli) are calculated and plotted at cross-arm intervals over the full depth of the IVM gauge from near to crest level to foundation level. These plots, which include all the case studies, are referred to as the "vertical strain profile at end of construction" and they broadly define the range of the data. The deformations in the upper 10 to 20% of the embankment have generally been excluded because of narrowing of the embankment and reduction in the lateral confinement, which often gives relatively large vertical strains resulting in low estimates of apparent moduli.

The vertical strain (and secant modulus) plots during construction only present data from selected cross-arm intervals from selected representative case studies. For these cases the cross-arm interval and its h/H ratio are identified in the legend. The purpose of these plots is to highlight the change in vertical strain (or secant modulus) with increasing vertical stress (or fill height), which is more clearly evident in these plot types.

4.1.3.1 Vertical Deformation for Dry Placed Earthfills and Other Cases with Small Lateral Core Displacement

The types of case studies analysed in this section are those for which simplification to an essentially onedimensional analysis of the earthfill core is considered a suitable approximation of the deformation behaviour, and they are:

- Embankments with relatively broad core widths and dry placed earthfill cores, typically those placed at moistures contents drier than about 0.5 to 1% dry of Standard optimum. The limitation on core width is for case studies with thick cores (combined slope greater than 1H to 1V), but several case studies with broader medium width cores have been included.
- Embankments with cores of dominantly sandy to gravelly soil types with low plasticity silty fines or with low plasticity clayey fines of low fines content, and within which positive pore pressures during construction were relatively low. The limitation on pore water pressure development generally restricts these cases to medium to thick core widths that are generally placed on the dry side of Standard optimum moisture content.
- Zoned embankments with thin cores where the compressibility properties of the core are similar to the shoulders. Two cases have been included Naramata and Frauenau, both dominantly sandy and gravelly cores with silty fines placed on the dry side of optimum.

Figure 4.15 presents vertical strain versus vertical effective stress as the embankment is raised for selected cross-arm intervals of selected case studies. The data is from the lower third of the core (i.e. h/H > 0.66). Effective vertical stresses at the midpoint between the nominated cross-arms have been estimated from total stresses estimated as described in Section 4.1.1.1 with positive pore water pressures then deducted. Note that negative pore water pressures at low stresses have been ignored and therefore the actual effective stresses at these low stress levels will be higher than indicated in the Figure 4.15 and other figures in this section.

The plots (Figure 4.15) show an apparent pre-consolidation pressure beyond which the incremental vertical strain (in terms of the log of stress) is greater. From the intersection of the two parts of the curves, these apparent pre-consolidation pressures range from 700 to 1000 kPa for the silty gravel, silty sand and clayey sand (with low content fines) earthfills, to 200 to 400 kPa for the clayey earthfills. The influence of negative pore water pressures, which has been ignored, will steepen the stress-strain relationship at low stress levels in these partially saturated earthfills.

Figure 4.16 presents the secant modulus versus vertical effective stress as the embankment is raised for selected cross-arm intervals of selected case studies; the modulus calculated by dividing the estimated vertical effective stress by the vertical strain. The modulus has been termed a "confined secant modulus" based on the assumption of negligible lateral strain. Figure 4.16 shows a relatively broad variation in confined secant modulus at a given stress level, more so for the dominantly sandy and gravelly core types probably because of the effect of variation in fines content and fines plasticity as well as compaction conditions.

For most cases shown in Figure 4.16 the confined secant modulus generally increases with increasing effective vertical stress, but with a large variation in the incremental change in modulus over a given stress level. Other cases show the confined secant modulus to remain relatively steady with increasing stress whilst some even show it to decrease. A gradual increase in the confined secant modulus with increasing stress, for the stress range of the data, is considered the typical response indicating the earthfill becomes less compressible as the voids ratio decreases.

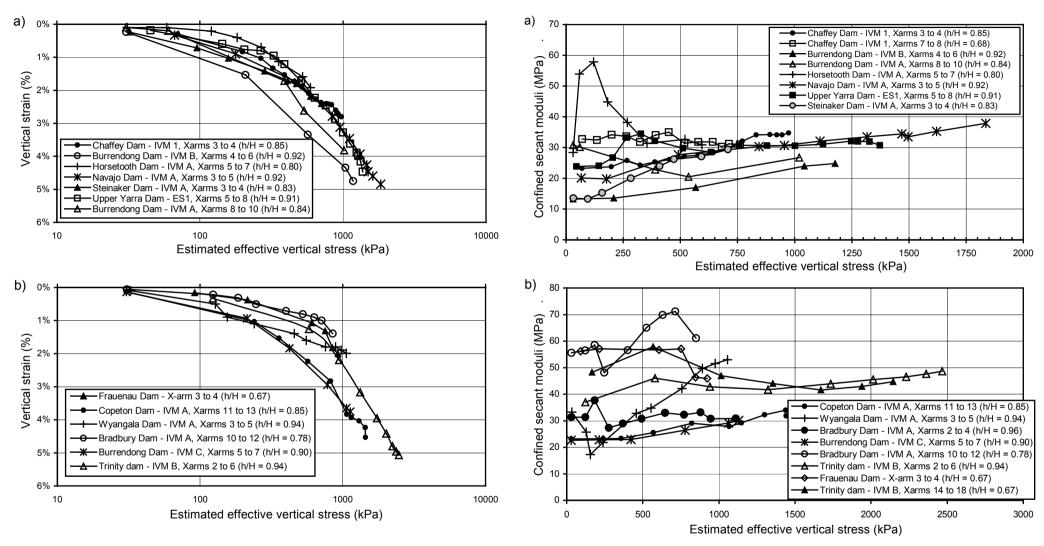


Figure 4.15: Vertical stress versus strain of the core during construction for selected cross-arm intervals of selected case studies; (a) dry placed clay cores, and (b) dry placed dominantly sandy and gravelly cores with plastic fines.

Figure 4.16: Confined secant moduli of the core versus vertical stress during construction for selected cross-arm intervals of selected case studies, (a) dry placed clay cores, and (b) dry placed dominantly sandy and gravelly cores with plastic fines.

Of the case studies that show an increase in the confined secant modulus with increasing stress the secant modulus at low stress levels is sometimes quite low; for example, the cross-arm intervals shown for Burrendong, Navajo and Steinaker dams. Evaluation of the stress-strain relationship for several of these cases (Figure 4.17) shows that the tangent confined moduli at low stresses is much lower than that at stress levels in excess of 200 to 300 kPa, and is near constant at the higher stress levels. Given that the earthfill core for these embankments has been placed at moisture contents on the dry side of Standard optimum and well-compacted in thin lifts this type of stress-strain behaviour is unlikely. It would be expected that the tangent confined moduli at low stresses would be at least similar to that observed at stresses up to 500 to 1000 kPa.

The most likely cause of this observation is suspected to be settlement on seating of the cross-arm after installation possibly due to a slightly uneven bedding surface or loose nature of the bedding material on which the cross-arm sits. Settlements of 20 to 30 mm due to seating can make a large difference in the calculated tangent and confined secant moduli at low stresses.

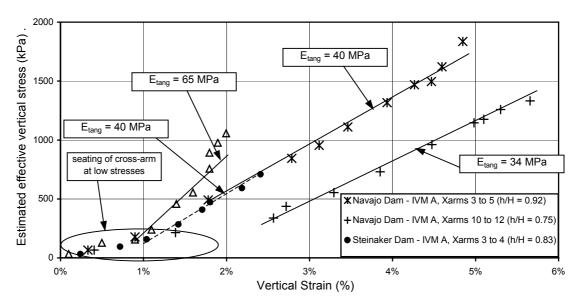


Figure 4.17: Seating settlements at low stresses suspected as cause of low moduli estimates at low stresses.

Table 4.5 presents a summary of the range and average of the confined secant moduli at various effective vertical stress levels from the case studies for well-compacted dry placed central core zones. It includes:

- Data from the case studies represented in Figure 4.15, including values from analyses of other cross-arm intervals in these embankments that are not shown.
- Data from case studies where the vertical strain was also analysed for specific cross-arm intervals but not shown in Figure 4.15.
- Data from the end of construction vertical strain profiles (Figure 4.18 and Figure 4.19).
- Data from Gould (1953, 1954) for a number of USBR dams of dry placed earthfills mainly from embankments with very broad earthfill zones. Gould presented his data in terms of vertical strain versus effective vertical stress. It has been converted here to values of secant moduli.

Seating effects have only been considered in the confined secant modulus values for those cases where the stress-strain relationship indicated this to have had a clearly identifiable influence on the deformation behaviour. For most of the cases represented in Table 4.5 the data was not available from which to make an assessment of seating. Therefore, the real lower limits and averages, particularly at the lower vertical stress levels, are possibly slightly higher than indicated. Categorisation based on material type is justified from the findings by Gould (1954) and from analysis of the case study data collected for this study.

	_						
		Confined Secant Modulus (MPa)*1					
Cara Matarial Tyra	No.	Effective Vertical Stress Range (kPa)					
Core Material Type	Cases	500 to 700* ²	1000	1500 to 2000			
Clayey soil types (CL) – sandy clays and	20	15 to 45	15 to 50	25 to 42			
gravelly clays		(27)	(30)				
Silty soils (ML), data from Gould.	3	30 to 60	-	-			
Clayey Sands to Clayey Gravels (SC/GC),	11	20 to 65	30 to 65	25 to 65			
plastic fines, > 20% fines		(33)	(40)				
Silty Sands to Silty Gravels (SM/GM),	9	35 to 65	45 to 65	35 to 65			
plastic fines, > 20% fines		(46)	(50)				
Sandy and Gravelly soils - non-plastic	6	35 to 80	35 to 90	40 to 90			
fines or < 20% plastic fines		(60)	(60)				

Table 4.5: Confined secant moduli during construction for well-compacted, dry placed earthfills.

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Notes *1 Values represent range of confined secant moduli, values in brackets represent average

The range of confined secant moduli within any core material type is relatively broad, up to 0.5 to 2 times the mean. Part of this variation is likely explainable by variations in material properties (particle size distribution, plasticity, etc.), variations in compacted density ratio and inaccuracies in the IVM measurement readings. Correlation between the compacted density ratio at placement (and dry density) and confined secant moduli for a given earthfill type did not identify any clear trend, however, the database was limited.

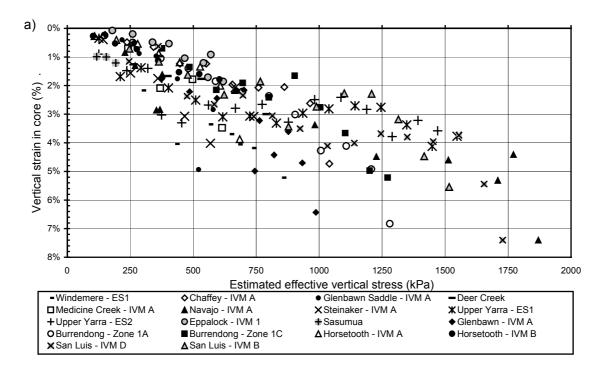
Figure 4.18 and Figure 4.19 present the vertical strain and confined secant moduli versus effective vertical stress at end of construction for all case studies of dry placed earthfills. As discussed earlier, they differ from Figure 4.15 and Figure 4.16, which are for selected cross-arm intervals of selected case studies during construction. Essentially there is no difference between the plots, but Figure 4.18 and Figure 4.19 present the data for all case studies representative of dry placed earthfills and show a broader range of variation of vertical strain and modulus.

For the dominantly sandy and gravelly earthfill types the data (Figure 4.18b and Figure 4.19b) show:

- Higher confined secant moduli for silty sand and silty gravel type earthfills with non-plastic fines, or with low plastic fines content (Bradbury, Naramata, Andong and Frauenau).
- On average, lower confined secant moduli in the clayey sand and clayey gravel type earthfills (Copeton, Granby, Deer Creek and Burrendong Zone 1B).
- Likely arching effects in the narrow, dry placed earthfill core at Naramata.

For the clay earthfills the plots (Figure 4.18a and Figure 4.19a) show a reasonable degree of consistency between case studies. Several of the points that show high strains at a given vertical stress are within wetter placed earthfills near to the interface with the foundation. In general, relatively high vertical strains are observed at this interface region, most probably due to plastic type deformations of the wet placed earthfill in the foundation contact zone.

^{*2} Includes Gould (1953, 1954) data.



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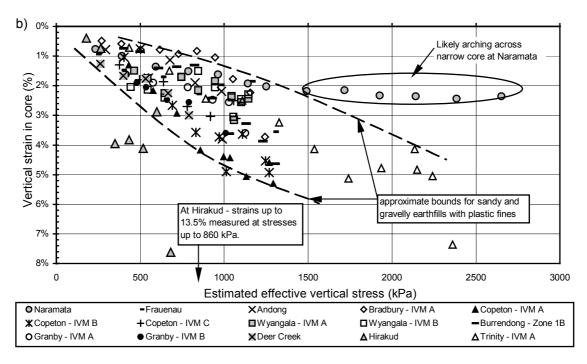
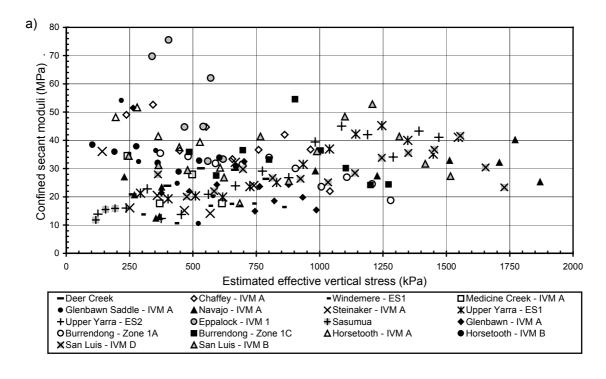


Figure 4.18: Vertical strain versus effective vertical stress in the core at end of construction for (a) dry placed clay earthfills and (b) dry placed dominantly sandy and gravelly earthfills.



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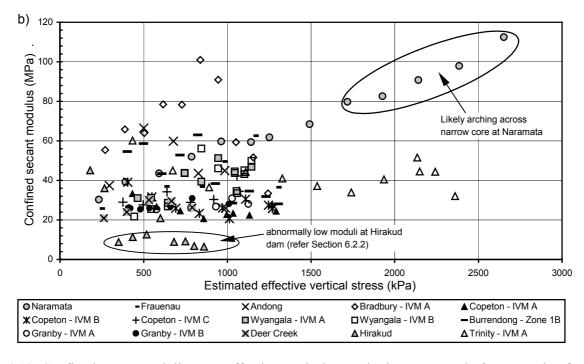


Figure 4.19: Confined secant moduli versus effective vertical stress in the core at end of construction for (a) dry placed clay earthfills and (b) dry placed dominantly sandy and gravelly earthfills.

4.1.3.2 Vertical Deformation for Wet Placed Earthfills and Other Cases with Large Lateral Core Displacement

The types of case studies analysed in this section are zoned embankments where the deformation of the earthfill core cannot be adequately modelled by simplification to a one-dimensional analysis. For these case studies the lateral displacement of the core during construction is generally large and has a significant influence on the vertical deformation behaviour of the core. Analysis of the deformation behaviour of the core should be undertaken on a case-by-case basis using finite element methods to model the embankment construction. Notwithstanding this, there is some merit in comparative analysis of the case studies as a general guide to possible identification of "abnormal" deformation behaviour.

The figures in this section are presented versus fill height. It would have been preferable to use total vertical stress, but the data includes a number of case studies where arching was significant and no information of total stresses from pressure cells was available. Rather than attempt to estimate the total vertical stress the fill height was used for all case studies. The case study data on the vertical strain in the core has been sorted based on core width and is presented in the following figures:

- Figure 4.20 incremental vertical strain versus fill height for selected cross-arm intervals from selected case studies.
- Figure 4.21 and Figure 4.22 vertical strain profile of the core (i.e. versus fill height) at end of construction. Figure 4.21 is of dominantly sandy and gravelly earthfill cores and Figure 4.22 is of clay cores.

The difference between the plot types, as previously discussed, is that the incremental vertical strain plots highlight the change in vertical strain with increasing fill height, and the vertical strain profiles at end of construction include all the case studies and highlight the degree of variation from the data.

From Figure 4.20, the general trend for thin cores is for an apparent decrease in the incremental vertical strain with increase in fill height. The medium width cores show a similar trend but it is not as evident as it is for the thin cores. Arching across the core is likely to be a significant factor influencing this behaviour, particularly for the thin cores. If measured total vertical stress were used as the x-axis this trend would not be evident for most of the case studies because the influence of arching would be included in the total vertical stress. The analysis of the core deformation during construction at Blowering dam shows this to be the case (refer Section 3 of Appendix C).

Figure 4.21 and Figure 4.22 highlight the very high vertical strains measured within some regions of the core of several zoned embankments including Blowering, Beliche, Tedorigawa and Chicoasen. High lateral strains and plastic deformation of the wet placed earthfill core are likely to be the reason for the large vertical strains observed in most, but not all cases. Further discussion on several of these cases is presented in Section 6.2.

In Figure 4.21 and Figure 4.22 the bounds shown on several of the figures are an interpretation, based on evaluation of the data, of the general limits of "normal" type deformation during construction in wet placed earthfills. However, assessment of whether or not a point or series of points located outside the bounds shown is potentially "abnormal" requires further evaluation, and this is the main purpose of the indicative bounds. In general, for those cases where a series of points plot below the lower bound (such as for Beliche dam in Figure 4.21a) or one or two points plot well below the lower bound it is likely that the deformation behaviour is an outlier to the general trend.

For several case studies the point or points that plot outside the bounds are not necessarily indicative of an outlier or potentially "abnormal" deformation behaviour. Several case studies show a large degree of variation

in vertical strain between consecutive cross-arm or depth intervals, showing an almost zigzag type pattern (e.g. IVM ES1 at Dartmouth in Figure 4.21b). This type of effect is, in some cases, due to the inaccuracies or errors of measurement in the IVM gauge and does not necessarily represent a large localised region of vertical strain. These points are not likely to be outliers.

Conversely, it should also be recognised that a case study that shows potentially "abnormal" type deformation behaviour in the core during construction may not necessarily be identified as such in plots such as Figure 4.21 and Figure 4.22. Although, of the case studies represented in the plots those within the bounds are all considered to be indicative of "normal" type deformation behaviour.

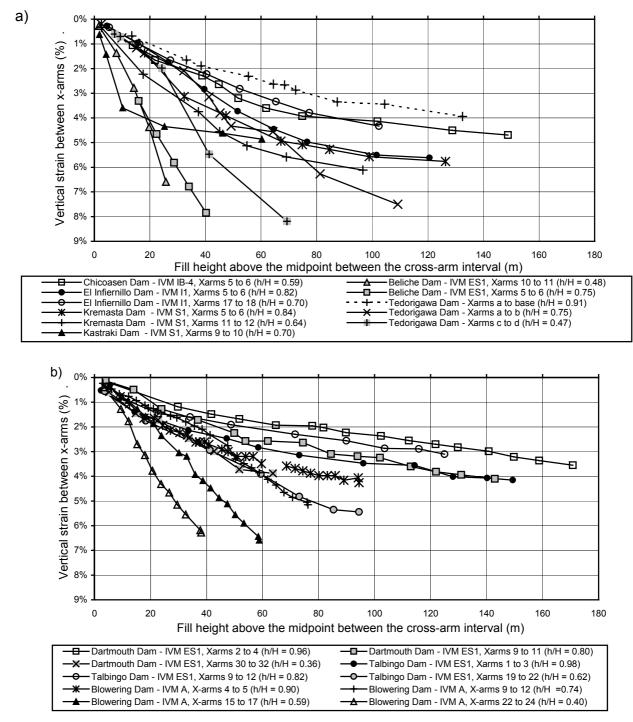
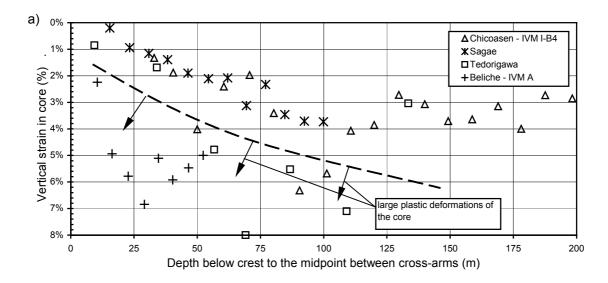


Figure 4.20: Vertical strain in the core versus fill height during construction at selected cross-arm intervals in selected case studies of earthfills placed close to or wet of Standard optimum, for (a) thin core (combined slopes less than 0.5H to 1V), and (b) medium cores (combined slope \geq 0.5H to 1V and \leq 1H to 1V).



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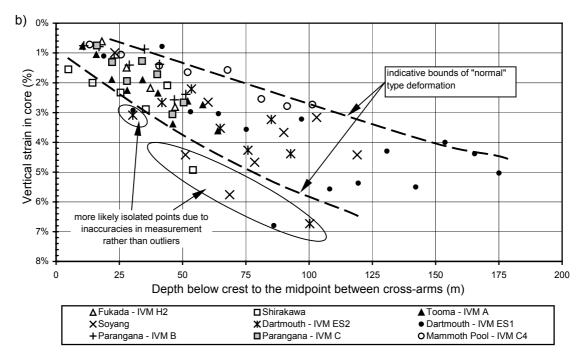


Figure 4.21: Vertical strain in the core versus fill height at end of construction for dominantly sandy and gravelly earthfills with plastic fines placed close to or wet of Standard optimum, for (a) thin cores (combined slopes less than 0.5H to 1V), and (b) medium to thick cores (combined slope \geq 0.5H to 1V).

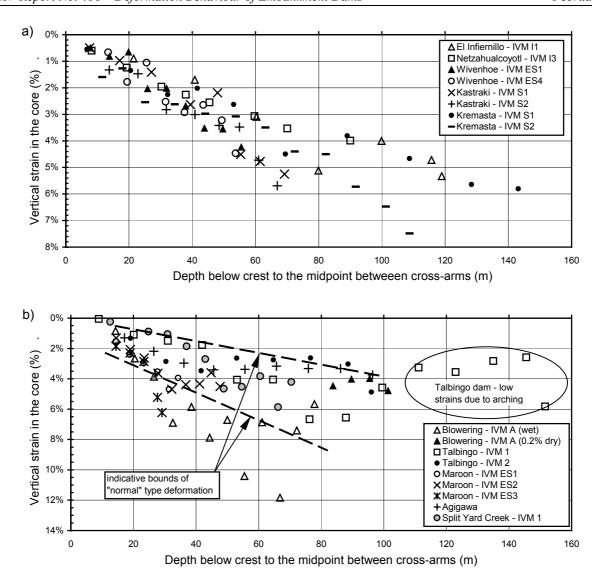


Figure 4.22: Vertical strain of the core versus fill height at end of construction for clay earthfills placed close to or wet of Standard optimum moisture content, for (a) thin core (combined slopes less than 0.5H to 1V), and (b) medium to thick cores (combined slope \geq 0.5H to 1V).

4.1.3.3 *Total Settlement of the Core During the Construction Period*

Another type of plot considered useful in the identification of potential "abnormal" deformation behaviour of the core during construction is the total settlement of the core that occurs during the period of embankment construction. Figure 4.23 presents the total settlement of the earthfill core or central earthfill zone during the period of construction versus embankment height for the case studies. The results show a general trend of increasing settlement (as a percentage of the embankment height) with increasing embankment height. At embankment heights of less than 50 m total settlements were generally in the order of less than 1% up to 2.5%, and for embankment heights greater than 150 m in the order of 2 to 5%.

Several sub-groups of material type and core width with a similar total settlement versus embankment height relationship are identifiable (Figure 4.23b), they are:

- Dominantly fine grained earthfills, mainly clays but includes some dominantly silty soils, covering thin to very broad cores and dry to wet placement conditions. These cases generally plot on the higher side with settlements in the order of 3 to 3.5% for embankment heights greater than about 100 m.
- Dominantly sandy and gravelly earthfills with plastic fines (i.e. clayey and silty sands and gravels), dry to wet placement conditions and medium width to very broad core sizes. The total settlement for this sub-

group is similar to that for the clay earthfills, reaching settlements in the order of 3 to 4% for embankment heights greater than about 100 m.

- Dominantly sandy and gravelly earthfills with non-plastic fines or with low content plastic fines (typically less than 10 to 20%) for thin to thick core widths. The total settlement for these core types is at the low end for embankment dams, approximately half that of the clayey earthfill types at a given embankment height. The low settlement reflects the low compressibility of these earthfills when well compacted. Settlements are typically in the range 1 to 2% for embankment heights up to 150 m.
- Dominantly sandy and gravelly earthfills with thin width cores. The total settlement for these core types is in between that of the clayey earthfills and non-plastic sandy and gravelly earthfills, reaching maximum settlements in the range 1.5 to 3% for embankment heights greater than about 100 m.

The data within each material / core width sub-group shows a reasonable fit as indicated by the high regression coefficients of the trendlines (Figure 4.23b). The trendlines are quadratic equations and are given in Table 4.6 along with the standard errors and regression coefficients. Outliers to the general trend (Beliche, Hirakud, Blowering, Tedorigawa and Nurek) were excluded from the correlations. These cases are discussed further in Section 6.2.

The settlement to embankment height curves from Figure 4.23b are useful as a guideline for estimation of the total core settlement during construction and identification of potentially "abnormal" deformation behaviour for embankment heights up to 100 to 125 m. Above about 125 m only one or two points define the shape of the relationships and therefore settlement estimates will be less reliable.

Correlations between the settlement as a percentage of the embankment height and embankment height in the form of a power function are summarised in Table 4.7. The regression coefficients indicate a good correlation exists for the sub groups and the range in standard error is relatively low, from 0.24 to 0.47%.

The correlation shown for several of the sub groups, in particular for the clayey earthfills, is coincidental. The clay earthfill sub-group includes dry to wet placement conditions and core widths from thin to very broad. It is evident from previously presented data that the deformation behaviour of a wet placed, thin to medium width clayey core is significantly different to that of a dry placed, very broad clayey earthfill core. Therefore, for these two different material placement conditions and core width geometries to be included in the same sub group is somewhat coincidental.

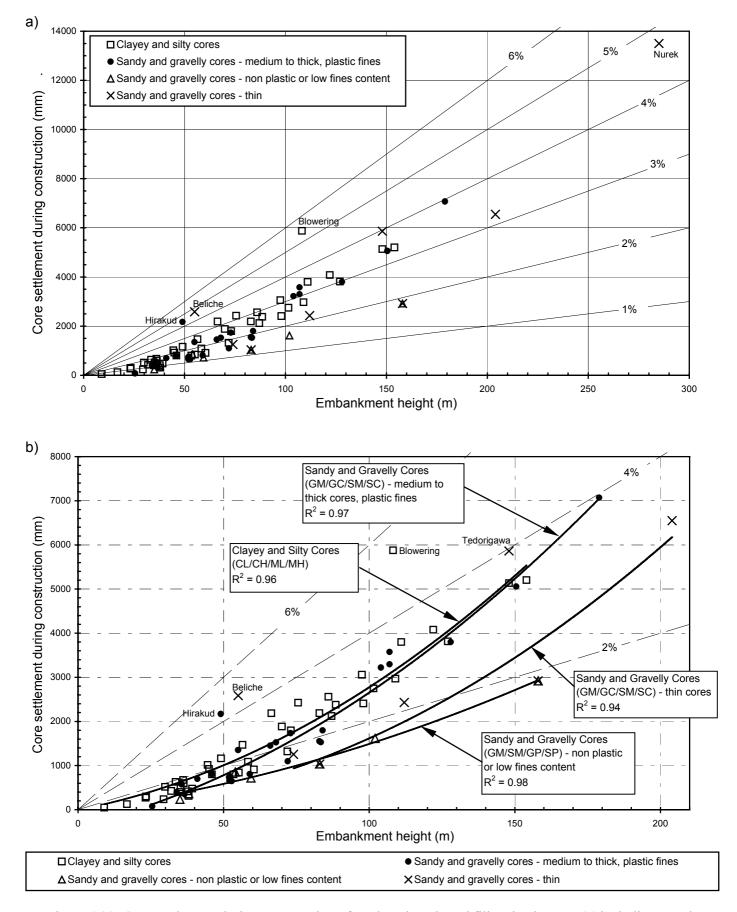


Figure 4.23: Core settlement during construction of earth and earth-rockfill embankments (a) including Nurek dam, and (b) excluding Nurek dam.

Table 4.6: Equations of best fit for core settlement versus embankment height during construction

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Core Material / Shape Type	No.	Equation for Settlement* ¹	$R^2 *^2$	Std. Err. of
	Cases			Settlement *3
				(mm)
Clay cores – all sizes	42	Settlement = H(0.152H + 12.60)	0.96	275
Sandy and gravelly cores:				
medium to thick, plastic	25	Settlement = H(0.183H + 7.461)	0.97	290
thin	5	Settlement = H(0.136H + 2.620)	0.94	635
non-plastic	7	Settlement = H(0.063H + 8.57)	0.98	130

Note:

Table 4.7: Equations of best fit for core settlement (as a percentage of embankment height) versus embankment height during construction

Core Material / Shape Type	No.	Equation for Core Settlement *1	$R^2 *^2$	Std. Err. of
	Cases	(as a percentage of embankment height)		Settlement *3
				(%)
Clay cores – all sizes	42	Settlement (%) = $0.179 H^{0.60}$	0.77	0.41
Sandy and gravelly cores:				
medium to thick, plastic	25	Settlement (%) = $0.079 H^{0.76}$	0.81	0.39
thin	5	Settlement (%) = $0.063 H^{0.72}$	0.70	0.47
non-plastic	7	Settlement (%) = $0.187 H^{0.46}$	0.71	0.24

Note:

^{*1} settlement in millimetres, embankment height H in metres. *2 R^2 = regression coefficient *3 Std. Err. = standard error of the settlement

settlement as a percentage of the embankment height, embankment height *H* in metres.

^{*} 2 R 2 = regression coefficient

^{*3} Std. Err. = standard error of the settlement (percent of embankment height)

4.2 DEFORMATION ON FIRST FILLING

The following discussion summarises the factors affecting the deformation behaviour of earthfill and earth – rockfill embankments during first filling of the reservoir, drawing on relevant information from the published literature. The discussion does not go into case study specific details, but some of the issues raised are discussed in greater detail in the individual case study analysis in Section 6.0.

Analyses of first filling generally consider the main water barrier element to be impermeable during first filling and the water load to act on the upstream face of this zone. Permeable earth and rockfill zones upstream of the main earthfill zone will become saturated during first filling. Therefore, the factors affecting the deformation behaviour of embankments during reservoir filling are primarily the water load, the compressibility properties of the earthfill core and downstream shoulder, and the susceptibility to collapse compression on wetting of the permeable earth and rockfills zones upstream of the core. Other factors that can affect the embankment deformation during first filling include time dependent deformations, such as creep and earthfill consolidation, and differential settlement of the foundation neither of which is considered in detail in this section.

Nobari and Duncan (1972b), Sherard (1973) and others, describe the effect of the water load (Figure 4.24) as threefold:

- (i) Applied water load on the upstream side of the core or earthfill forming the water barrier, resulting in a net increase in the total stresses within and downstream of this zone. The resultant deformation is downstream and downward.
- (ii) Applied water load on the upstream foundation. Sherard (1973) comments that the foundation has an important influence on the embankment deformation if it is compressible or susceptible to collapse compression on wetting. Differential foundation settlements can arise due to differential loading conditions from the reservoir and in cases where the foundation differs under the embankment, such as in the case of a cut-off trench to bedrock under the central core.
- (iii) Decrease of effective stresses in the permeable earth and rockfill zones upstream of the core. Deformations will be upward and upstream due to the unloading of effective stresses, but are likely to be small due to the high tangent modulus on unloading and reloading of gravels and rockfills compared to their modulus during loading.

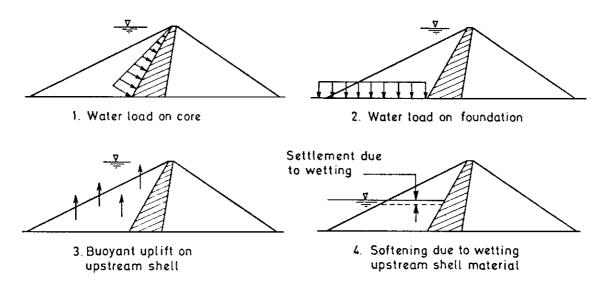


Figure 4.24: Effect of reservoir filling on a zoned embankment (Nobari and Duncan 1972b)

4.2.1 Effect of Water Load on the Core

Assuming the central earthfill zone of the embankment behaves in undrained conditions during first filling, the effect of reservoir filling will be for a net increase in total stress acting on the upstream face of the core zone. The changes in total stress within the central earthfill zone and downstream shoulder zones due to first filling will depend on:

- The magnitude of the change in total stress acting on the upstream face of the core due to first filling, its
 location and the direction in which it acts as controlled by the orientation of the upstream face of the
 earthfill zone.
- The existing stresses within the embankment at end of construction.

The magnitude of the total stress difference is greatest where the load acts on the upstream slope of the embankment, as in the case of a homogeneous embankment or embankments with facing elements designed as the main water barrier element (e.g. concrete or asphalt faced rockfill dams). For typical slopes of homogeneous earthfill embankments of greater than 2 - 3H to 1V, the applied direction of the water load is dominantly vertical (Figure 4.25) and stress changes within the embankment will mainly occur in the upstream slope. Resultant deformations of the crest or under the embankment axis will therefore generally be small.

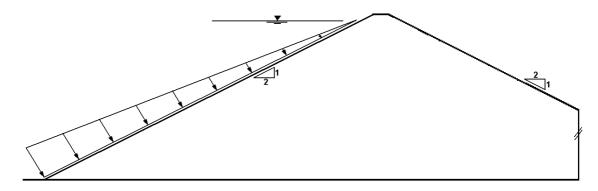


Figure 4.25: Water load acting on a homogeneous earthfill dam.

For zoned earthfill embankments with near vertical narrow cores and free draining gravels or rockfill in the upstream shoulder the direction of the applied water load is near horizontal (Figure 4.24, Part 1). The magnitude of the difference in applied stress on the upstream face from end of construction to reservoir full will be much less than for water loads acting on the outer upstream face. It will be dependent on the existing stress conditions within the embankment at end of construction, the hydrostatic stress from the water and the reduction in effective stresses in the upstream rockfill on reservoir filling.

The earlier finite difference analysis (Section 4.1.1.2, Figure 4.4) has been extended to compare the lateral stress distributions at end of construction and end of first filling for two cases:

- The central earthfill zone having the same compressibility properties as the rockfill.
- A wet placed core of low undrained strength (Case 3).

Given that a linear elastic perfectly plastic Mohr-Coulomb model has been used for the embankment materials the stress distributions will not necessarily be accurate at the end of first filling but they will be indicative of the general trend. Where the core has similar compressibility properties to the shoulder the lateral stresses at end of construction are relatively low (Figure 4.26a) compared to the core of low undrained strength (Figure 4.27a), as previously discussed in Section 4.1.1.2. Assuming the core to behave undrained during reservoir filling, the change in total lateral stress in the core and downstream shoulder on reservoir filling is shown to be greater for the core with similar compressibility properties to the rockfill (Figure 4.26b). For the low undrained strength

core, lateral stress changes in the core and downstream rockfill on reservoir filling are smaller in comparison (Figure 4.27b) because of the already high lateral stresses present at the end of construction.

For both cases the lateral stress conditions in the core and downstream rockfill increase so resultant lateral deformations will be in a downstream direction. The high horizontal component of the applied stress vector from the water load acting on the upstream face of the core, compared to its vertical component, results in greater changes in lateral than vertical stress in the core and downstream shoulder. Therefore, deformations will be dominantly horizontal. In reality settlements are observed on the crest and downstream shoulder during first filling, most likely due to a combination of time dependent creep and consolidation, and potentially due to collapse compression in the downstream shoulder.

The magnitude of the deformation will depend on the changes in stress conditions, the compressibility properties of the core and downstream rockfill and the embankment geometry. For the downstream rockfill shoulder the existing lateral stress conditions are shown to be important, but also important is the effective stress path on first filling, which is for a net decrease in deviatoric stress and net increase in bulk stress.

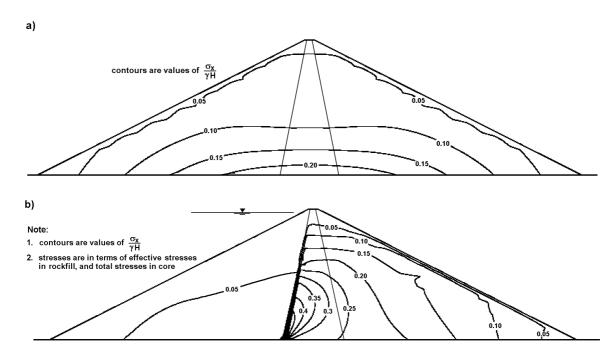


Figure 4.26: Lateral stress distribution at (a) end of construction, and (b) reservoir full condition, for central core earth and rockfill embankment with core of similar compressibility properties to the rockfill.

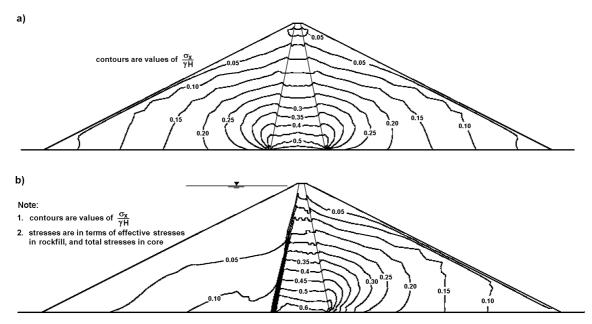


Figure 4.27: Lateral stress distribution at (a) end of construction, and (b) reservoir full condition, for a central core earth and rockfill embankment with wet placed core of low undrained strength.

4.2.2 Collapse Compression During First Filling

Collapse compression or collapse settlement on wetting of embankment materials is an important factor in the deformation behaviour during first filling for a large number of embankments. In most cases collapse compression is associated with wetting of the upstream rockfill shoulder during first filling and numerous central core earth and rockfill embankments exhibit this effect including El Infiernillo dam (Marsal and Ramirez de Arellano 1967), Cougar dam (Pope 1967), Round Butte dam (Patrick 1967) and Beliche dam (Naylor et al 1997) amongst many others. Collapse compression can also occur in the downstream rockfill or gravel zones following heavy rainfall or due to inundation of the downstream toe, and earthfills are also potentially susceptible.

Evidence of collapse compression during first filling at the main section includes differential settlement across the crest with greater settlement of the upstream shoulder, crest spreading and longitudinal crest cracking in some case studies.

The susceptibility of a rockfill to collapse compression is dependent on a number of factors including; the rock substance unconfined compressive strength and its reduction on saturation, moisture condition at placement (dry placed, water added or sluiced), degree of compaction, particle size distribution (coarser rockfills are potentially more susceptible) and applied stress levels prior to saturation. Case study and laboratory test data (Nobari and Duncan 1972a; Marsal 1973; Alonso and Oldecop 2000) indicates the potential for or magnitude of collapse compression is; greater for dry placed rockfills, decreases with increasing compactive effort, and increases with increasing stress level. Dry dumped and poorly sluiced rockfills are particularly susceptible to collapse compression as evidenced by the very large deformations observed at Cogswell dam in 1933 following a very heavy rainstorm (Bauman 1958) and the deformations observed at Strawberry and Dix River dams (Howson 1939) on flooding during construction. Terzaghi (1960) attributed the deformation to a reduction in compressive strength of the rockfill on saturation, mainly in the outer surface of the particles, leading to failure at the highly stressed point contacts in an angular rockfill mass. Rock types that are generally more susceptible to collapse compression due to greater strength reduction of the substance strength on saturation include

sedimentary rock types from sandstones down to mudrocks (mudrocks are potentially more susceptible), limestones, and metamorphic rocks from sedimentary parent rocks. Igneous rocks tend to be less susceptible.

Laboratory compression curves for a rockfill sample in dry and wet states (Nobari and Duncan 1972a) are shown in Figure 4.28. Collapse compression on wetting occurs for the dry rockfill when the stress state is above the normal compression line for wetted or saturated rockfill, and the collapse strain is equivalent to the difference in strain between the dry and wetted states at a given confining stress. Alonso and Oldecop (2000) showed that collapse deformations of similar magnitude to that occurring on sample flooding were obtained by imposing 100% relative humidity on a rockfill sample, indicating that flooding or wetting of the voids between the rock particles was not required for collapse deformation. They concluded that "any situation leading to a change in moisture content in the rock pores is enough to cause collapse deformation", which is consistent with the observation of collapse deformations induced by flooding, such as on reservoir filling, or rainfall.

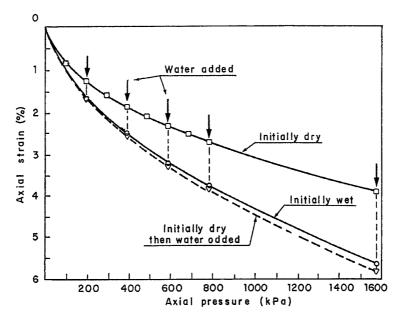


Figure 4.28: Compression curves for dry and wet states and collapse compression from the dry to wet state for Pyramid gravel in the laboratory oedometer test (Nobari and Duncan 1972a).

Earthfills are also susceptible to collapse compression but its occurrence is not as frequently observed in modern embankment dams. This is mainly due to the high compactive effort used and placement at moisture contents close to Standard optimum moisture content.

If we consider the idealised results of a series of constant suction oedometer tests on a compacted cohesive earthfill (Figure 4.29), collapse compression on saturation occurs for a partially saturated soil when at a state, in void ratio – stress space, above that for the saturated soil. At low stress levels heave will occur on saturation. A more typical void ratio – stress relationship for a dry placed well-compacted cohesive earthfill (within 1 to 2% dry of Standard optimum) is for a reduction in matric suction with increasing stress (as shown) due to the compression of air voids in the soil structure and increase in the degree of saturation. This is verified by the pore water pressure response in piezometers during construction (Section 4.1.1.4). Therefore, this reduction in matric suction reduces the susceptibility of the earthfill to collapse compression on wetting compared to the constant suction condition.

Another important aspect for well-compacted cohesive earthfills placed within 1 to 2% dry of Standard optimum is that the air voids retained in the soil structure are not readily removed on wetting to give complete

saturation as they are for dry placed free draining rockfills, sands or gravels. Therefore, on wetting the reduction in matric suction will not be significant as a result of the retained air voids in the soil structure and collapse compression, if any, will be limited. Sampling of the "moist placed" medium plasticity compacted clay from below the phreatic surface at Belle Fourche dam (Sherard 1973) more than 25 years after placement indicated the soil structure still retained an air voids content of about 8%.

Factors that will affect the susceptibility of an earthfill to collapse compression are:

- Placement moisture content relative to Standard optimum. The drier soils are placed the more susceptible
 they are to collapse compression on wetting.
- Compacted density. For a dry placed earthfill, the lower the density ratio of a given soil type the more susceptible it is to collapse compression. The variation in density within a compacted layer is also important. The lower portion of thick placed layers, where the density is often lower, is more susceptible to collapse compression than would be the upper, more heavily compacted part of the layer.
- Earthfill material type. For similarly dry placed earthfills, say at 3% dry of Standard optimum, cohesionless and low cohesion earthfills appear, from the case study data, to be more susceptible to collapse compression than cohesive earthfills

A number of factors, other than those mentioned above, are likely to influence the susceptibility to collapse compression including particle size distribution, soil plasticity, degree of saturation and the ability of the partially saturated soil to retain air voids in the soil structure on saturation.

Examples of collapse compression of earthfills are much more limited than for rockfills in embankment dams. The database includes several suspected examples which are discussed in Section 6.0. The material types involved are mainly silty to clayey sands and gravels that were placed well dry of Standard optimum moisture content.

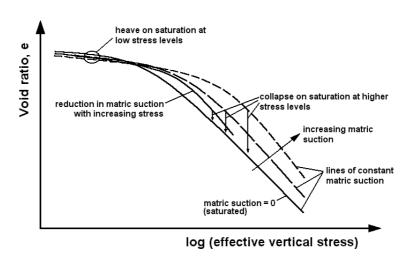


Figure 4.29: Idealised effect of matric suction on collapse compression of earthfills (Khalili 2002).

The effect of collapse compression of the upstream rockfill or gravel zone on the embankment deformation behaviour during first filling is dependent on a number of factors including the strain induced by collapse compression, the embankment zoning and geometry, the strength and compressibility of the embankment materials and the stress conditions at end of construction.

For earthfill cores of relatively low drained compressibility, typified by dry placed well-compacted earthfills, collapse compression of the upstream rockfill on first filling will result in greater settlement of the rockfill relative to the core (Figure 4.30). The deformation at Cherry Valley dam (Lloyd et al 1958) is an example of

this type of deformation where the settlement of the upstream rockfill was more than four times that of the central core and longitudinal cracks on the crest were observed at the upstream core / rockfill interface. Shear stresses, acting as down drag on the upstream face of the core, are developed at the upstream core / rockfill interface due to the differential settlement (Squier 1970).

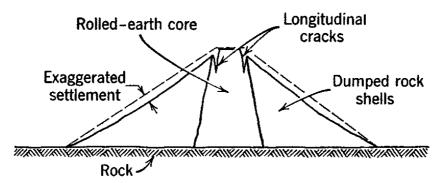


Figure 4.30: Cracking caused by post construction differential settlement between the core and the dumped rockfill shoulders (Sherard et al 1963).

For zoned earth and rockfill embankments with wet placed cohesive earthfill cores the deformation behaviour of the core during first filling is significantly different to that for dry placed earthfills. On collapse compression of the upstream rockfill the core, because of its low undrained strength, is not able to support the load that could be transferred to it as a result of differential settlement. In effect the core is reliant on the shoulders for support. Therefore, if the upstream shoulder settles due to collapse compression on wetting then the core must deform with the upstream rockfill, largely as plastic type deformations, and limited differential settlements occur at the upstream interface.

The deformation behaviour of the wet placed clay core of El Infiernillo dam during first filling (Marsal and Ramirez de Arellano 1967; Squier 1970; Nobari and Duncan 1972b) is an example of this type of deformation behaviour. Collapse compression of the dry placed upstream rockfill occurred during first filling. The measured vertical strains in the core were similar in magnitude and elevation to those in the upstream rockfill, indicating that the core essentially deformed with the upstream shoulder. The actual deformation behaviour at El Infiernillo is more complex than summarised due to the influence of the upstream filters and effect of lateral spreading of the core. The deformation behaviour at El Infiernillo is discussed further in Section 6.3 and in Section 1.9 of Appendix B.

Two-dimensional finite difference analysis was undertaken to further assess the effects of collapse compression of the upstream rockfill and the shear strength properties of the core on the embankment deformation behaviour during first filling. The modelled embankment (Figure 4.31) consisted a 100 m high central core earth and rockfill dam with narrow core on a rigid bedrock foundation. Filters and zoning of the rockfill shoulders were omitted for simplicity. Two different core types were modelled; a very stiff dry placed core of low drained compressibility and high undrained strength, and a wet placed core of low undrained shear strength that behaves undrained during construction and on first filling. For each core type the upstream rockfill was modelled with and without collapse compression. Similar properties were used for the downstream rockfill in each case. The shear strength and compressibility properties of the core and rockfill are given in Table 4.8 for the following four analysed cases:

- Case 8 "dry" placed clay core of very stiff strength consistency and relatively low drained compressibility with collapse compression in the upstream rockfill.
- Case 9 "dry" placed clay core, but without collapse compression in the upstream rockfill.
- Case 10 "wet" placed clay core of low undrained strength with collapse compression in the upstream rockfill.

• Case 11 – "wet" placed clay core, but without collapse compression in the upstream rockfill.

Embankment construction was modelled in ten stages each of ten metres thickness. A Mohr-Coulomb linearelastic perfectly plastic model was used for both the core and rockfill during construction. The rockfill was modelled using the loading or "construction" compressibility properties. At end of construction the deformations were initialised to zero.

First filling was modelled assuming rapid filling of the reservoir such that the permeable upstream rockfill was saturated and the core was effectively impermeable. A load was applied to the upstream face of the core equivalent to the hydrostatic water load and buoyant unit weights were used in the saturated portion of the upstream rockfill. A reservoir full condition was at a height of 98 m above the foundation.

First filling was modelled differently depending on whether or not collapse compression of the upstream rockfill was to be incorporated into the analysis. For the cases where collapse compression was not included (Cases 9 and 11) first filling was modelled in one stage. Changes to the material properties on first filling were:

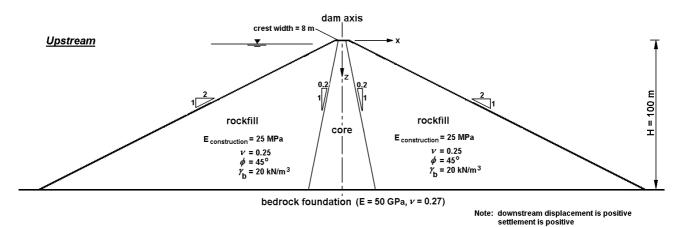
- Upstream rockfill buoyant unit weight and unloading compressibility properties of the saturated portion.
- Downstream rockfill increased shear modulus (the bulk modulus remained the same) to reflect the
 effective stress path in the downstream shoulder, which shows an increase in bulk stress and decrease in
 deviatoric stress on first filling.

For the cases incorporating collapse compression of the upstream rockfill (Cases 8 and 10) first filling was modelled in a series of 10 stages; the first 9 stages in 10 m increments of rising reservoir level and the last stage as an 8 m increment to full supply level. Collapse compression was modelled as per the method by Justo (1991), which involved:

- For the newly saturated layer of each stage, collapse compression was modelled by:
 - Reduction of the effective stresses within the newly saturated layer of the filling stage. The reduction factor is equivalent to the compressibility ratio between the "wetted" and "dry" states of the rockfill (Figure 4.32). In this case a ratio of 0.7, or a 30% reduction, was used.
 - Reduction of the Young's Modulus of this layer to those for the "wetted" rockfill in loading.
- For previously wetted layers, the unloading / reloading moduli were used for the saturated upstream rockfill.

The downstream rockfill for Cases 8 and 10 was also modelled with an increased shear modulus as per the modelling for Cases 9 and 11. In addition, the "dry" placed, very stiff core for the model incorporating collapse compression of the upstream rockfill (Case 8) was also modelled with an increased shear modulus. The ratio of the loading to unloading moduli used was a factor of 10 for the rockfill and a factor of 5 for the earthfill core.

The deformation results on first filling from the finite difference modelling are shown in Figure 4.33 to Figure 4.36 for Cases 8 to 11 respectively. The settlement and displacement of the embankment at the end of first filling are presented for all cases, and for the reservoir at 40% of the embankment height for Case 10 only (wet placed core modelling collapse compression of the upstream rockfill). It should be noted that the deformations are not (and are not intended to be) an accurate representation of actual deformation due to the use of simplistic models for the strength and compressibility of the embankment materials. Importantly, though, the general trends of the deformation behaviour are considered to be a reasonable representation of those in reality.



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Figure 4.31: Embankment model for finite difference analysis during first filling.

Table 4.8: Properties of central core used in finite difference analyses

	"Dry Plac	ced" Core	"Wet Placed" Core		
Embankment Zone and Properties	Case 8	Case 9	Case 10	Case 11	
	(collapse)	(no collapse)	(collapse)	(no collapse)	
Earthfill core:					
- modulus, E (MPa)* ¹	30	30	20	20	
- Poisson's ratio, v	0.33	0.33	0.48	0.48	
- cohesion, c' or c _u (kPa)	5	5	50	50	
- friction angle, ϕ' or $\phi_{\rm u}$ (0)	32	32	2	2	
- bulk unit weight, γ (kN/m³)	20	20	20	20	
Rockfill:					
- Poisson's ratio, v	0.25	0.25	0.25	0.25	
- Cohesion, c' (kPa)	0	0	0	0	
- Friction angle, φ' (⁰)	45	45	45	45	
- Bulk unit weight, γ _{bulk} (kN/m ³)	20	20	20	20	
- Saturated unit weight, γ _{sat} (kN/m ³)	22.3	22.3	22.3	22.3	
- Young's Modulus (MPa)					
- loading (construction)	25	25	25	25	
- "wet" modulus (on collapse)	17.5	na	17.5	na	
- unloading / reloading for non-collapse.	250	250	250	250	
- unloading / reloading post collapse.	175	175	175	175	

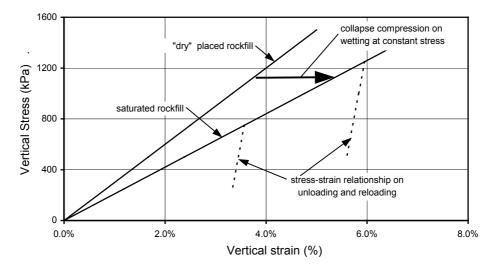
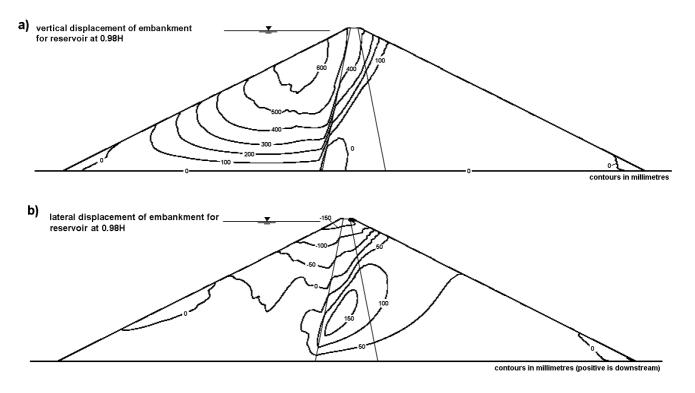


Figure 4.32: Model of the stress-strain relationship of the upstream rockfill incorporating collapse compression for the linear-elastic range in one-dimensional vertical compression.



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Figure 4.33: Case 8 – "dry" placed, very stiff core modelling collapse compression in the upstream rockfill; numerical results of (a) vertical and (b) lateral displacement on first filling.

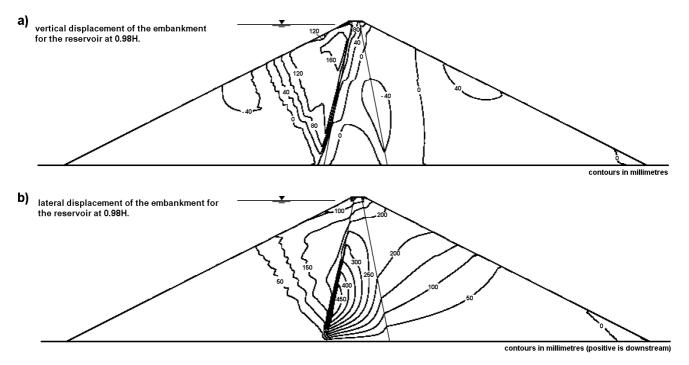
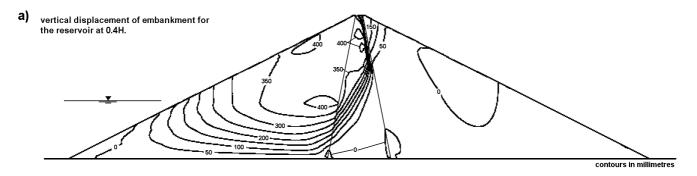
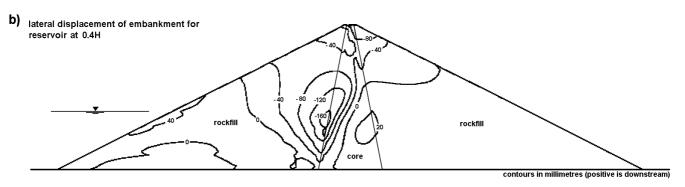
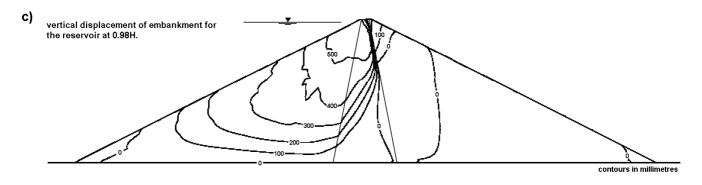


Figure 4.34: Case 9 – "dry" placed, very stiff core without collapse compression; numerical results of *(a)* vertical and *(b)* lateral displacement on first filling.



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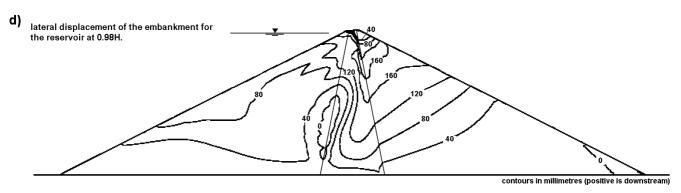
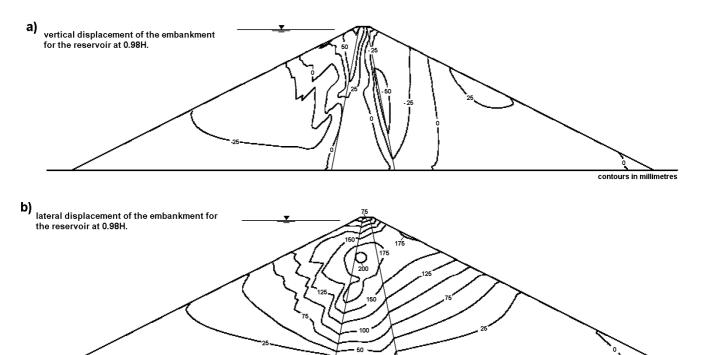


Figure 4.35: Case 10 – "wet" placed clay core of low undrained strength modelling collapse compression in the upstream rockfill. Numerical results of vertical and lateral deformation on first filling; (a) and (b) for reservoir at 40% of embankment height, (c) and (d) for reservoir at 98% of embankment height.

contours in millimetres (positive is downstream)



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Figure 4.36: Case 11 – "wet" placed clay core without collapse compression; numerical results of (a) vertical and (b) lateral displacement on first filling.

For the cases where collapse compression of the upstream rockfill was not modelled (Cases 9 and 11) the results show:

- Settlements in the upstream rockfill and core are comparatively very much smaller than those for the cases incorporating collapse settlement of the upstream rockfill.
- Upward vertical displacements are observed in the mid to lower upstream shoulder due to decreases in the
 effective vertical stress on first filling. These are obscured in the collapse compression analyses due to the
 collapse compression.
- Displacements are downstream in the core and shoulders.

For the cases where collapse compression of the upstream rockfill was modelled (Cases 8 and 10) the results show that whilst the settlements in the upstream shoulder are similar for the different core types, the settlements within the core show markedly different patterns of behaviour. For the "dry" placed core model (Case 8) the settlement trend shows (Figure 4.33a) a large differential settlements at the upstream interface of the core. This pattern of settlement is similar to that observed from the case studies, except that the differential settlement in the crest region is often more pronounced in the case studies. This is partly a result of the relatively large shear type deformation that has developed within the core in the model. By adopting higher shear strength parameters for the core greater differential settlement occur at the upstream core / rockfill interface and smaller settlements within the core.

In contrast, the numerical model for the "wet" core of low undrained shear strength shows virtually no differential settlement at the upstream core interface (Figure 4.35a and c), indicating the core is deforming with the upstream rockfill shoulder as observed in case studies with wet placed clay cores such as El Infiernillo and Canales (Giron 1997, refer Section 2.1 of Appendix B) dams. Instead, a concentration of vertical settlement is observed in the lower part of the core and a large differential settlement at the downstream interface of the core. The concentration of deformation at the downstream interface of the core is clearly demonstrated at Canales dam.

The lateral displacement of the core on first filling shows a similar "bow" shaped pattern for all four cases, with the region of maximum downstream displacement on the dam axis occurring in the central region of the core. The displacement for the cases incorporating collapse compression show a tendency for regions of the core and upstream shoulder to displace upstream. For the "wet" placed core (Case 10) the lateral deformation of the core was largely upstream for the reservoir at 40% of the embankment height (Figure 4.35b) and then largely downstream at the end of first filling (Figure 4.35d), still retaining its bow shape. This upstream then downstream deformation pattern of the crest is not uncommon in zoned embankments with narrow, near vertical central cohesive cores placed wet of Standard optimum; it was observed at El Infiernillo, Thomson, Gepatsch and Nurek dams.

4.2.3 Lateral Surface Deformation Normal to the Dam Axis During First Filling.

As discussed in Section 4.2.1 a significant portion of the post construction lateral displacement occurs during first filling, particularly for embankments with thin to medium width central cores and permeable upstream fill (mainly central core earth and rockfill dams). The lateral surface displacements during first filling for the case studies of earthfill and zoned earth-rockfill embankments are presented in the following figures and tables.

For analysis of the post construction surface deformation, the embankment has been divided into three regions (Figure 4.37); the mid to downstream region of the crest, the mid to upper region of the downstream slope and the upper upstream slope to upstream crest region. The regional division was primarily established to separate the deformation of the core of zoned embankments from that of the upstream and downstream shoulders due to the sometimes large difference in observed deformation behaviour, particularly where poorly compacted rockfill was used in the shoulders. It is recognised that because the deformation of the shoulders affects that of the core, any assessment of the embankment deformation requires consideration of the embankment as a whole, but given the large number of case studies within the database it is impractical to do so for all dams. In the discussion on individual embankments in Section 6.0 and in Appendix B the deformation of the embankment as a whole is discussed for selected case studies.

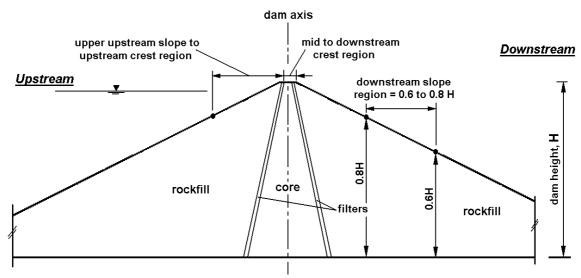


Figure 4.37: Regional division of the embankment for analysis of post construction surface deformation.

Representative surface monitoring points (SMP) for an embankment were generally taken from near to the maximum section, unless otherwise identified. Other selection criteria for SMPs are:

• For the downstream slope SMPs were selected from the slope region in the range 0.6 to 0.8 times the embankment height, which is often where the greatest displacements were measured. Embankments were not excluded if the only SMPs on the downstream slope were outside this region.

- For zoned embankments with thin to medium width cores SMPs representing the upper upstream shoulder to upstream crest region were preferentially selected from the upstream edge of the crest provided the SMP was located over the upstream shoulder zone.
- For embankments with very broad earthfill zones (including earthfill embankments) the crest region was
 considered as a whole; i.e. from the upstream edge to the downstream edge. For these embankments SMPs
 from only the upper upstream slope region were included with the analysis of the upper upstream slope to
 upstream crest region.

Figure 4.38 to Figure 4.40 present the case study results for the lateral surface displacements during first filling:

- Figure 4.38 for the mid to downstream region of the crest. The data has been sorted based on the width of the central core, and the material type and compaction of the downstream shoulder.
- Figure 4.39 for the mid to upper region of the downstream slope, sorted based on the material type and compaction of the downstream shoulder.
- Figure 4.40 for the upper upstream slope and upstream crest region, sorted based on material type and compaction of the upstream shoulder.

The terms "well-compacted", "reasonable to well-compacted", "reasonably compacted" and "poorly compacted" used to describe the compaction of the rockfill in the embankment shoulders are defined in Section 1.2.2. The term "gravels" refers to free draining coarse grained gravelly and bouldery soils with very low fines content (less than about 5 to 8% passing 75 micron). The term "earthfill" covers a broad range of material types from clays to silty gravels to weak rockfills that breakdown significantly on compaction.

Zero time for measurement of displacement, t_0 , is established at the end of embankment construction. In most cases the displacement for each case study captures a large portion of the period of first filling, however, for several cases where the embankment impounded water to relatively high levels during construction some of the displacement due to first filling would have been missed (e.g. Vatnedalsvatn). No differentiation with respect to time of first filling is taken into consideration in the figures. In some cases the reservoir was filled within a matter of days to weeks whilst for others first filling took many years, more than 10 years for several cases.

Highlighted in the figures are those embankments where the rockfill was known to have been placed without the addition of water. These cases are highlighted because of the susceptibility of dry placed rockfill, particularly if not well compacted or if weathered, to collapse compression on wetting. It is recognised that for several of these highlighted embankments the rockfill was likely to have been wetted from rainfall in regions where the climate is very wet, but they have been included rather than to make this judgement.

The figures present information on the general trend and magnitude of the lateral displacement on first filling, and highlight case studies with "abnormally" large deformation or with trends different to the broader group. A more detailed analysis of the overall post construction lateral deformation behaviour is presented in Section 4.3 and the discussion on individual embankments in Section 6.0.

For the displacement of the crest (Figure 4.38) the material forming the downstream shoulder and the width of the core region do have some influence (refer below) but generally speaking it is difficult to establish bounds of deformation other than a general range. Reasons for this are considered to include factors such as the time of first filling, the stress conditions within the downstream shoulder prior to first filling (refer Section 4.2.1), the effect of collapse settlement of the upstream rockfill and resultant core deformation (refer Section 4.2.2), positioning of the SMP on the crest, valley shape, and the curvature of the embankment axis amongst other factors. Attempts to further differentiate the data for one or more of these aspects often resulted in too great a dilution of the database to draw any significant conclusions.

Table 4.9 presents a summary of the lateral crest displacement on first filling. The displacement range for most case studies is from 50 mm upstream to 300 mm downstream (Figure 4.38). For most groups in Table 4.9 the range is less than about 100 to 200 mm, or less than 0.1 to 0.2% of the embankment height. Greater displacements are observed for dams with reasonably to poorly compacted rockfill in the downstream shoulder or embankments with cores of silty sands to silty gravels and of thin to medium width.

Isolated cases of large displacement also occur within the other groupings. For the embankments with cores of very broad width, the relatively large upstream displacement for Rector Creek and Mita Hills is possibly due to collapse compression of dry placed dominantly sandy earthfills, for the slide area at San Luis dam due to the upstream trending hill slope on which this section of the embankment was constructed, and for Horsetooth dam the foundation is thought to have had a significant influence on the displacement. These cases are discussed further in Section 6.0. South Holston would also appear to be an outlier in its sub-grouping with an upstream displacement of 64 mm (-64 mm) on first filling. At Glenbawn dam the crest SMP is located at the downstream edge over the poorly compacted rockfill zone, which would account for the large displacement measured.

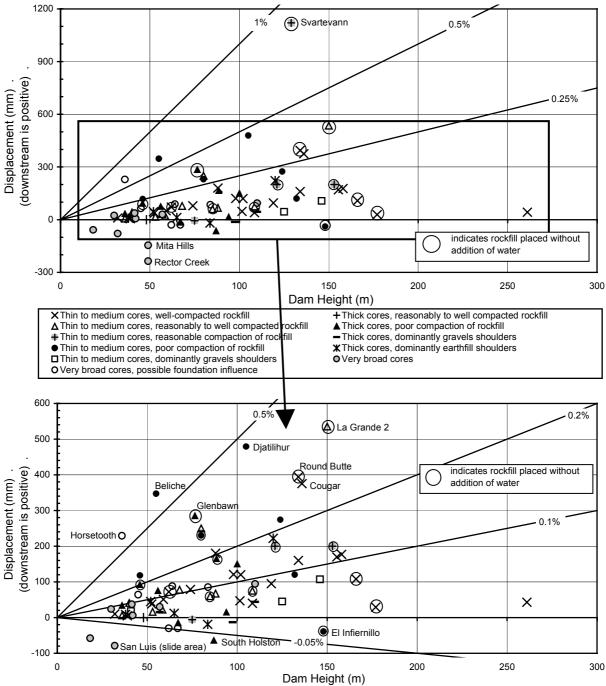
Table 4.9: Lateral displacement of the crest (centre to downstream edge) on first filling.

Core	Core Downstream Shoulder		Core *2	No.	Displacemen	nt Range	
Width	Material	Rating *1	Classification	Cases	(mm)	% of dam height * ³	Comments
			CL/CH/GC/SC, wet	13	29 to 180	0.0 to 0.12	
		Well-comp	CL/SC/GC, dry	5	5 to 80	0.0 to 0.12	
			SM/GM – dry and wet	3	177 to 394	0.10 to 0.30	Naramata (177 mm), Cougar (375 mm), Round Butte (394 mm)
Thin to	Rockfill	Reas to well	CL/CH/SC/GC	5	6 to 78	0.0 to 0.11	
medium		Reas to well	SM/GM	5	6 to 535	0.03 to 0.36	LG2 (535 mm), Frauenau (250)
		Reas	All types	4	-6 to 1120 (most < 200)	-0.01 to 0.17	Svartevann = 1120 mm (SM core)
		Poor	All types	7	-39 to 480	0.09 to 0.63	No correlation to core soil type, Beliche = 347 mm (0.63%), Djatiluhur = 480 mm (0.46%)
	Gravels	-	All types	3	18 to 107	0.04 to 0.07	
	All	cases with thic	k cores	19	-64 to 285 (most -19 to 222)	-0.02 to 0.20	
		Reas to well	All types	1	0	-	Maroon dam
Thick	Rockfill	Poor	All types	9	-64 to 285 (most –15 to 165)	-0.02 to 0.19	Sth Holston = -64 mm (-0.07%), Glenbawn = 285 mm (0.37%)
	Gravels	-	All types	4	-13 to 44	-0.01 to 0.04	
	Earthfill	-	All types	5	-19 to 222	-0.02 to 0.19	Navajo = 222 mm, rest < 45 mm
					(most –19 to 45)	(-0.02 to 0.09)	
Very Broa	ad		All types	15	-236 to 229 (most –58 to 94)	-0.02 to 0.14	Rector Creek = -236 mm Mita Hills = -146 mm San Luis (slide area) = -79 mm Horsetooth = 229 mm

Notes: *1 compaction rating of rockfill; well-comp = "well-compacted", reas to well = "reasonably to well compacted", reas = "reasonable compaction", poor = "poorly compacted".

^{*2} symbols represent soil classification to Australian Standard AS 1726-1993, "wet" = placed at or on the wet side of Standard optimum moisture content, "dry" = placed on the dry side of Standard optimum.

^{*3} range of displacement as a percentage of dam height excludes possible outliers (Svartevann, South Holston, Glenbawn, Rector Creek, Mita Hills, San Luis (slide area) and Horsetooth dams).



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Figure 4.38: Lateral displacement of the crest (centre or downstream region) on first filling versus embankment height (displacement is after the end of embankment construction).

Further details on the embankments comprising the 13 largest downstream displacements on first filling (excluding the very broad earthfill case studies) are summarised in Table 4.10. All have a lateral displacement equal to or greater than 200 mm downstream on first filling. These case studies highlight several factors associated with high lateral displacements:

- The case studies are predominantly of central core earth and rockfill dams with thin cores.
- The predominant core type is silty sands to silty gravels, particularly for the larger deformations (4 of the top 5).
- For most cases (8 of 13) the downstream rockfill is of poor to reasonable compaction.
- In 7 cases the downstream rockfill was known to have been placed without the addition of water. Of these, 2 of the 3 well or reasonably to well compacted cases were placed without water, all the reasonable

compaction cases were placed without water and 3 of the 4 poorly compacted cases were placed without the addition of water.

The data indicates that potentially important factors leading to relatively large downstream displacement on first filling are:

- Collapse settlement of dry placed or poorly compacted rockfills in the downstream shoulder, possibly as a result of post construction wetting from heavy rainfall.
- Relatively high increases in stress level in the downstream rockfill from the applied water load. The earlier finite difference analysis for zoned embankments (Section 4.1.1.2, Figure 4.4 and Section 4.2.1, Figure 4.26) showed that where the compressibility properties of the core are similar to that of the shoulders the lateral stresses are relatively low at end of construction, and that on first filling the magnitude of increase in lateral stress in the core and downstream shoulder is relatively high. Following on from this, it is reasonable to consider that potentially larger downstream displacements occur for earth and rockfill embankments with thin to medium width earthfill cores of well-compacted silty sands to silty gravels. Although not conclusive, it may be a factor leading to large displacement on first filling.

A similar pattern of behaviour as the crest is observed for the lateral displacement of the mid to upper region of the downstream slope on first filling (Figure 4.39). The data shows that:

- For embankments with very broad earthfill zones or earthfills in the downstream shoulder, displacements are typically less than 100 mm downstream (or less than 0.1% to 0.15% of the embankment height). The much greater displacement at Horsetooth dam is due to the influence of the compressible foundation.
- For embankments with gravels in the downstream shoulder, displacements are typically less than 100 to 150 mm (or less than 0.11% of the embankment height). The higher displacement at Trinity dam (175 mm, 0.11%) may reflect the use of dozer track rolling and the limited amount of water used in compaction of the gravels.
- For embankments where wetted rockfills are used in the downstream shoulder, displacements are typically less than 100 to 200 mm (or less than 0.12 to 0.18% of the embankment height). The displacements at Ataturk and Beliche are clear outliers to the general trend, and Blowering and Windemere are slightly above the general trend. At Beliche, Blowering and Ataturk dams the higher displacements are possibly due to the use of weathered materials and/or rockfills susceptible to strength loss on wetting. At Windemere placement in 2 m layers and possibly without the addition of water in the outer downstream slope may be a factor.
- For embankments where dry placed rockfills are used in the downstream shoulder, displacements are typically in the order of 150 to 900 mm, or in the range 0.10 to 0.85% of the embankment height. Collapse compression on wetting due to rainwater infiltration is possibly a significant factor in the large displacements observed. Notably, greater displacements are observed for the dry placed and reasonably to poorly compacted rockfills (greater than 0.2 to 0.25%), which will be more susceptible to collapse compression on wetting. For the dry placed, well and reasonably to well compacted rockfills displacements are generally less than 0.25 to 0.30% of the embankment height.

Table 4.10: Thirteen cases with largest downstream crest displacement on first filling

	Height	Downstream		Core *	3	Do	wnstrean	n Shoulder	Lateral	Time to First	% of First
Dam Name	(m) *1	Slope * ² (H to 1V)	Width	Class ⁿ	Moisture Content	Material	Rating * ⁴	Placement *5	Displacement (mm)	Filling * ⁶ (years)	Filling * ⁷
Svartevann	129	1.43H	Thin (u)	SM/GM	0.4% wet	Rockfill – granite & gneiss	reas	2m lifts, no water, 8p 13t SDVR	1120	2.23	30%
La Grande 2	150	1.6H	Thin (u)	SM	(1% dry to 2% wet), wet?	Rockfill – granite	reas to well	0.9 to 1.8m lifts, no water, 10t SDVR	535	1.2	100%
Djatiluhur	105	1.3H (upper)	Thin	СН	2.2% wet	Rockfill – andesite	poor	1 to 2 m lifts, dumped, watered	480	2.25	30%
Round Butte	134	1.7H	Thin	SM	dry ?	Rockfill – basalt	well	0.6m lifts, no water, 4p 9t SDVR	395	0.6	18%
Cougar	136	1.6H	Thin (u)	GM	1.0% wet	Rockfill – basalt & andesite	well	0.6m lifts, 4p 10t SDVR	375	0.67	most
Beliche	55	1.95H	Thin	GC	(± OMC), wet ?	Rockfill – schists & greywacke	poor	1m lifts, light compaction	345	4.1	50%
Glenbawn	76.5	2.5H	Thick	CL	2% dry	Rockfill – limestone	poor	1.8m lifts, dumped, no water	285	4.7	100%
Mud Mountain	124	1.75H	Medium	SM/GM	dry ?	Rockfill – andesite & tuff	poor	12m lifts, dumped & sluiced	275	>17	most
Frauenau	80	1.6H	Thin	SM	dry?	Rockfill	-	No information	250	-	most
Matahina	80	2.3H	Medium	CL	1 to 1.5% dry	Rockfill – ignimbrite	poor	1.0m lifts, no water, dumped	230	0.24	100%
Navajo	120	2.5H	Thick	CL/ML	0.8% dry	Earthfill	-		220	11	most
Gepatsch	153	1.5H	Thin	GM/GC	0.5 to 2% wet	Rockfill - gneiss	reas	2.0m lifts, no water, 4p 9t SDVR	200	1.92	30%
Vatnedalsvatn	121	1.5H	Thin	SM	0.5% wet	Rockfill – quartz, granite & gneiss	reas	Reas (1.5m lifts, no water, 6p 11t SDVR)	200	0.2	10%

Notes: *1 dam height at SMP (surface measuring point)

^{*2} downstream slope angle, horizontal to vertical. Values given are to 1V.

^{*&}lt;sup>3</sup> (u) indicates core oriented slightly upstream. Classifications to Australian Standard AS 1726-1993. Placement moisture content relative to Standard Optimum. Values in brackets are specification, ? indicates not sure but likely from pore water pressure response.

^{*4} compaction rating of rockfill; well-comp = "well-compacted, reas to well = "reasonably to well compacted", reas = "reasonable compaction", poor = "poorly compacted".
*5 shoulder placement. 8p 13t SDVR = 8 passes of a 13 tonne smooth drum vibrating roller.

^{*6} time to first filling after the end of embankment construction.

percent of first filling is a reference to the reservoir level prior to the end of construction. 30% means the reservoir was filled to within 0.3H (H = dam height) of the reservoir full condition. Therefore low percentages indicate the reservoir was or had been close to full supply level prior to the end of construction and high percentages or "most" indicates little water was impounded during construction.

Case studies with large downstream displacement of the crest also had large downstream displacement of the downstream slope. The exception is Navajo dam (crest displacement = 222 mm or 0.19%, downstream slope displacement = 90 mm or 0.07%) where earthfill was placed in the downstream shoulder. Other notable cases with large downstream displacement include:

- Eppalock dam, a 47 m high central core earth and rockfill dam with a medium plasticity sandy clay core of thick width placed on the dry side of Standard optimum (refer Section 1.10 of Appendix B). The shoulders were of dry placed and poorly compacted rockfill (placed in 2 to 4 m thick layers and spread by tractor) derived from basalt, but was weathered and contained a high fraction of finer sized rockfill. The displacement of the downstream slope on first filling (290 mm or 0.63% of the dam height) was much greater than for the crest (90 mm or 0.2% of the dam height).
- Eildon dam, an 80 m high central core earth and rockfill dam with a well-compacted medium plasticity sandy to gravelly clay core of thick width placed on the dry side of Standard optimum (refer Section 1.8 of Appendix B). The shoulders were of dry placed and poorly compacted rockfill (placed in 2 m thick layers with no formal compaction) sourced predominantly from quartzitic sandstone. The outer downstream random rockfill shoulder consisted of "unsuitable" rock that was poorly graded and contained a high fraction of finer sized rockfill. The displacement of the downstream slope on first filling was very large at 660 mm (or 0.83% of the dam height). The crest displacement was not monitored during first filling.
- El Infiernillo, Dartmouth, Thomson and Blowering dams also show much greater displacement of the downstream slope than the crest. All comprise wet placed clayey sand to sandy clay cores of thin to medium width. For these embankments the core is likely to be strongly reliant on the shoulders for support due to its likely low undrained strength and thin to medium width. Therefore, plastic deformation is likely to be a significant factor in the deformation behaviour of the core during first filling of these embankments affected by collapse settlement of the upstream rockfill shoulder on wetting, as described for El Infiernillo dam (after Nobari and Duncan 1972b) in Section 4.2.2. Thomson and Dartmouth show an upstream then downstream displacement during first filling similar to that at El Infiernillo.

At Eildon and Eppalock dams, embankments with thick cores of high undrained strength and relatively low drained compressibility, the displacement records suggest that the displacement of the dry placed, poorly compacted downstream rockfill shoulder is not likely to be greatly influenced by first filling. The large displacement of the downstream slope for these cases is more likely due to the large differential settlement between the well-compacted core and downstream rockfill, and the influence of the downstream slope of the core on the vector of deformation. Collapse compression in the downstream rockfill due to wetting from rainfall would contribute to the large deformations of the downstream slope observed during first filling. Similar reasoning would also explain the differential displacement at Copeton dam (crest displacement = 75 mm, downstream slope = 360 mm) where the medium width clayey sand core was dry placed and no water used during placement of the rockfill.

The deformation behaviour of several of these embankments, including Ataturk dam, are discussed in more detail in Section 6.3 and in Appendix B.

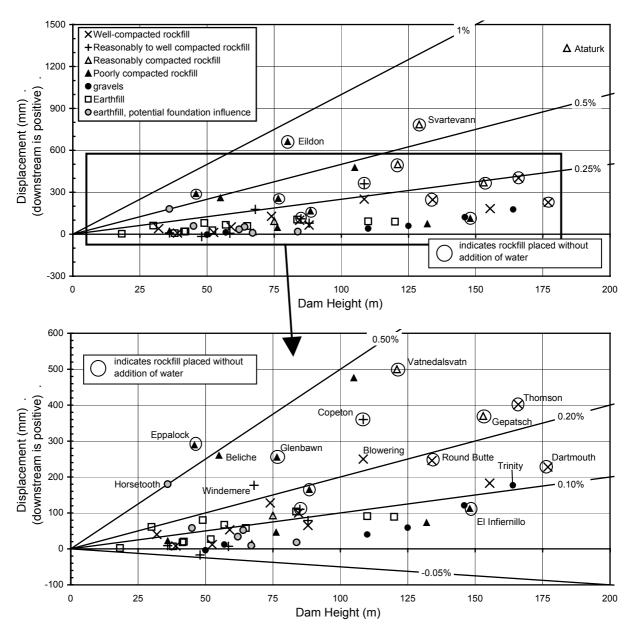


Figure 4.39: Lateral displacement of the downstream slope (mid to upper region) on first filling versus embankment height (displacement is after the end of embankment construction).

The displacement of the upper upstream slope or upstream crest region during first filling (Figure 4.40) highlights Svartevann and Gepatsch dams as outliers to the general displacement trend. For most cases studies the displacement is in the range 200 to 300 mm upstream (-200 to -300 mm) to 300 mm downstream.

The displacement on first filling of the crest and downstream slope were also plotted against the downstream slope (Figure 4.41 and Figure 4.42). The plots indicate a general trend of decreasing magnitude of displacement with flattening of the downstream slope, with some outliers of cases with dry placed rockfill in the downstream shoulder. However, this trend is only evident at the upper bound of lateral displacement as a large number of case studies with relatively steep downstream slope angles show limited deformation on first filling.

The general trends of lateral displacement on first filling are:

- For the crest (centre to downstream region), displacements typically range from 50 mm upstream (-50 mm) to 200 mm downstream, or from -0.02% to 0.20% of the embankment height.
- For the downstream slope (mid to upper region), displacements are typically downstream in the range from 0 up to 200 to 250 mm (or less than 0.2% of the embankment height).

• In the upper upstream slope and upstream crest region, displacements typically range from 200 mm upstream (-200 mm) to 200 mm downstream. The mean is located closer to zero in this region than for the mid to downstream region of the crest.

Much larger downstream displacements on first filling are observed:

- In the crest region for zoned earth and rockfill embankments with thin cores and dry placed or poorly compacted rockfill in the downstream shoulder. In these cases displacements can reach up to 0.3 to 0.65% of the embankment height (up to 600 mm), and in one case 1100 mm or (0.87%) was measured.
- In the mid to upper region of the downstream slope for zoned embankments with dry placed or poorly compacted rockfill in the downstream shoulder. For these cases displacements range from 150 mm to 900 mm (or from 0.10 to 0.85% of the embankment height).

Overall the plots provide useful information on likely bounds of lateral displacement on first filling. They are also useful for identification of embankments that display excessively large displacements on first filling and therefore potentially "abnormal" deformation behaviour. Several of the outliers are discussed further in Section 6.0 and in Appendix B.

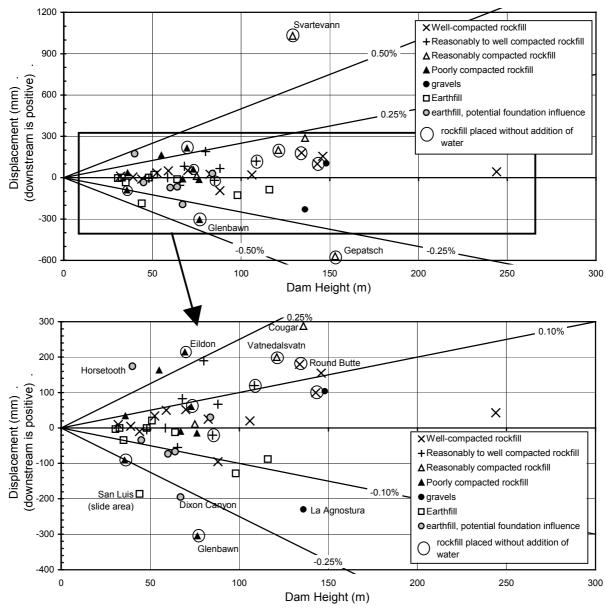
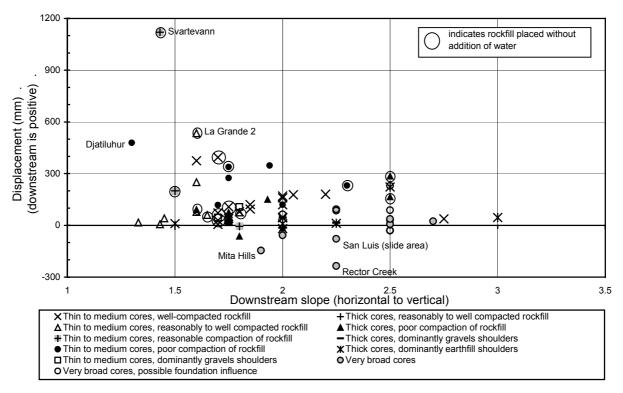


Figure 4.40: Lateral displacement at the upper upstream slope of upstream crest region on first filling versus embankment height (displacement after the end of embankment construction).



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Figure 4.41: Lateral displacement of the crest (centre to downstream region) on first filling versus downstream slope.

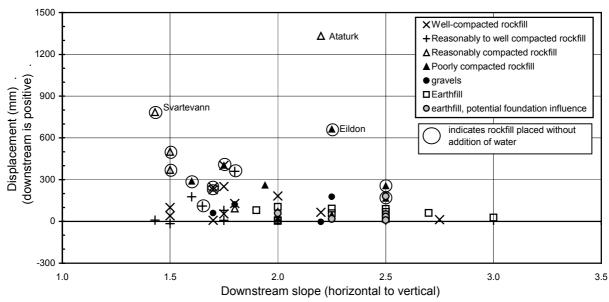


Figure 4.42: Lateral displacement of the downstream slope (mid to upper region) on first filling versus downstream slope.

4.3 POST CONSTRUCTION DEFORMATION BEHAVIOUR OF EARTH AND EARTH-ROCKFILL EMBANKMENTS

The post construction deformation data collected and presented for earthfill and earth-rockfill embankments includes:

- Surface monitoring point (SMP) data on the crest and slopes of the embankment. Both the vertical and horizontal deformation (in the plane normal to the dam axis) is presented.
- Internal vertical deformation data, mainly from IVM gauges, in the central core region of the embankment.

The data presented is usually for the maximum section of the embankment.

Zero time, t_0 , is established at the end of embankment construction to provide a consistent basis point for comparison. The unit of time used for all data is years.

Where possible the deformation records used are the actual records. This has been possible for a large number of the Australian dams and most of the United States Bureau of Reclamation (USBR) dams. For a number of case studies however the data has been digitised from published plots in the literature, and for these cases the deformation as plotted in this report in only a representation of the actual measured data as opposed to the actual data records.

The data is presented mostly in graphical format with data or case studies sorted into what are considered appropriate groupings. The number of plots presented is numerous, particularly for the deformation versus time plots, due to the large number of cases and number of groupings used. Total deformation plots incorporating a large number of case studies are also presented at selected time intervals and these visually provide a broad representation of the database.

The data for analysis of the post construction surface deformation behaviour is divided into the three regions of the embankment as previously identified in Section 4.2.3 (Figure 4.37). They are; the mid to downstream region of the crest, the mid to upper region of the downstream slope, and the upper upstream slope to upstream crest region.

4.3.1 Post Construction Internal Vertical Deformation of the Core

Analysis of the post construction internal vertical deformation of the core was undertaken for those embankments for which detailed results of the internal deformation monitoring were made available. This was generally limited to a number of Australian and USBR dams, but includes several dams published in the literature.

The post construction IVM records were analysed by plotting the cumulative post construction settlement or vertical strain between the individual cross-arms. Assessment of the behaviour was based on the shape of the cumulative settlement or vertical strain profile at a specific time interval after construction and over time.

Typical plots of cumulative settlement are presented in Figure 4.43 and Figure 4.44 for Talbingo (IVM ES1) and Copeton (IVM B) dams respectively. They are considered representative of "normal" type deformation behaviour.

For Talbingo dam the greater post construction vertical strains (average of 0.7% at 24 years post construction) in the lower 40 to 50 m of the 160 m high dam are considered to be related to consolidation due to dissipation of

high pore water pressures. On first filling pore pressures increased by some 200 to 400 kPa above the already high construction pore water pressures. Over the next 20 plus years the pore water pressures slowly dissipated, reducing by as much as 600 to 1100 kPa below the levels at end of first filling. In the mid region of the core the change in pore water pressure due to dissipation was much less than in the lower section and is reflected in the lower vertical strains (average of 0.3% at 24 years post construction). The localised zone of high settlement between 55 and 58 m below crest level is potentially indicative of "abnormal" behaviour. From the data it is evident that the differential settlement between these IVM gauges occurred in the period between 0 and 0.15 years post construction and prior to first filling, and since then has increased only marginally. The reason for the behaviour is not known, but given its timing in relation to end of construction and first filling it is not likely to be "abnormal".

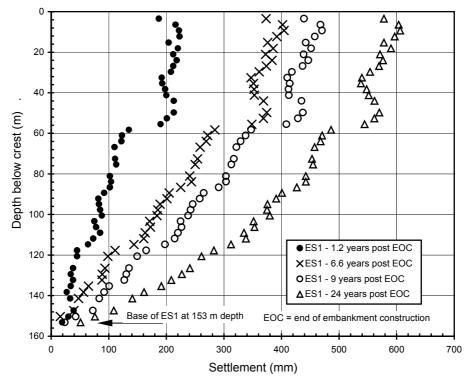
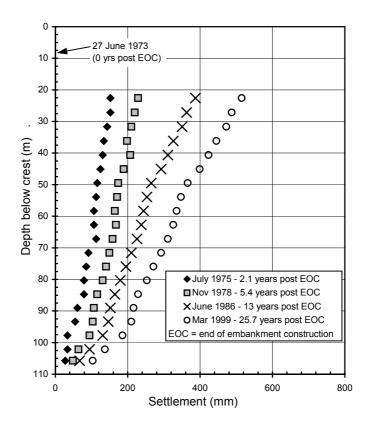


Figure 4.43: Post construction internal settlement of the core at Talbingo dam (IVM ES1 at the main section).

The behaviour of IVM B at the 110 m high Copeton dam is similar to that at Talbingo dam. Higher vertical strains (average of 1.1% at 26 years post construction) were observed in the lower 10 m of the core where pore pressure dissipation post first filling was in the order of 450 kPa. In the mid to upper regions of the core pore pressures were negligible at end of construction and slowly rose post first filling. Vertical strains in this region of the core were on average 0.43% at 26 years post construction, much less than in the lower 10 m of the core.

In contrast the behaviour of IVM A at the 55 m high Bellfield dam (Figure 4.45), located in the thick central clay core, is different. The IVM records from end of construction to February 1987 indicate two potential zone of high localised vertical strain. The records from February 1987 to November 1997 (Figure 4.45b) show a localised zone of higher vertical strain developing at 28 to 30 m below crest level, inconsistent with the localised zones of high strain prior to 1987. The narrow rockfill shoulders at Bellfield were dry placed and poorly compacted, and based on the post construction behaviour of Eppalock dam (of similar design and construction to Bellfield), the "abnormal" behaviour may represent local yielding of the core. Bellfield dam is discussed further in Section 6.3 and in Section 1.2 of Appendix B.



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Figure 4.44: Post construction internal settlement of the core at Copeton dam (IVM B, 9 m downstream of dam axis at main section).

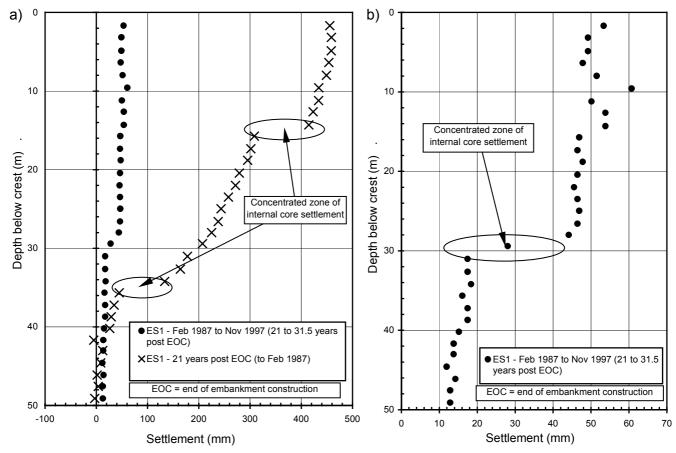


Figure 4.45: Post construction internal settlement of the core at Bellfield dam.

4.3.2 Post Construction Total Deformation of Surface Monitoring Points

The total post construction surface deformations are presented as plots of settlement versus dam height at selected times after the end of construction. Plots of the post construction lateral displacement during first filling are presented in Section 4.2.3. No additional lateral displacement plots are presented in this section.

Data is presented for the three embankment surface regions; the mid to downstream crest, the mid to upper downstream slope, and the upper upstream to upstream crest region (refer Figure 4.37). Base readings for the data are generally in the range from 0 to 0.5 years after the end of construction, most within 0.2 years, and include the period of first filling for most dams.

The data are mainly for embankments on rock foundations. Several embankments on soil/rock or soil foundations have been included where the foundation is considered to have a limited influence on the dam settlement, or where it can be excluded by deducting foundation settlement from base plate or base cross-arm readings from IVM gauges.

The settlement data is presented in the following figures:

- Mid to downstream crest region Figure 4.46 for 3 years, Figure 4.47 for 10 years and Figure 4.48 for 20 to 25 years after the end of construction.
- Mid to upper downstream slope Figure 4.49 for 3 years and Figure 4.50 for 10 years after the end of construction.
- Upper upstream slope to upstream crest region Figure 4.51 for 3 years and Figure 4.52 for 10 years after the end of construction.

Plots for 20 to 25 years after the end of construction for the upstream and downstream slope regions are presented in Section 2.0 of Appendix A.

(a) Mid to Downstream Crest Region

The data for the crest region is sorted based on core width, core material type and placement moisture content. It can be seen from the figures that:

- The post construction crest settlements are generally much smaller than the core settlement during construction (refer Figure 4.23).
- Nearly all dams experience less than 1% crest settlement post construction for periods up to 20 to 25 years and longer after construction.
- Most experience less than 0.5% in the first 3 years and less than 0.75% after 20 to 25 years.

Table 4.11 summarises the typical range of crest settlement for various groupings of core width, material type and placement moisture content. Embankments that are clearly outliers (Ataturk, Djatiluhur, Beliche, Mita Hills, Roxo (near buttress), Rector Creek) and potential outliers (Svartevann, Belle Fourche and Dixon Canyon) have been excluded from this table. Smaller crest settlements (as a percentage of the embankment height) are observed for dry placed clayey sands to clayey gravels regardless of core width and dry to wet placed silty sands to silty gravels. A broader range of crest settlement is shown for clay cores, wet placed clayey sand to clayey gravel cores, and embankments with very broad core widths, most of which are dry placed clays to sandy clays to clayey sands. For zoned earth and rockfill dams, poor compaction of the rockfill is over-represented at the larger end of the range of crest settlement.

Table 4.11: Typical range of post construction crest settlement.

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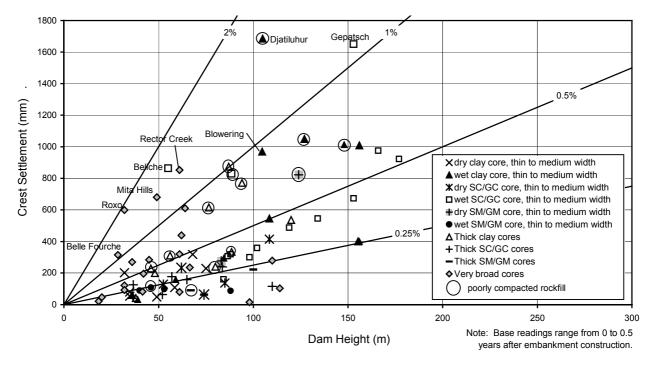
	Core	Properties	No.	Crest Settlement (% of dam height) *1				
Core Width	Class ⁿ	Moisture content	Cases	3 years	10 years	20 to 25 years		
	CL/CH	Dry	9	0.05 to 0.55	0.10 to 0.65	0.20 to 0.95		
Thin to medium	CL/CII	Wet	11	0.04 to 0.75	0.08 to 0.95	0.20 to 1.10		
Timi to medium	SC/GC	Dry	5	0.10 to 0.25	0.10 to 0.40	< 0.5		
		Wet	18	0.15 to 0.80	0.20 to 1.10	< 1.1		
Thin to Thick	SM/GM	All	16	0.06 to 0.30	0.10 to 0.65	< 0.5 to 0.7		
Thick	CL/CH	all (most dry)	12	0.02 to 0.75	0.10 to 1.0	0.5 to 1.0		
	SC/GC	all (most dry)	5	0.05 to 0.20	0.10 to 0.35	0.10 to 0.45		
Very Broad	all	all (most dry)	18	0.0 to 0.60	0.0 to 0.80	0.05 to 0.76		

*1 excludes possible outliers. Note: 6000 a) 3% Xdry clay core, thin to medium width ▲wet clay core, thin to medium width xdry SC/GC core, thin to medium width 5000 □ wet SC/GC core, thin to medium width #dry SM/GM core, thin to medium width • wet SM/GM core, thin to medium width Crest Settlement (mm) ▲Thick clay cores 4000 +Thick SC/GC cores Thick SM/GM cores ♦ Very broad cores 3000 2000 1000 200 100 150 250 300 Note: Base readings range from 0 to 0.5 Dam Height (m) years after embankment construction. b) 1400 -Gepatsch 1200 Djatiluhur 1000 Crest Settlement (mm) Srinagarind 800 Blowering Svartevann (left abut. Xdry clay core, thin to medium width Belich (A) ▲ wet clay core, thin to medium width Svartevann (main sect.) 600 $\ensuremath{\textbf{X}} \, \text{dry SC/GC core, thin to medium width}$ $\hfill\square$ wet SC/GC core, thin to medium width **(A)** +dry SM/GM core, thin to medium width 0.25% 400 • wet SM/GM core, thin to medium width ▲ Thick clay cores +Thick SC/GC cores -Thick SM/GM cores 200 ♦ Very broad cores poorly compacted rockfill 150 100 200 250 Note: Base readings range from 0 to 0.5

Figure 4.46: Post construction crest settlement at 3 years after end of construction, (a) all data, (b) data excluding Ataturk.

Dam Height (m)

years after embankment construction.



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Figure 4.47: Post construction crest settlement at 10 years after end of construction.

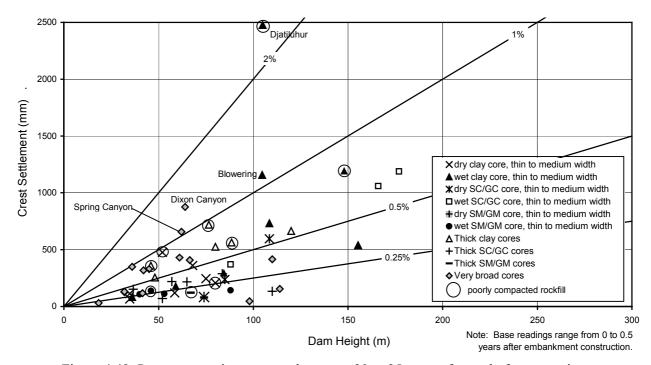


Figure 4.48: Post construction crest settlement at 20 to 25 years after end of construction.

(b) Mid to Upper Downstream Slope Region

For the mid to upper downstream slope region the data has been sorted based on material type of the downstream shoulder fill. The embankments with rockfill shoulders have been further divided into compaction rating of the rockfill. Embankments with dry placed and poorly compacted rockfills have been highlighted.

The height designated for each case study is the height from the SMP to foundation level.

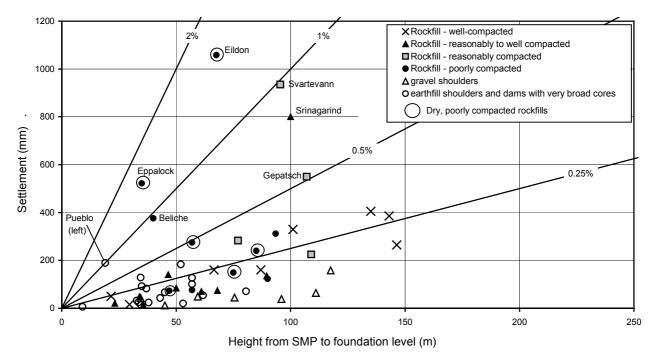


Figure 4.49: Post construction settlement of the downstream slope (mid to upper region) at 3 years after end of construction.

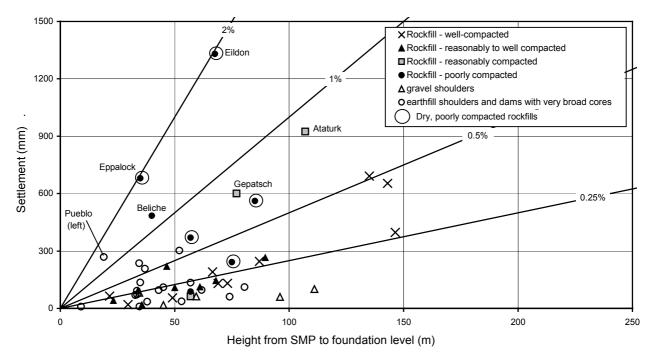


Figure 4.50: Post construction settlement of the downstream slope (mid to upper region) at 10 years after end of construction.

(c) Upper Upstream Slope and Upstream Crest Region

For the upper upstream slope and upstream crest region the data has been sorted based on material type of the upstream shoulder fill. The embankments with rockfill shoulders have been further divided into compaction rating of the rockfill. Embankments with dry, poorly compacted rockfills in the upstream shoulder have been highlighted.

The height designated for each case study is the height from the SMP to foundation level.

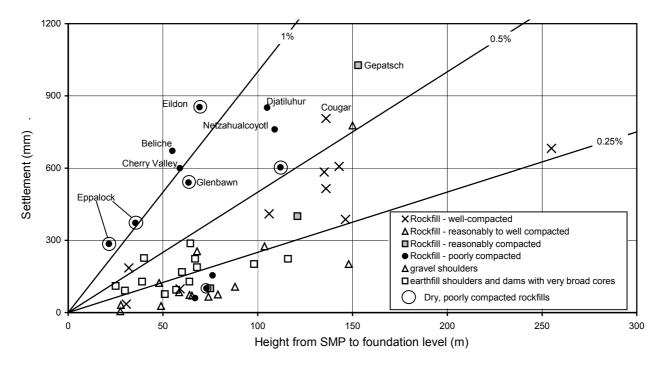


Figure 4.51: Post construction settlement of the upper upstream slope and upstream crest region at 3 years after end of construction.

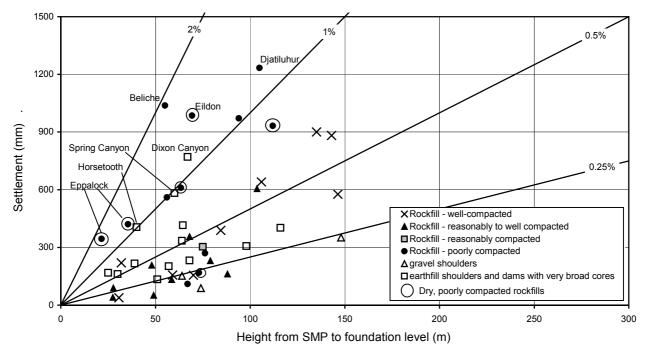


Figure 4.52: Post construction settlement of the upper upstream slope and upstream crest region at 10 years after end of construction.

Several zoned earthfill embankments with very broad cores where the foundation potentially has had an influence on the settlement have been included in the plots for the upstream and downstream slope. For these cases the height used is that from the SMP to bedrock foundation. Excluded though were cases where the foundation contributed significantly to the settlement such as at Fresno dam and the downstream slope of Horsetooth dam.

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From the figures and as summarised in Table 4.12 it can be seen that for the embankment shoulders:

- Settlements of up 1 to 2% are observed for poorly compacted rockfills, both in the upstream and downstream shoulder. Greater settlements are observed for the dry placed, poorly compacted rockfills.
- For reasonably compacted rockfills the range of settlement is quite broad, from 0.1% up to 1.0%, for a limited number of cases. At Svartevann and Gepatsch the greater settlements are attributable to large collapse type settlement of dry placed rockfills, and at Ataturk to collapse settlement of weathered rockfills.
- Much lower settlements, generally less than 0.5 to 0.7% at ten years after construction, are observed for well and reasonably to well compacted rockfills, and for embankments with compacted earthfill in the shoulder.
- Very low settlements (less than 0.25% at 10 years) are observed for embankments with gravel shoulders.

Possible outliers include Svartevann, Horsetooth (upstream slope), Dixon Canyon, Spring Canyon, Srinagarind and the downstream slope of the left abutment embankment at Pueblo dam. Several of these cases are discussed further in Section 6.0 and in Appendix B.

Material Type	Commontion	Do	wnstream Shoul	lder *1, *2	Upstream Shoulder *1, *2			
	Compaction Rating	Settlem	ent (% hgt from	SMP to fndn)	Settlement (% hgt from SMP to fndn)			
Type	Rating	No.	3 years	10 years	No.	3 years	10 years	
	well	11	0.05 to 0.35	0.05 to 0.55	12	0.10 to 0.60	0.10 to 0.70	
	reas to well	9	< 0.30	< 0.50	11	0 to 0.55	0.10 to 0.60	
Rockfill	reas	5	0.20 to 1.0	0.10 to 1.0	3	< 0.70	- * ⁴	
	poor	4	0.10 to ? * ⁴	0.15 to ? * ⁴	8	0.10 to 1.05	0.15 to 1.20	
	poor – dry *3	7	0.15 to 1.60	0.30 to 2.00	5	0.15 to 1.35	0.20 to 1.6	
Gravels	-	7	< 0.15	< 0.25	3	< 0.15	< 0.25	
Earthfills	-	20	0.0 to 0.40	0.0 to 0.70	14	0.05 to 0.60	0.10 to 0.70	

Table 4.12: Typical range of post construction settlement of the upstream and downstream shoulders.

Note: *1 Exclud

4.3.3 Post Construction Crest Settlement Versus Time

The post construction crest settlement versus time for the case studies are presented in Figure 4.53 to Figure 4.61 in the form of settlement as a percentage of the embankment height versus time (in years since end of construction) on a log scale. The case studies have been sorted based on core width, core material type and placement moisture content into the following figures:

- Figure 4.53 thin to medium core widths, dry placed clay cores.
- Figure 4.54 thin to medium core widths, wet placed clay cores.
- Figure 4.55 thin to medium core widths, dry placed clayey sand to clayey gravel cores.
- Figure 4.56 thin to medium core widths, wet placed clayey sand to clayey gravel cores.
- Figure 4.57 thin to medium core widths, silty sand to silty gravel cores both dry and wet placed.

^{*} Excludes possible outliers.

^{*2} Settlements quoted are a percentage of the height from the SMP to foundation level.

^{*&}lt;sup>3</sup> For the dry placed poorly compacted rockfills, settlements at Upper Yarra, Matahina and El Infiernillo are much less than for the other cases, for which settlements are close to or greater than 1% of the height.

^{*4} insufficient data.

- Figure 4.58 clay cores of thick core width.
- Figure 4.59 silty and clayey sand and gravel cores of thick core width.
- Figure 4.60 very broad cores where the foundation has limited influence on settlement.
- Figure 4.61 very broad cores where the foundation has or potentially has a significant influence on settlement. This figure is plotted with the total settlement in millimetres.

The foundation influence has been evaluated from IVM or foundation base plate records where available. However, this was not possible at Pueblo, Dixon Canyon and Spring Canyon dams because foundation settlements were not measured post construction. For Horsetooth and Steinaker dams the embankment only settlement from IVM records is included in Figure 4.60 and the combined embankment and foundation settlement from SMP records is included in Figure 4.61.

In most cases the base survey reading was within 0.5 years of end of embankment construction, but for a number of cases it was after this time. In several cases first filling had been completed prior to the start of monitoring, including but not limited to, Nillacootie, Peter Faust and Eildon dams.

The size of each figure has been set to the same margins for each plot area and the time interval standardised to cover from 0.1 to 100 years to allow for visual comparison between the figures. In most cases a standard vertical axis of 0 to 1.2% settlement has been used, but for several plots where settlements were larger a broader vertical axis has been adopted. If the case studies represented in a particular plot include an outlier/s (such as Ataturk in Figure 4.53a) a secondary plot excluding the outlier/s is presented as figure b.

Several other points to note from the figures are:

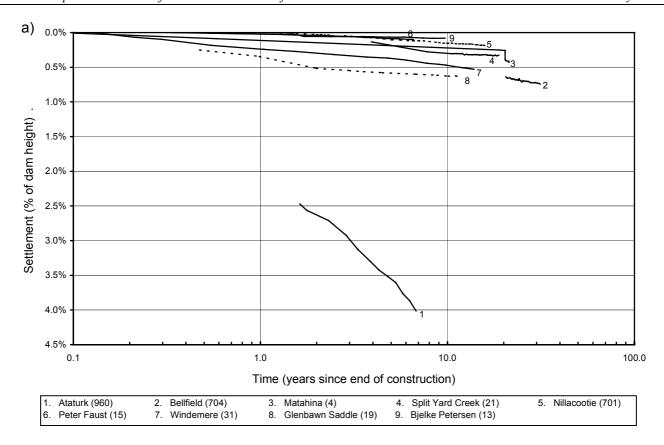
- Where the data does not extend to the y-axis it is because the base reading is or has assumed to be at the end of construction; i.e. zero time.
- The end of first filling has been indicated in the figures by an arrow for each case study. There are several reasons why first filling is not indicated for a number of cases studies:
 - First filling was not completed in the period of deformation shown (e.g. Ataturk and La Angostura dams).
 - First filling was completed prior to the base survey reading (e.g. Nillacootie, Peter Faust and Eildon dams).
 - For cases with the base reading at time = 0 years, first filling occurred in the time period up to the first reading point after the base survey (e.g. Talbingo and Bellfield dams).
 - The time of the end of first filling is not known (e.g. Tooma dam).

The assessment of "wet" or "dry" placement has been based on the average moisture content at placement in relation to Standard optimum moisture content and/or the pore water response during construction from piezometers installed in the core. The following criteria were used:

- "Wet" placement for cores placed from slightly dry (0.2 to 0.3% dry) to wet of Standard optimum moisture content and/or where the pore water pressure response indicated positive pore water pressure coefficients that exceeded about 0.1 to 0.2 at end of construction. As previously discussed (Section 4.1.1.4) earthfills placed wetter than about 0.5% dry of Standard optimum tend to develop positive pore water pressures during construction, although this is material type and stress level dependent.
- "Dry" placement for cores placed on average drier than about 0.5% dry of Standard optimum moisture content or where positive pore water pressures developed during construction indicated a pore pressure coefficient less than about 0.1 to 0.2.

For a number of cases no details were available on the moisture content at placement or the pore water pressure data during construction if piezometers had been installed. For these cases a judgement was made as to whether the core was likely to have been placed "dry" or "wet". Some of the judgements made were:

- For the Japanese dams Seto, Kurokawa, Shimokotori, Taisetsu and Kisenyama, all central core earth and rockfill dams with thin to medium core widths, limited information was available on the core materials. Most of these dams were constructed in the 1970's and were assumed to be predominantly clayey gravels and wet placed given material availability and typical procedures used in Japan for embankment construction at the time (Takahashi and Nakayama 1973; Shiraiwa and Takahashi 1985; Kanbayashi et al 1979).
- For embankments with silty sand or silty gravel cores no distinction was made with respect to "wet" or "dry" placement for the post construction deformation. Where no information was available it was assumed that construction pore water pressures, if developed, had been dissipated to small values a short time period after end of construction. Embankments with limited or no data on placement moisture condition or pore water pressure response included Mud Mountain, Round Butte, Cherry Valley and Mammoth Pool dams.
- For the Tennessee Valley Authority dams Nottely, Watuaga and South Holston, wet placement of the clay cores was assumed as implied by Leonard and Raine (1958) and Blee and Riegel (1951).
- For Googong, Spilt Yard Creek, Peter Faust, Bjelke Peterson, Corin and Benmore dams the classification is likely to be borderline between "wet" and "dry" as placement was or was likely to have been within about 0.5% of Standard optimum moisture content. All were classified as "dry" placed.
- Cairn Curran, a zoned earthfill embankment with very broad earthfill core, "dry" placement was assumed.



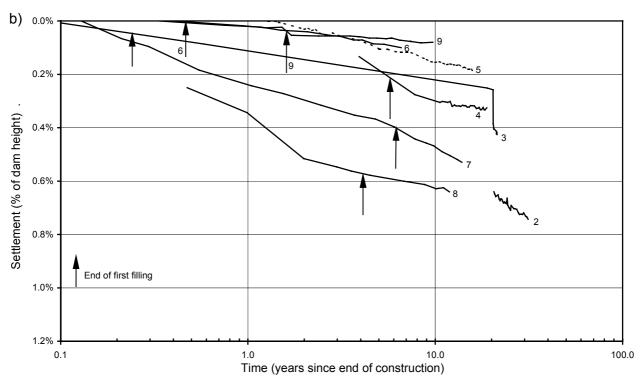
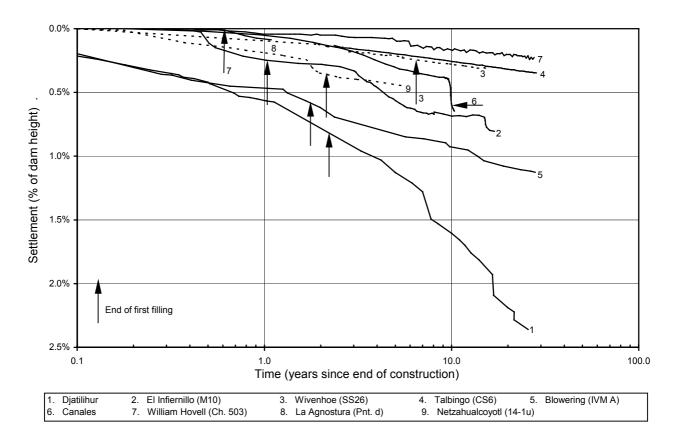


Figure 4.53: Crest settlement versus time for zoned embankments with thin to medium width central core zones of dry placed clayey earthfills; (a) all data, (b) data excluding Ataturk.



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Figure 4.54: Crest settlement versus time for zoned embankments with thin to medium width central core zones of wet placed clayey earthfills.

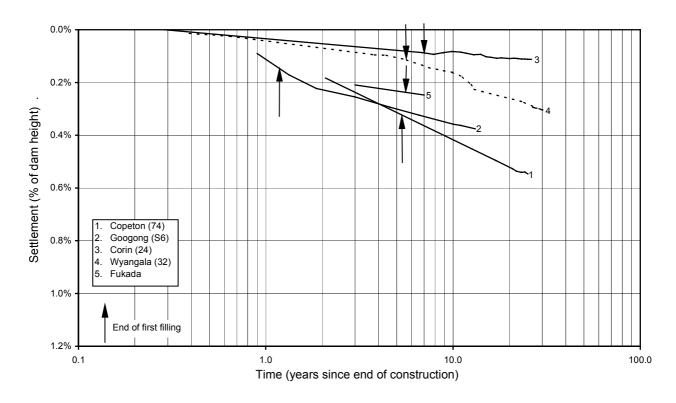
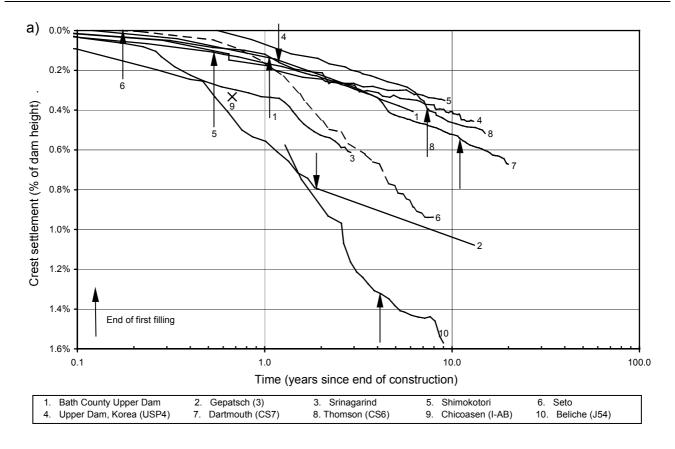


Figure 4.55: Crest settlement versus time for zoned embankments with thin to medium width central core zones of dry placed clayey sand to clayey gravel (SC to GC) earthfills.



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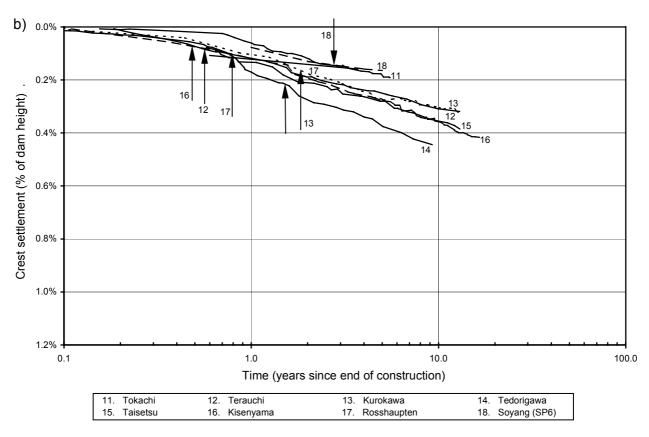
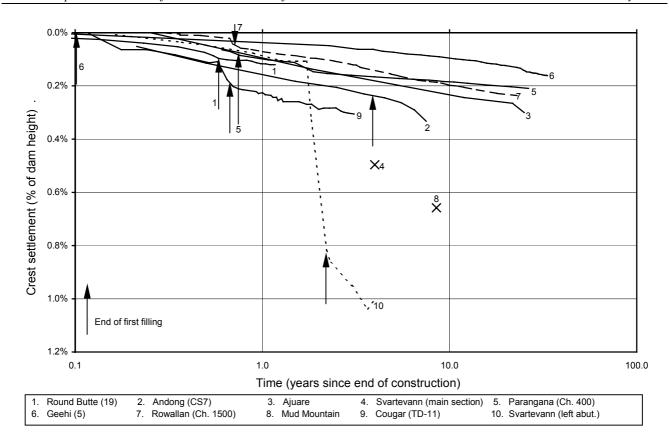


Figure 4.56: Crest settlement versus time for zoned embankments with thin to medium width central core zones of wet placed clayey sand to clayey gravel (SC to GC) earthfills.



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Figure 4.57: Crest settlement versus time for zoned embankments with thin to medium width central core zones of silty sand to silty gravel (SM to GM) earthfills.

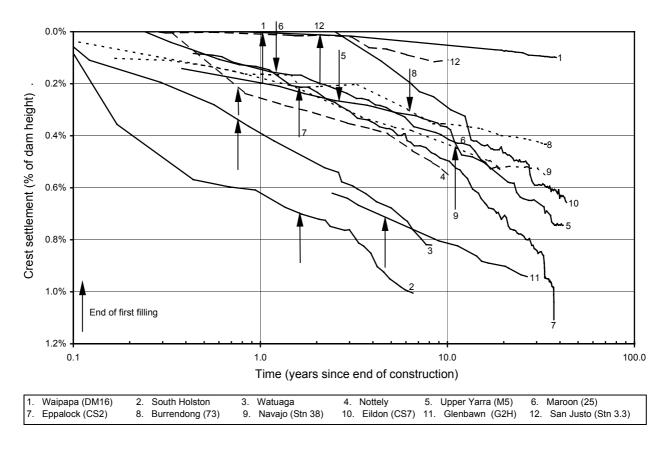
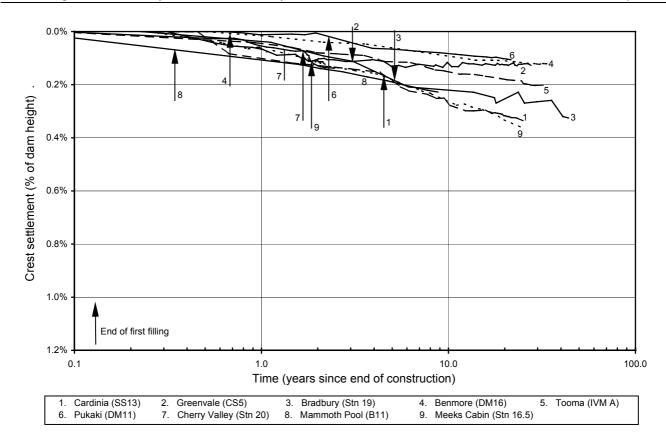
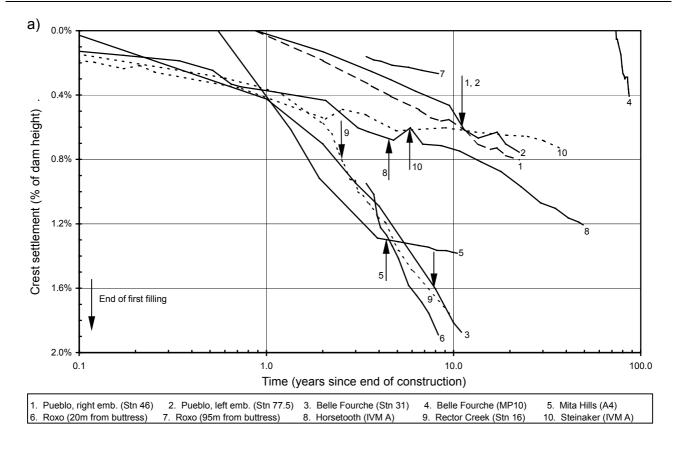


Figure 4.58: Crest settlement versus time for zoned embankments with central core zones of clayey earthfills of thick width (1 to 2.5H to 1V combined width).



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Figure 4.59: Crest settlement versus time for zoned embankments with central core zones of silty to clayey gravel and sand (SC, GC, SM, GM) earthfills of thick width (1 to 2.5H to 1V combined width).



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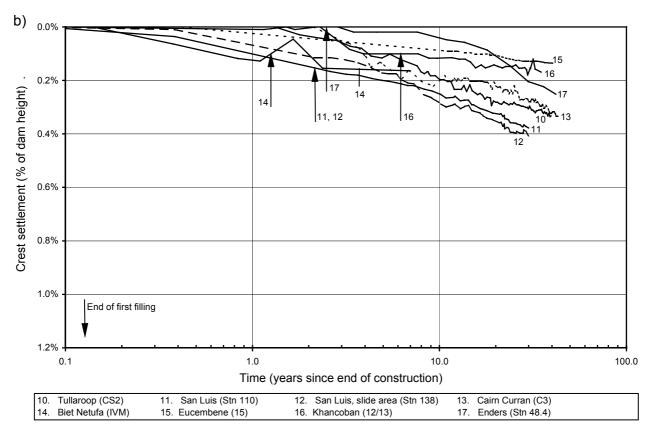
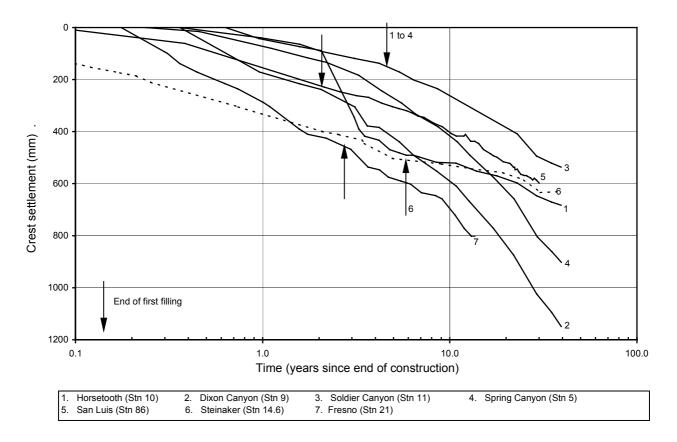


Figure 4.60: Crest settlement versus time for embankments with very broad core widths (> 2.5H to 1V combined width) and limited foundation influence.



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Figure 4.61: Crest settlement versus time for embankments with very broad core widths (> 2.5H to 1V combined width) and potentially significant foundation influence.

The typical trend of post construction crest settlement versus log time is for a near linear relationship for most case studies, particularly after the end of first filling. Although, a gradual increase in the crest settlement rate (rate per log cycle of time) with time, or in-fact a decrease in rate with time is not unusual.

Before getting into a detailed discussion on the post construction crest settlement behaviour, the data on the long-term or post first filling crest settlement rate (rate per log cycle of time) is presented. Figure 4.62 presents the long-term crest settlement rate versus embankment height for zoned earthfill and earth-rockfill embankments with cores of thin to thick widths, and Figure 4.63 for earthfill and zoned embankments with very broad core widths. The data has been sorted based on reservoir operation. For the embankments of thin to thick core width the data has also been sorted based on core material type and those embankments with poorly compacted upstream rockfill zones have been highlighted. Table 4.13 presents a summary of the typical range of crest settlement rate for various sub-groupings of the case study data.

The assessment of reservoir operation post first filling is:

- "Fluctuating" reservoir where the reservoir is subject to a seasonal (usually annual) or regular (more than once per year) drawdown that is typically greater than about 0.1 to 0.15 times the height of the embankment at its maximum section.
- "Steady" or "slow" reservoir operation for embankments where the reservoir remains steady over time, is subject to fluctuations that occur slowly over time (i.e. slow drawdown over several years) or where the seasonal or regular fluctuation is less than 0.1 to 0.15 times the height of the embankment at its maximum section.

The crest settlement rate for each case study has been determined from the settlement versus log time plots in the above figures from the portion of the curve after the end of first filling. Where the settlement rate increases with time, such as for Enders dam in Figure 4.60b, the longer term rate has been used. For case studies where the settlement post first filling includes short periods of rapid acceleration, such as at Eppalock, Eildon and Djatiluhur dams, the representative rate for these embankments excludes the periods of rapid acceleration.

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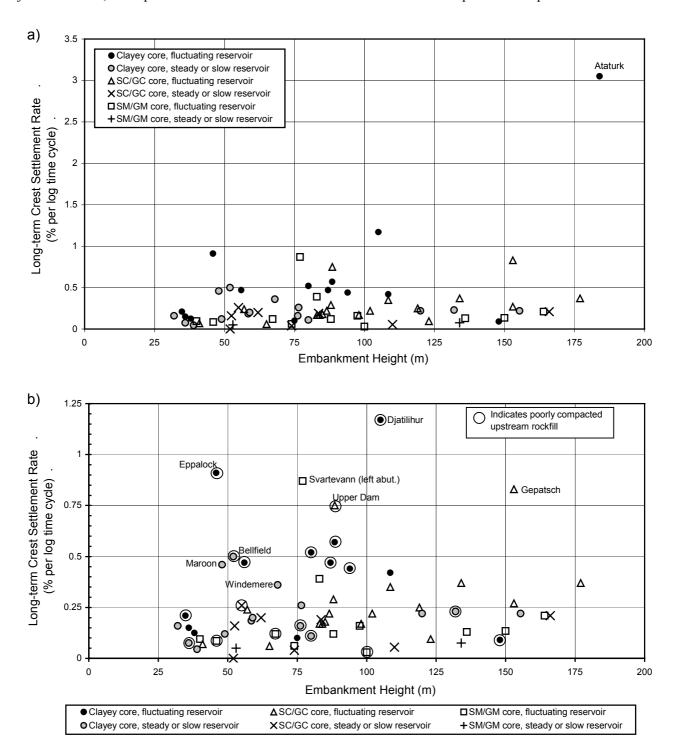
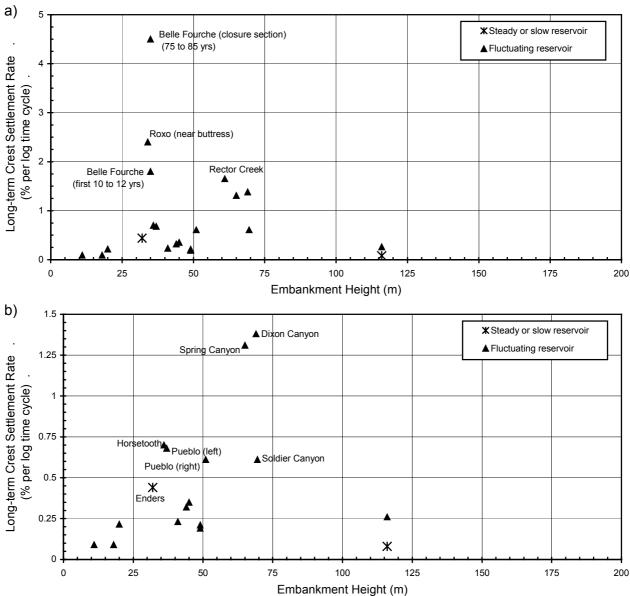


Figure 4.62: Long-term post construction crest settlement rate for zoned earthfill and earth-rockfill embankments of thin to thick core widths, (a) all data, and (b) data excluding Ataturk.



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Figure 4.63: Long-term post construction crest settlement rate for embankments of very broad core width, (a) all data, and (b) data excluding Belle Fourche, Roxo and Rector Creek.

Table 4.13: Summary of long-term crest settlement rates (% per log cycle of time).

Core Width	Core Classification	Reservoir Operation	No. Cases	Long-term Settlement Rate (% per log cycle of time)	Comments
		Fluctuating	14	0.09 to 0.57	Higher rates at Djatiluhur (1.17%) and Eppalock (0.91%).
	CL/CH	Steady or slow	15	0.04 to 0.50 (most < 0.26)	Very high at Ataturk (3.1%). Higher rates at Maroon (0.46%), Windemere (0.36%) and Bellfield (0.50%).
Thin to thick	SC/GC	Fluctuating	18	0.06 to 0.37	Higher rates at Upper Dam (0.75%) and Gepatsch (0.83%).
		Steady or slow	8	0 to 0.26	
	SM/GM	Fluctuating	12	0.03 to 0.21	Higher rates at Svartevann (0.87%) and Andong (0.39%).
		Steady or slow	2	< 0.10	Only two cases, Round Butte & Parangana
Very broad	All types (most CL)	Fluctuating	17	0.07 to 0.70 (most < 0.35)	Very high rates (> 1.8%) at Belle Fourche & Roxo (near buttress). High rates at Rector Creek (1.65%), Dixon Canyon (1.38%) and Spring Canyon (1.31%).
		Steady or slow	2	0.08 and 0.44	Only two cases, Eucembene (0.08%) and Enders (0.44%).

As a whole the total settlement, settlement versus time and settlement rate plots provide a comprehensive summary of the post construction crest settlement behaviour for earthfill and earth-rockfill embankments. The data allows for comparisons of magnitude and rate based on core width, core material type and moisture content at placement as well as consideration of reservoir operation post first filling and the use of poorly compacted rockfills in the upstream shoulder. Table 4.11 and Table 4.13 provide typical ranges of total crest settlement and long-term crest settlement rate sorted for most of these factors.

Clear and possible outliers to the general trend of post construction crest settlement are generally evident in at least two of the plot types, usually the settlement versus time plot and then either or both of the total settlement and settlement rate plots. The settlement versus time plots are the most comprehensive for assessment of potential outliers. In addition to highlighting large total settlement or high settlement rates other trends that are potentially indicative of "abnormal" deformation behaviour are evident, such as acceleration in settlement over short time periods. Embankments indicating this type of behaviour include Djatiluhur, Canales, El Infiernillo, Beliche, Eppalock, Eildon, Upper Yarra, Svartevann, Matahina and possibly Blowering and Wyangala dams. A number of these case studies are discussed in Section 6.0. Often, but not always, these accelerations in settlement rate are associated with drawdown (refer to point d below).

It is important to note that use of the term "abnormal" in relation to the deformation behaviour of an embankment does not necessarily equate with potential instability. In nearly all cases identified as "abnormal" or potentially "abnormal" the overall stability of the embankment is not in question.

The factors affecting the post construction settlement of the mid to downstream region of the crest are:

(a) Influence of core material type and core width on the post construction crest settlement

Core material type has an influence on the post construction crest settlement. In general:

- Earthfill cores of silty sands to silty gravels typically show lower total crest settlements (Table 4.11) and lower long-term settlement rates (Table 4.13).
- Clay earthfill cores, on average, tend to have high long-term crest settlement rates (Figure 4.62 and Table 4.13) with clayey sand to clayey gravel cores showing intermediate rates.

Core width also influences the post construction crest settlement behaviour, but its influence is only evident for the crest settlement during and shortly after first filling. During first filling, crest settlements for most of the case studies with very broad earthfill cores (Figure 4.60 and Figure 4.61) and thick earthfill cores (Figure 4.58 and Figure 4.59) are limited, while those for thin to medium width cores are larger. This is likely to be due to several factors:

- The direction of the water load on first filling acting on the upstream face of the core and its distance from the dam axis.
- For embankments with thick cores, the core is generally not reliant on the shoulders zones for support and
 consequently not significantly influenced if collapse settlement on first filling occurs in the upstream
 shoulder. Conversely, embankments of thin to medium core width are more reliant on the shoulders for
 support and therefore can be significantly influenced by collapse settlements in the upstream shoulder on
 first filling.

The core width appears to have no recognisable influence on the long-term post first filling crest settlement rate (Table 4.13, Figure 4.62 and Figure 4.63). For embankments with very broad core width the long-term crest settlement rates (Figure 4.63) are highly variable and are discussed further in (f) below.

(b) Influence of compaction moisture content on the post construction crest settlement

For embankments with cores of thin to medium width it is evident that, after excluding the case studies likely affected by collapse settlement of the upstream rockfill, the range of crest settlement at any given time for cores placed wet are comparable to those for dry placed cores. For example, for the cases where monitoring started close to zero time the crest settlement at 10 years after end of construction are in the range:

- 0.2 to 0.65% for dry placed clay earthfills (Figure 4.53).
- 0.15 to 0.7% for wet placed clay earthfills (Figure 4.54).
- 0.1 to 0.4% for dry placed clayey sand to clayey gravel earthfills (Figure 4.55).
- 0.2 to 0.5% for wet placed clayey sand to clayey gravel earthfills (Figure 4.56).

This is not to say that compaction moisture content has a negligible influence on the post construction settlement behaviour, but that other factors have a greater influence and over-shadow that of compaction moisture content such as reservoir operation and collapse settlement of the upstream rockfill for wet placed cores.

The effect of compaction moisture content on the post construction internal vertical strain is clearly evident at Talbingo dam. In the lower region of the wet placed core at Talbingo dam reductions in pore water pressure post first filling were very large, up to 600 to 1100 kPa, and post construction vertical strains were up to 2.5 times greater than in the mid to upper region where the core was placed drier and reductions in pore water pressures were consequently much smaller (Figure 4.43 in Section 4.3.1). This reduction in pore water pressure in the core at Talbingo dam was much greater than that at Blowering, Thomson and Dartmouth dams (dams all greater than 100 m height) yet the post construction crest settlement at Talbingo is less than for these other embankments.

The limited influence of compaction moisture content on post construction crest settlement can also be attributable to the usually small reduction in pore water pressure from end of construction to close to equilibrium conditions for wet placed clay cores. In some cases, such as at Talbingo dam, where the reduction in pore water pressure is large (more than 200 to 500 kPa) it is usually confined to small regions of the core. Therefore, the net influence of consolidation due to dissipation of pore water pressures on the overall crest settlement will be relatively small.

Table 4.11 does indicate that a higher range of crest settlement is observed for wet placed clay and clayey sand to gravel cores of thin to medium width. However, the range of settlement for most of these case studies with wet placed cores is similar in magnitude to that of dry placed cores, as shown in Figure 4.53 to Figure 4.56. The reason for the higher range of settlement is that the data for wet placed cores does includes several case studies where the crest settlement was thought to be strongly influenced by collapse compression in the upstream rockfill.

(c) Influence of rockfill placement procedures for zoned embankments with rockfill in the upstream shoulder

The data for zoned earth and rockfill embankments shows that case studies with poorly compacted rockfill in the upstream shoulder are over-represented at the higher end of the total settlement plots (Figure 4.46 to Figure 4.48) and the higher end of long-term settlement rate plot (Figure 4.62). A more correct conclusion from the data would be that embankments where the rockfill in the upstream shoulder is susceptible to large magnitude deformations due to collapse compression on wetting represent a very large portion of the data at the higher end of these plots.

Earlier findings (Section 4.2.2) indicate that the magnitude of deformation on collapse compression is related to a number of factors other than degree of compaction, including placement moisture content, rock substance

unconfined compressive strength and its reduction on saturation, stress level and particle size distribution. Assuming the magnitude of deformation of the upstream rockfill due to collapse compression is large on first filling, the shear strength properties of the core strongly influence the magnitude of crest settlement, with wet placed cores of low undrained strength shown to be susceptible to large crest settlements on first filling. The post construction settlement data tends to confirm this, although large settlements are not just confined to wet placed cores, they are also observed for dry placed cores. This is discussed further in Section 6.3.

From Figure 4.46, the plot of total settlement at 3 years after construction, crest settlements greater than 0.35 to 0.4% of the embankment height occur in embankments where the magnitude of settlement of the upstream rockfill due to collapse settlement on wetting was significant, including:

- A number of embankments with poorly compacted rockfill in the upstream shoulder.
- Embankments with dry placed and reasonably compacted (compacted in 2 m lifts) rockfill in the upstream shoulder (Gepatsch and Svartevann dams).
- Embankments with wetted and reasonably compacted weathered rockfill in the upstream shoulder (Ataturk and Beliche dams).
- Blowering dam, where the well-compacted rockfill was susceptible to large loss in strength on wetting.

The greater total settlement in these embankments is also reflected in the plots at 10 years and 20 to 25 years after construction.

The long-term crest settlement rate data (Figure 4.62) shows that for almost all cases where the settlement rate is greater than about 0.4% per log cycle of time the upstream rockfill (and usually the downstream rockfill as well) was poorly compacted and/or susceptible to large magnitude deformations due to collapse compression. This is an important observation because the influence of rockfills susceptible to collapse compression on the embankment deformation behaviour is generally only related to the embankment performance during and shortly after first filling. But the high long-term settlement rates for these embankments suggests that they are more susceptible to softening effects and degradation long-term as a consequence of factors including poor lateral support of the core, lateral spreading, internal cracking and softening in undrained strength. This is discussed further in Section 6.3 referencing the deformation behaviour from selected case studies.

(d) Influence of reservoir operation on the long-term crest settlement rate

The reservoir operation has a significant influence on the long-term post first filling crest settlement rate for zoned embankments as shown in Figure 4.62 and summarised in Table 4.13. Most of the periods of rapid acceleration of the crest settlement after first filling are associated with drawdown events. Exceptions to this are the earthquake influence at El Infiernillo and Matahina dams.

The difference in crest settlement behaviour due to reservoir operation is highlighted in Figure 4.64 and Figure 4.65. Both figures are of central core earth and rockfill embankments with clay cores of thin to thick width, Figure 4.64 for embankments where the reservoir is relatively steady and Figure 4.65 for embankments where the reservoir undergoes seasonal (often annual) drawdown. For the seasonal drawdown case studies the rockfill placement consisted of:

- Placed dry in 1.5 to 2 m layers and spread with dozer tracking for Eildon and Eppalock, and possibly Upper Yarra.
- Wetted and well-compacted for Blowering dam, but the rockfill is susceptible to large strength loss on wetting.
- Reasonably to well compacted for William Hovell.

For Eildon, Eppalock, Upper Yarra and Blowering dams the periods of acceleration of the crest settlement post construction are associated with large drawdown events. Other case studies where acceleration in crest settlement occurs on drawdown include Djatiluhur, Beliche and Wyangala dams. These cases are discussed further in Section 6.3 and in Appendix B.

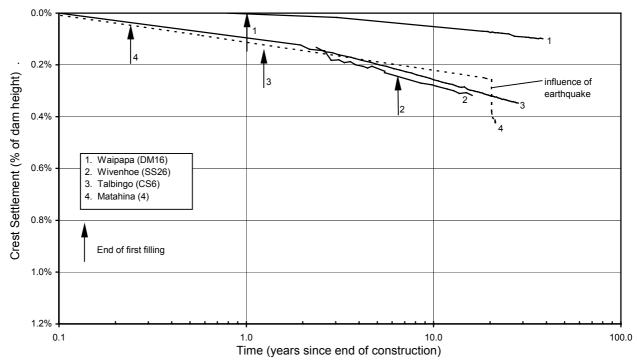


Figure 4.64: Crest settlement versus time for central core earth and rockfill embankments with central core zones of clayey earthfills and steady post first filling reservoir operation.

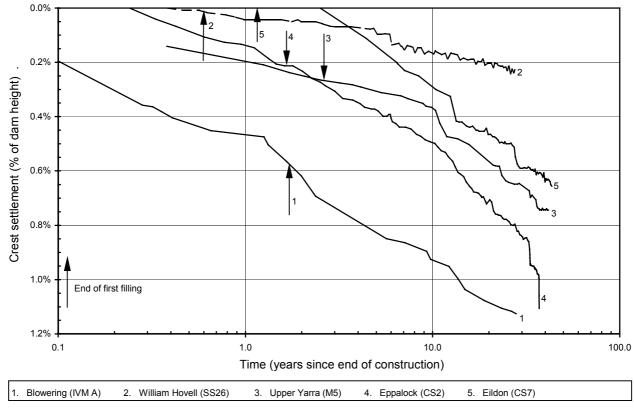


Figure 4.65: Crest settlement versus time for central core earth and rockfill embankments with thin to thick central core zones of clayey earthfills subjected to seasonal reservoir fluctuation.

(e) Crest settlement during first filling

Higher rates of crest settlement (per log cycle of time) during first filling are generally observed for zoned embankments of thin to medium core width, but only some of these embankments show this behaviour (Figure 4.53 to Figure 4.57). Dams with higher rates of settlement during first filling include:

- Split Yard Creek and Glenbawn Saddle dams, both of dry placed clay cores of thin to medium width.
- Blowering, Canales and Netzahualcoyotl dams, all wet placed clay cores of thin to medium width.
- Googong dam, a dry placed clayey sand core of medium width.
- Thomson, Beliche, Gepatsch and Dartmouth dams, all wet placed clayey sand to clayey gravel cores of thin to medium width.
- Svartevann, Cougar and Rowallan dams, all wet placed silty sand to silty gravel cores of thin to medium width
- Nottely and South Holston dams, both clay cores of thick width and possibly wet placed.
- Mita Hills and Horsetooth dams, embankments with very broad earthfill zones.

Apart from Mita Hills and Horsetooth, all embankments are central core earth and rockfill embankments.

It is suspected that the mechanism of deformation for most of the case studies includes collapse settlement of the upstream rockfill on first filling and resultant plastic deformation with potential development of localised shearing within the core. This mechanism was discussed in Section 4.2.2 with the aid of finite difference modelling and is discussed in more detail for selected case studies in Section 6.3 and Appendix B.

(f) Influence of very broad core width

For earthfill embankments or zoned embankments with very broad cores notable aspects of the post construction crest settlement behaviour are:

- For a number of case studies the settlement versus time plots (Figure 4.60 and Figure 4.61) show an increasing rate of settlement after first filling. This trend is clearly evident for Enders dam and the Horsetooth Reservoir dams (Horsetooth, Dixon Canyon, Spring Canyon and Soldier Canyon), and to a lesser degree for San Luis and several other embankments.
- The long-term settlement rate of these embankments shows a high degree of variability (Figure 4.63), ranging from 0.07% to 4.5% within the closure section at Belle Fourche dam many years after construction.

With respect to the long-term settlement rate, the very high rates (> 1.6% per log time cycle) at Belle Fourche (within the closure section), Rector Creek, Mita Hills and Roxo (adjacent to the buttress) are considered "abnormally" high. At Belle Fourche (closure section) it is notable that the settlement rate at 75 to 85 years after construction is almost 3 times what is was at 10 to 12 years after construction. The high rates at Dixon Canyon (1.38%) and Spring Canyon (1.31%) dams are also considered to be "abnormally" high. For these two dams there is a query over the influence of the foundation on the long-term crest settlement, but from review of the data records (refer Section 3.3 of Appendix B) this influence is considered to be limited and most of the long-term crest settlement is attributable to deformation within the earthfill.

The remaining embankments are considered to represent "normal" long-term settlement rates with values ranging from 0.07% to 0.7% per log time cycle. Embankment height appears to have little influence on the settlement rate as indicated by the low rates for the two largest embankments, Eucembene and San Luis (main section) dams, compared to some of the other case studies of much lower embankment height.

For these embankments with very broad earthfill zones the development of the phreatic surface and the influence of reservoir fluctuation on piezometric levels within the embankment are considered to influence the post construction deformation behaviour. For most of the case studies the earthfill zones are dominantly clayey

and of low permeability, and an important consideration for the deformation behaviour of these embankments is the effect of the gradual development of the phreatic surface and associated changes in effective stress conditions within the embankment. For a number of the case studies (of dominantly clayey earthfills) piezometric records indicate the phreatic surface took tens of years to reach steady levels in the central core region of the broad earthfill core, whilst for more permeable earthfills steady levels were reached within several years. Some examples of this are:

- At Belle Fourche dam (35 m high), constructed of medium plasticity clays placed on the dry side of Standard optimum, borehole "well" records indicate it took more than 10 to 15 years to establish steady levels in the upstream earthfill zone and more than 20 years in the central embankment region.
- At Fresno dam (24 m high), constructed of low plasticity sandy clays placed at an average of 0.2% dry of Standard optimum, pore pressures reached close to equilibrium conditions more than 4 years but less than 15 years after end of construction (Daehn 1955).
- At Steinaker dam (49 m high), central earthfill zone of medium plasticity clays placed on the dry side of Standard optimum, piezometer records indicate it took 15 to 30 years to establish steady levels in the central embankment region.
- At San Luis dam (115 m high), central earthfill zone of low to medium plasticity clays placed on the dry side of Standard optimum, piezometer records indicate it took 15 to 30 years to establish steady levels in the central embankment region.
- At the Horsetooth Reservoir embankments (Horsetooth, Dixon Canyon, Spring Canyon and Soldier Canyon dams of 48 to 74 m height), constructed of predominantly sandy clays placed on the dry side of Standard optimum, USBR (1997) referred to the piezometer levels still rising more than 50 years after end of construction.
- At Khancoban dam, earthfill zone of silty sands, reasonably steady levels were established within 2 to 5 years after end of construction.

For those embankments with cores of low permeability, reservoir fluctuation would not be expected to greatly influence the crest settlement rate due to the likely limited effect it would have on pore water pressures in the central region of the embankment. However, for more permeable earthfills the reservoir operation may have a more substantial influence on the post construction crest deformation behaviour. At Enders dams the increased rate in settlement 8 to 10 years after the end of construction (Figure 4.60b) is coincident with the increased magnitude of fluctuation of the reservoir. In the first 8 to 10 years of operation annual fluctuations were limited to less than about 3 m, but after this period increased to 5 to 8 m (Figure 4.66). Piezometric records indicate that pore pressures within the central embankment region also fluctuated with reservoir level but at a reduced amplitude.

In comparison, the post construction settlement at the main section at San Luis dam (Figure 4.60b) showed no significant change in rate of settlement beyond about 8 years when the seasonal reservoir fluctuation increased substantially (Figure 4.67). Piezometers in the central region of the main earthfill zone (of mostly low to medium plasticity sandy clays) showed a negligible to very limited response to drawdown.

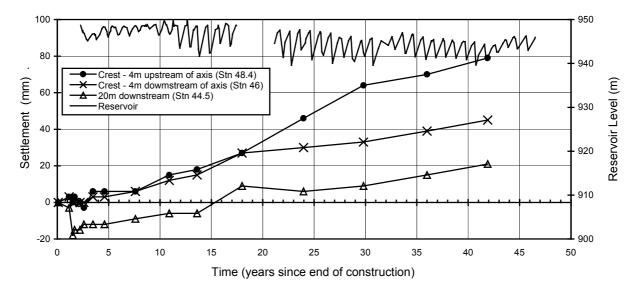


Figure 4.66: Post construction settlement and reservoir operation at Enders dam (data courtesy of USBR).

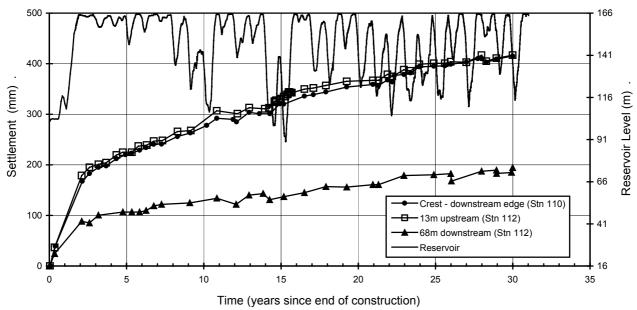


Figure 4.67: Post construction settlement at main section and reservoir operation at San Luis dam (data courtesy of USBR).

4.3.4 Post Construction Horizontal Displacement of the Crest Normal to the Dam Axis.

The post construction horizontal displacements of the crest are presented in Figure 4.68 to Figure 4.76 in the form of total displacement in millimetres versus time (in years since end of construction) on a log scale. The case studies have been sorted based on core width, core material type and placement moisture content into the following figures:

- Figure 4.68 thin to medium core widths, dry placed clay cores.
- Figure 4.69 thin to medium core widths, wet placed clay cores.
- Figure 4.70 thin to medium core widths, dry placed clayey sand to clayey gravel cores.
- Figure 4.71 thin to medium core widths, wet placed clayey sand to clayey gravel cores.
- Figure 4.72 thin to medium core widths, silty sand to silty gravel cores both dry and wet placed.
- Figure 4.73 clay cores of thick core width.
- Figure 4.74 silty and clayey sand and gravel cores of thick core width.

- Figure 4.75 very broad cores where the foundation has limited influence on deformation.
- Figure 4.76 very broad cores where the foundation has or potentially has a significant influence on settlement.

For most case studies the displacement includes the period of first filling, or at least a substantial portion of it. For several embankments though first filling had been (or was virtually) completed prior to the start of monitoring and includes, but is not limited to, Nillacootie, Eildon, Belle Fourche and Peter Faust dams.

For several of the embankments with very broad earthfill cores the distinction of foundation influence could not be readily evaluated, as no data on the foundation deformation was available. These cases include Pueblo, Dixon Canyon and Spring Canyon dams.

The size of each figure has been set to the same margins for each plot area and the time interval standardised to cover from 0.1 to 100 years to allow for visual comparison between the figures. In most cases a standard vertical axis of 100 mm upstream (-100 mm) to 300 mm downstream (+300 mm) displacement has been used. For several plots where larger displacements were recorded a broader vertical axis has been adopted. Other points to note about the figures are explained in the preamble for the crest settlement versus time plots (Section 4.3.3).

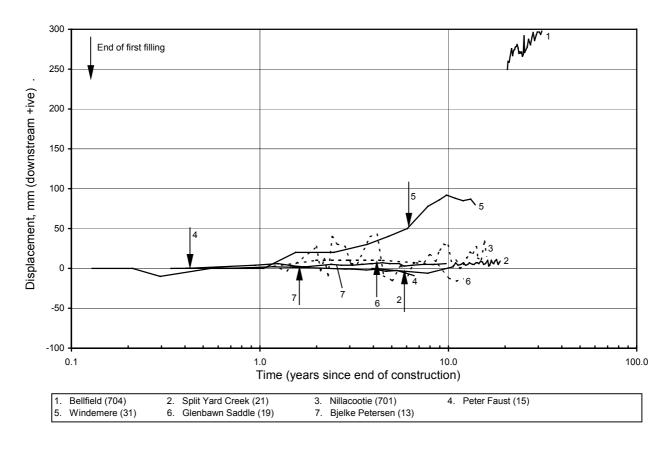
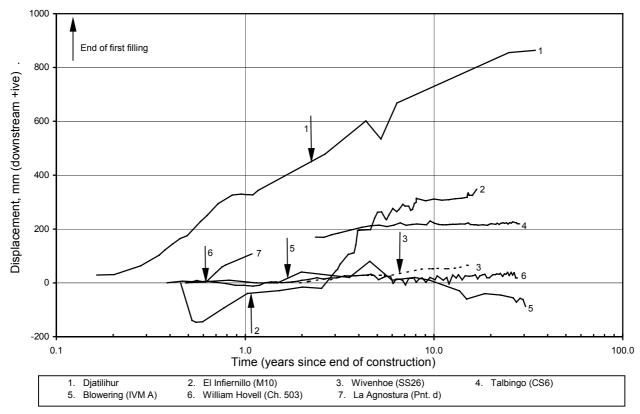


Figure 4.68: Crest displacement versus time for zoned embankments with thin to medium width central core zones of dry placed clayey earthfills.



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Figure 4.69: Crest displacement versus time for zoned embankments with thin to medium width central core zones of wet placed clayey earthfills.

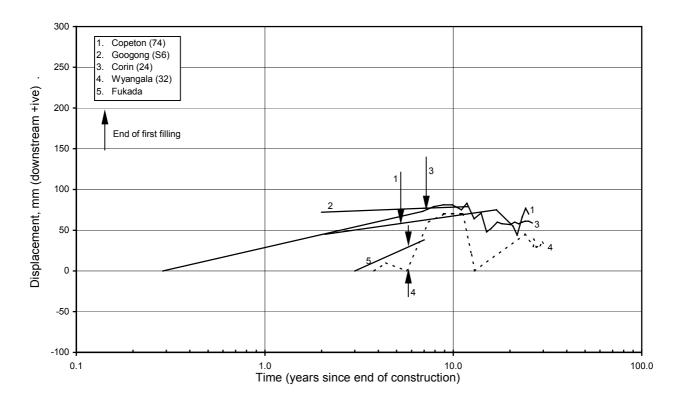
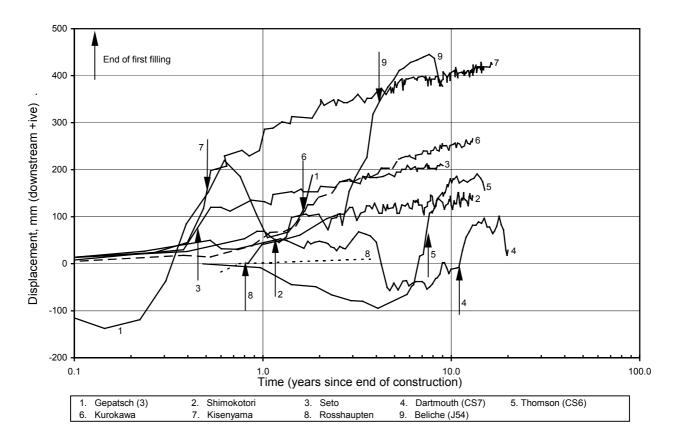


Figure 4.70: Crest displacement versus time for zoned embankments with thin to medium width central core zones of dry placed clayey sand and clayey gravel (SC/GC) earthfills.



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Figure 4.71: Crest displacement versus time for zoned embankments with thin to medium width central core zones of wet placed clayey sand and clayey gravel (SC/GC) earthfills.

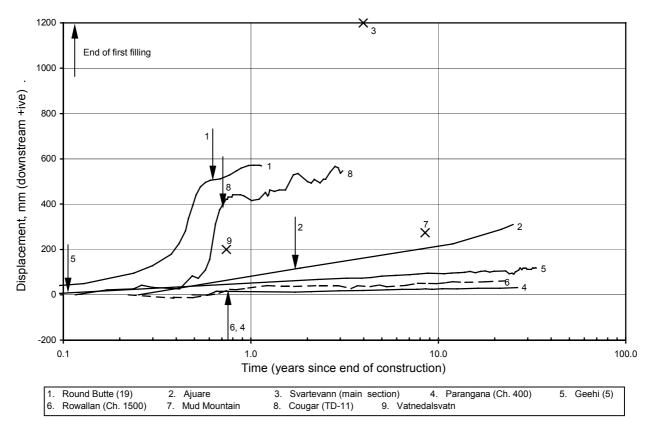
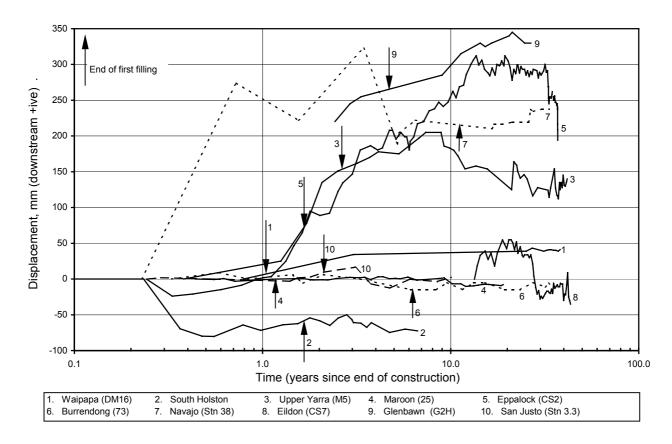


Figure 4.72: Crest displacement versus time for zoned embankments with thin to medium width central core zones of silty sand and silty gravel (SM/GM) earthfills.



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Figure 4.73: Crest displacement versus time for zoned embankments with central core zones of clayey earthfills of thick width (1 to 2.5H to 1V combined width).

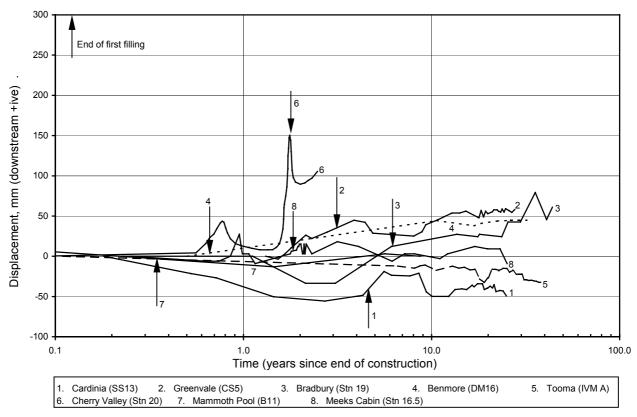
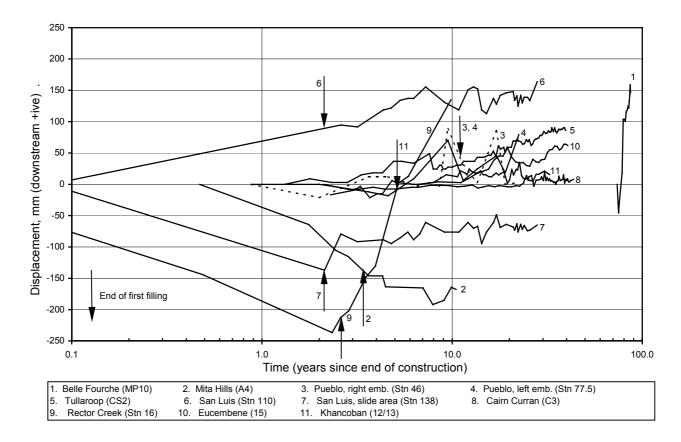


Figure 4.74: Crest displacement versus time for zoned embankments with central core zones of silty to clayey sand and gravel earthfills of thick width (1 to 2.5H to 1V combined width).



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Figure 4.75: Crest displacement versus time for embankments with very broad core zones (> 2.5H to 1V width) and with limited foundation influence on the embankment deformation.

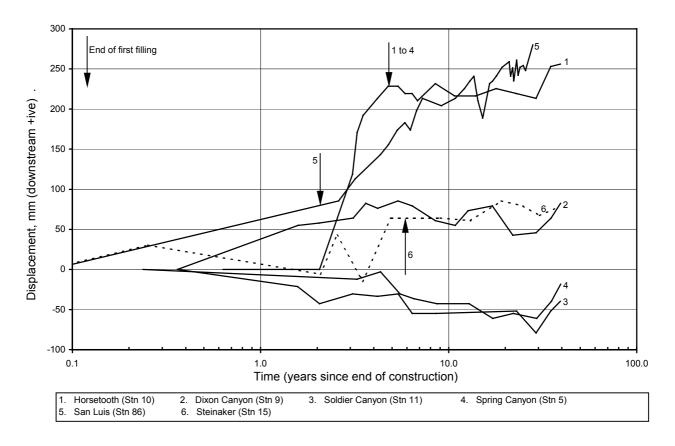


Figure 4.76: Crest displacement versus time for embankments with very broad core zones (> 2.5H to 1V width) and with potentially significant foundation influence on the embankment deformation.

A typical pattern of crest displacement behaviour is difficult to define, however, for a large number of the case studies:

- A large portion of the crest displacement occurs during first filling, in most cases the displacement being downstream.
- Post first filling, most cases show a slight to steady downstream trend, approaching near zero displacement rates in the long-term (after 10 to 20 years).
- Fluctuations about the general trend occur due to reservoir fluctuation, with upstream displacement on drawdown and downstream displacement on first filling.

Plots of crest displacement versus reservoir level presented for a number of embankments in the literature highlight this general trend.

(a) Displacement during first filling

The lateral displacement at end of first filling was discussed in Section 4.2.3. Plots of the displacement at end of first filling versus embankment height were presented for the crest (Figure 4.38) as well as the upstream and downstream slopes. Typical ranges of the crest displacement at end of first filling are given in Table 4.9 sorted on core width, core properties (material type and moisture content at placement) and the material type and placement method of the downstream shoulder.

For most cases the crest displacement is dominantly downstream reaching a maximum at the end of first filling.

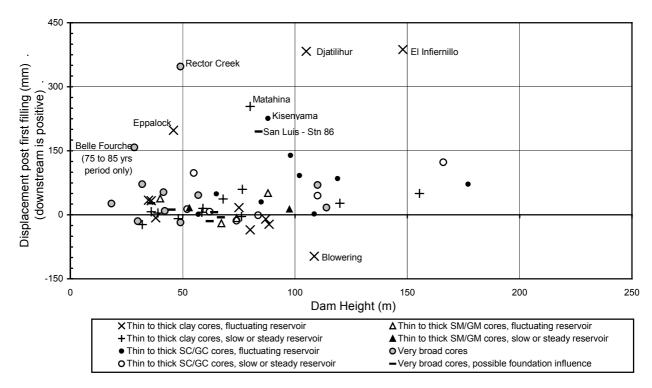
As discussed in Section 4.2.2, central core earth and rockfill embankments with wet placed cores of low undrained strength and upstream rockfill susceptible to collapse settlement on saturation had the potential to deform upstream during the early stages of reservoir filling followed by downstream displacement as the reservoir approached full supply level. Nobari and Duncan (1972b) proposed this concept to explain the crest displacement behaviour at El Infiernillo dam. From the crest displacement versus time plots it is apparent that this behaviour also occurred at Gepatsch and Thomson dams, and possibly also at Blowering and Dartmouth dams; all of which are central core earth and rockfill embankments with thin to medium width cores of wet placed clays, clayey sands or clayey gravels.

(b) Displacement Post First Filling

Post first filling the general trend for crest displacement, as indicated in the figures, is for a steady or gradual downstream displacement with time approaching low or near zero displacement rates long-term.

Figure 4.77 presents the lateral displacement post first filling versus embankment height. The displacement data is for at least five years after first filling (up to 30 years or more for some cases). Where possible the long-term data point has been taken for the reservoir at close to full supply level. For Belle Fourche the data is only for monitoring from the period 75 to 85 years post construction from the closure section, and the actual post first filling displacement is likely to be much greater than shown for this region of the embankment. For Matahina dam the post first filling crest displacement is prior to the earthquake in 1987.

The data shows that for most cases the displacement post first filling is generally in the range 35 mm upstream (-35 mm) to 100 to 150 mm downstream, indicating that post first filling crest displacements are generally small and are not dependent on dam height. Displacements outside this range may be indicative of "abnormal" deformation behaviour. Several of these case studies are discussed further in Section 6.0.



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Figure 4.77: Crest displacement normal to dam axis post first filling

4.3.5 Post Construction Deformation of the Mid to Upper Downstream Slope

Post construction deformation data for the mid to upper region of the downstream slope has been presented in previous sections, including:

- Lateral displacement normal to the dam axis at the end of first filling in Section 4.2.3. Figure 4.39 presents the lateral displacement at end of first filling versus embankment height and Figure 4.42 versus the downstream slope angle.
- Total settlement plots at 3 years (Figure 4.49) and 10 years (Figure 4.50) after the end of embankment construction in Section 4.3.2. Table 4.12 summarises the typical range of settlement sorted based on material type and placement method of the downstream shoulder material. The data was plotted against the height from the SMP to foundation level.

Additional data on the post construction deformation of the mid to upper downstream slope presented below includes:

- Figure 4.78 Long-term settlement rate (% per log cycle of time) versus the height from the SMP to foundation level.
- Table 4.14, summarising the long-term settlement rate data.
- Figure 4.79 Settlement, as a percentage of the height from the SMP to foundation level, versus log time for selected case studies.
- Figure 4.80 to Figure 4.82 Lateral displacement versus log time for selected case studies.

A full compliment of the total settlement (at 3, 10 and 20 to 25 years after construction), settlement versus time and displacement versus time plots are presented in Section 2.1 of Appendix A.

For all plots the data has been sorted based on material type forming the downstream shoulder; rockfill, gravels or earthfill. Embankments with rockfill in the downstream shoulder have been further sorted based on compaction rating, and for the total deformation and settlement rate plots case studies where the rockfill was placed without the addition of water have been highlighted.

Material Type	Compaction Rating	No. Cases	Long-term Settlement Rate *1, *2 (% per log cycle of time)	Comments				
Rockfill	well 10		0.0 to 0.33					
	reas to well	9	$0.04 \text{ to } 0.31 \text{ (most } \ge 0.15)$					
	reas	4	0.10 and 0.31	Ataturk = 0.96%, Gepatsch = 0.99%				
	poor	7	0.10 to 0.25					
	poor – dry	6	0.20 to 0.75					
Gravels	-	6	0.02 to 0.065	0.24% at Meeks Cabin				
Earthfills	-	19	0.0 to 0.40	Refer Figure 4.78 for outliers.				

Table 4.14: Range of long-term settlement rate of the downstream slope.

Note: *1 Exc

Most of the essential features of the post construction deformation behaviour of the downstream slope are captured in the plots and tables of total deformation and long-term settlement rate.

The typical trend for post construction settlement is for near constant or slightly increasing rate of settlement with the log of time. Periods of higher settlement rate are observed for some of the embankments with poorly and reasonably compacted rockfill in the downstream shoulder. Notable features of the post construction settlement are:

• In terms of total settlement:

- Settlements in the order of 1 to 2% are observed for poorly compacted rockfills. Greater settlements are observed for the dry placed, poorly compacted rockfills.
- For reasonably compacted rockfills the range of settlement is quite broad, from 0.1% up to 1.0%, but the number of cases is limited. Settlements toward the upper range are observed for dry placed and/or weathered rockfills, where settlements due to collapse compression are likely to be significant.
- Much lower settlements, generally less than 0.5 to 0.7% at ten years after construction, are observed for well and reasonably to well compacted rockfills, and compacted earthfills.
- Very low settlements (less than 0.25% at 10 years) are observed for embankments with gravel shoulders.

• In terms of the long-term settlement rate:

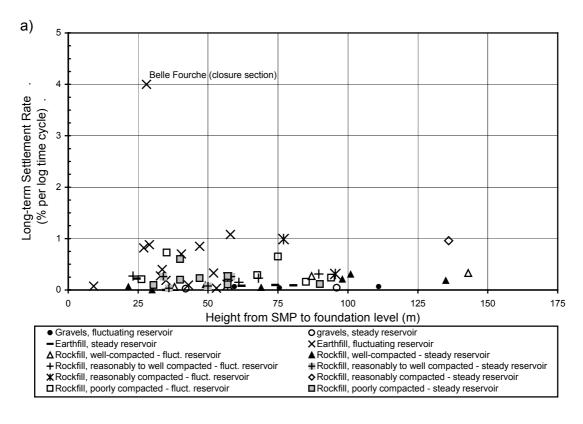
- For most cases the long-term settlement rate is less than 0.4% per log cycle of time.
- Higher settlement rates are observed for a number of earthfill embankments. At Belle Fourche (within the closure section) the very high rate (close to 4% per log cycle) is a clear outlier and indicative of "abnormal" behaviour.
- Higher settlement rates apply for poorly compacted rockfills, ranging from 0.2% to 0.75% per log cycle of time, with the dry placed cases generally at the higher end of the range.
- Higher settlement rates also apply for reasonably compacted rockfills that are susceptible to significant settlements due to collapse compression from wetting (dry placed and/or weathered rockfills). For both Ataturk and Gepatsch dams the indicated rates may be an over-estimate because they have been derived from data inclusive of the first few years after construction.
- Very low settlement rates apply for embankments with gravel downstream shoulders (< 0.10%).

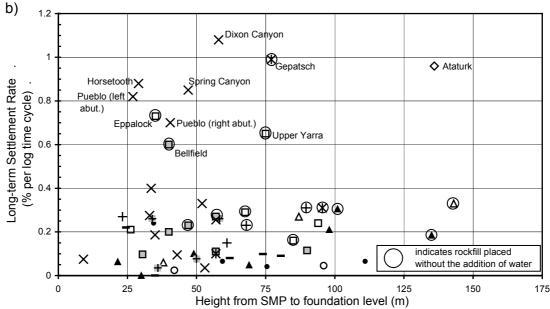
^{*1} Excludes possible outliers.

^{*2} Settlements are a percentage of the height from the SMP to foundation level.

Reservoir operation post first filling has a limited effect on the settlement rate.

As indicated, the post construction settlement behaviour of the downstream slope within the closure section at Belle Fourche dam is a clear outlier and indicative of "abnormal" behaviour. Other case studies where the magnitude of settlement or long-term settlement rate is possibly indicative of "abnormal" behaviour includes Svartevann, Horsetooth, Dixon Canyon, Spring Canyon, Srinagarind, Pueblo (both the left and right embankments), Ataturk and possibly Gepatsch dams. A number of these cases are discussed further in Section 6.0 and in Appendix B.





Note: refer to (a) above for legend.

Figure 4.78: Long-term settlement rates for the downstream slope (mid to upper region) versus embankment height; (a) all data, and (b) data excluding Belle Fourche.

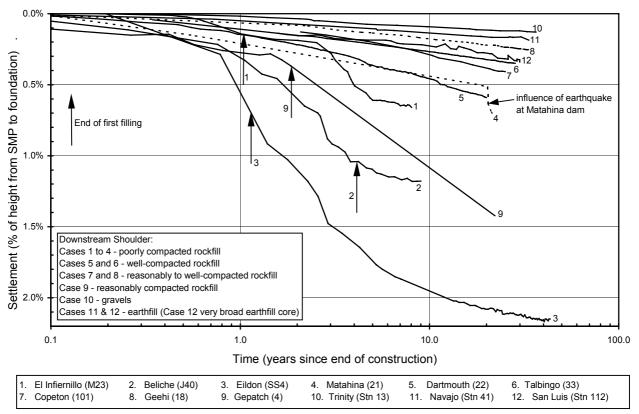


Figure 4.79: Post construction settlement of the downstream shoulder (mid to upper region) for selected case studies.

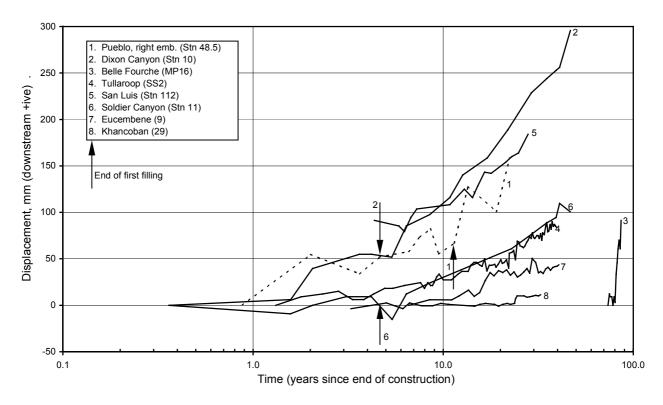
The displacement versus log time for selected case studies are presented in Figure 4.80 to Figure 4.82, and for all case studies in Figures A2.11 to B2.18 in Appendix A. As for the crest displacement, a typical pattern of behaviour for the horizontal displacement of the downstream slope is difficult to define. The general trend is for downstream displacement on first filling and continued downstream displacement long-term, although for several case studies the displacement is slightly upstream. Several trends that are evident are:

(a) Earthfill and zoned embankments with very broad core widths

For embankments of very broad core width the displacement (Figure 4.80) shows a steady downstream rate of displacement with log time, the rate long-term usually being higher than in the early years after construction. First filling generally has little influence on the deformation behaviour, except at Horsetooth dam where the displacement was influenced by the orientation of the cut-off trench and the large deformation of the foundation.

The limited of influence of first filling is largely due to the location and orientation of the applied water load on the upstream face of the very broad earthfill zone. The gradual development of the phreatic surface within the embankment over many years (refer Section 4.3.3, point f) is considered a factor in the observation of higher rates of displacement post first filling.

The magnitude of post construction displacement of the downstream slope is small for earthfill embankments and embankments with very broad core widths. Displacements at 25 to 45 years after construction range from 10 mm to 200 mm downstream, or 0.05% to 0.30% of the embankment height (excludes Horsetooth and Dixon Canyon dams). The displacement at Dixon Canyon dam is much larger at almost 300 mm (0.46% of the embankment height), and at Horsetooth dam is 260 mm, 180 mm of this occurring on first filling due mainly to foundation influence. At Belle Fourche dam the very high rate of displacement of the downstream slope in the closure section (Figure 4.80), where 90 mm was measured over the period 75 to 85 years after construction, is considered to be "abnormally" high.



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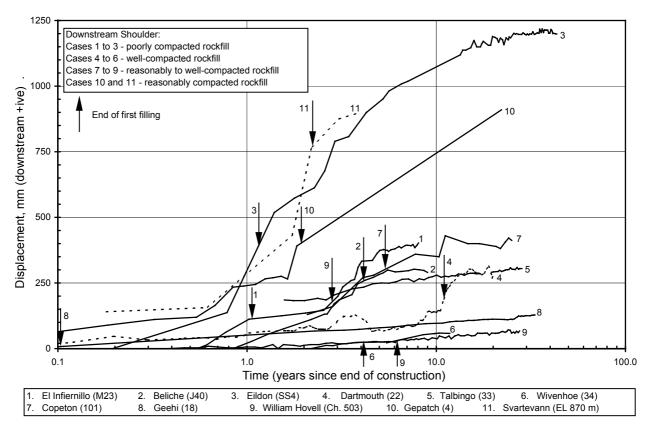
Figure 4.80: Post construction displacement of the downstream shoulder (mid to upper region) for selected case studies of embankments with very broad core widths.

(b) Central core earth and rockfill embankments

For central core earth and rockfill embankments the general trend of displacement of the downstream slope is for higher rates of downstream displacement (rate per log time) during the first few years after end of construction decreasing to much lower rates long-term (Figure 4.81). Two explanations for this are; firstly, for thin to medium central cores the water load has a high horizontal component and is applied close to the centre of the embankment resulting in an increase in lateral stress in the downstream shoulder on first filling (Section 4.2.1) and downstream displacement of the downstream shoulder. Secondly, wetting of the downstream rockfill from rainfall infiltration or tail water inundation causing large deformations in those rockfills that are susceptible to large deformations due to collapse compression. Rockfills most susceptible, as previously identified, are those that are dry placed and reasonably to poorly compacted, or where the rock substance strength is susceptible to large loss of strength on wetting.

The magnitudes of displacement of the downstream shoulder is generally in the range:

- For well and reasonably to well compacted rockfills displacements long-term (more than 10 to 20 years after construction) are typically less than 0.20% of the embankment height, or less than about 100 to 300 mm. Larger displacements, up to 0.25% to 0.40% of the embankment height, can occur where the rockfill has been placed dry, the rock substance strength is susceptible to large strength loss on wetting and/or the outer zone of the rockfill has been placed in thicker layers (i.e. reasonably compacted).
- For reasonably and poorly compacted rockfills displacements can reach values up to 1.0% to 1.6% of the embankment height long-term. Those most susceptible to the larger displacements are rockfills that have been dry placed and poorly compacted. Post construction displacements close to and in excess of 1 m have been measured at Ataturk (184 m high), Eildon (80 m high), Gepatsch (153 m high) and Svartevann (129 m high) dams. As a percentage of the embankment height the highest displacements have been measured at the 80 m high Eildon dam (1.52% of the embankment height) and the 46 m high Eppalock dam (1.63% of the embankment height). For both these dams the rockfill was dry placed and poorly compacted.

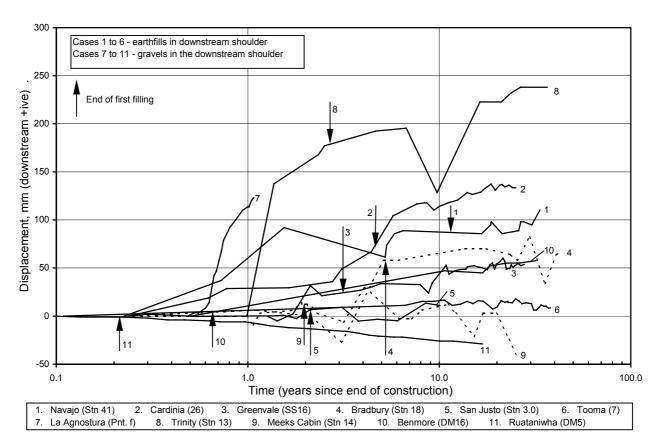


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Figure 4.81: Post construction displacement of the downstream shoulder (mid to upper region) for selected case studies of central core earth and rockfill embankments.

(c) Zoned embankments with compacted earthfill and gravels in the downstream shoulder

For zoned embankments with compacted earthfills and gravels in the downstream shoulder, the post construction displacement (Figure 4.82) is relatively small at less than about 150 to 250 mm (less than 0.15 to 0.20% of the embankment height) some 20 to 30 years after construction. For most of the case studies the displacement rate (per log cycle of time) decreases with time approaching near zero or very low values long-term. In general, first filling has a limited to negligible influence on the displacement, except at La Angostura dam, where the embankment comprised a thin clay core, and possibly Trinity dam.



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Figure 4.82: Post construction displacement of the downstream shoulder (mid to upper region) for embankments with compacted earthfills and gravels in the downstream shoulder.

4.3.6 Post Construction Deformation of the Upper Upstream Slope and Upstream Crest

Post construction deformation data of the upper upstream slope and upstream crest region has been presented in previous sections, including:

- Lateral displacement normal to the dam axis at the end of first filling in Section 4.2.3. Figure 4.40 presents the lateral displacement at end of first filling versus embankment height.
- Total settlement plots at 3 years (Figure 4.51) and 10 years (Figure 4.52) after the end of embankment construction in Section 4.3.2. Table 4.12 summarises the typical range of settlement sorted based on material type and placement method of the upstream shoulder material. The data was plotted against the height from the SMP to foundation level.

Additional data on the post construction deformation of the upper upstream slope and upstream crest region presented below includes:

- Figure 4.83 Long-term settlement rate (% per log cycle of time) versus embankment height. The calculated rate excludes short periods of increased settlement rate post first filling shown for some of the embankments, such as on drawdown at 17 years for Djatiluhur dam (Figure 4.84).
- Table 4.15, summarising the long-term settlement rate data.
- Figure 4.84 and Figure 4.85 Settlement (% of height from SMP to foundation) versus log time for poorly compacted rockfills and selected other case studies respectively.
- Figure 4.86 and Figure 4.87 Lateral displacement versus log time for selected case studies.

A full compliment of the total settlement (at 3, 10 and 20 to 25 years after construction), settlement versus time and displacement versus time plots are presented in Section 2.2 of Appendix A.

For all plots the data has been sorted based on material type forming the upstream shoulder; rockfill, gravels or earthfill. Embankments with rockfill in the downstream shoulder have been further sorted based on compaction rating, and for the total deformation and settlement rate plots case studies where the rockfill was placed without the addition of water have been highlighted.

Several trends are apparent for the post construction settlement for the upper upstream shoulder and upstream crest region. For most case studies the usual trend is for near constant or slightly increasing rate of settlement with the log of time. In some cases a decrease or increase in settlement rate is observed post first filling or the settlement versus time curve shows a slight curvature of increasing rate with log time, but the transition is generally smooth. This type of settlement time is generally observed for:

- Zoned embankments with permeable earthfills or gravels in the upstream shoulder.
- Embankments with very broad earthfill cores. Horsetooth is an exception, but this may be due to foundation influence.
- Well-compacted and reasonably to well compacted rockfills in the upstream shoulder. Exceptions are La Angostura (possibly affected by the outer dumped rockfill zone), Dartmouth, Blowering, Wyangala and La Grande 2 dams.
- A limited number of case studies with reasonably and poorly compacted rockfills in the upstream shoulder, e.g., Tooma and Burrendong, both of which comprised dumped and sluiced rockfills placed in 1.8 to 3 m lifts.

For a number of the poorly and reasonably compacted rockfills higher settlement rates or periods of higher settlement rate during first filling followed by a relatively steady, reduced settlement rate post first filling are observed. Relatively large collapse settlement of the upstream rockfill on first filling is likely to be the cause of this settlement trend.

A limited number of the case studies show periods of higher settlement rate post first filling, generally occurring on drawdown. These cases include Dartmouth, Blowering, Wyangala, Cougar, Djatiluhur, Beliche, Eildon and Eppalock dams, most of which are represented in Figure 4.84 or Figure 4.85. This type of settlement trend is potentially indicative of "abnormal" deformation behaviour. Assuming the rockfill is of high permeability, close to the highest vertical stress levels are reached in the wetted rockfill during first filling. On drawdown the vertical stresses in the upstream rockfill will increase, but it is essentially in re-loading. Effective stress levels greater than previously experienced by the wetted rockfill can potentially occur in the lateral direction, normal to the dam axis, as the water load acting on the core is reduced and therefore transferred onto the upstream rockfill. Another possible mechanism is localised instability of the core on drawdown due to the reduction in lateral stresses acting on the core on drawdown. These mechanisms, along with a number of the cases studies, are discussed further in Section 6.3.

Table 4.12, Figure 4.51 and Figure 4.52 provide information on the general range of settlement of the upper upstream slope and upstream crest region sorted based on material type and placement method of the upstream shoulder material. In summary, the general ranges of settlement as a percentage of the height from the SMP to foundation level are:

- In the order of 1 to 2% for poorly compacted rockfills. Greater settlements are observed for the dry placed, poorly compacted rockfills. As shown in Figure 4.84 a large portion of the settlement occurs on first filling as collapse type settlement.
- For several embankments with reasonably compacted rockfill in the upstream shoulder, notably Gepatsch and Cougar dams, greater settlements are observed and are probably due to large collapse type settlements in the dry placed rockfill at Gepatsch and the weathered rockfill at Cougar.

- Less than 0.5% to 0.7% settlement at 10 years for the well and reasonably well compacted rockfills, and for embankments with earthfill zones in the upstream shoulder, including earthfill embankments.
- Less than 0.25% settlement at 10 years for embankments with gravel shoulders, but there are only three cases with deformation data from the database.

The long-term settlement rate data (Figure 4.83 and Table 4.15) indicates:

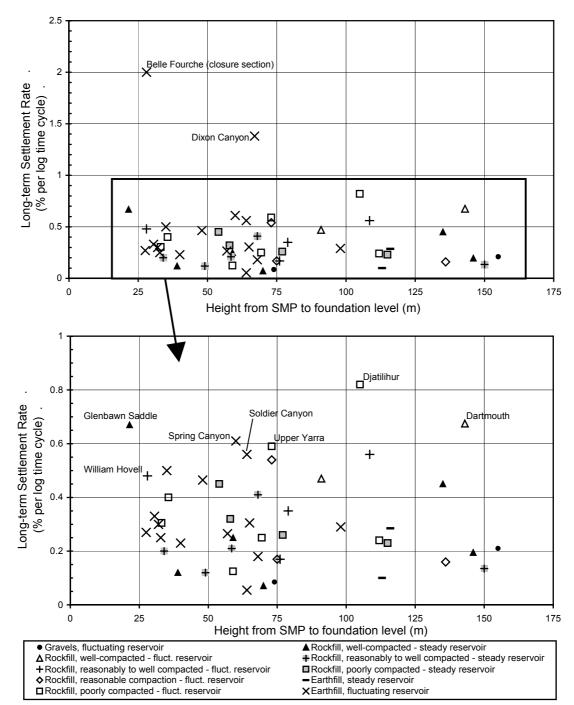
- For most cases the long-term settlement rate is less than 0.4 % per log cycle of time.
- Higher rates, up to 0.8% per log cycle of time, are observed for a number of embankments. There is no apparent consistency in material type and reservoir operation for these cases.
- Embankment height, reservoir operation, material type and compaction rating do not appear to have a significant influence. Gravels possibly have a lower settlement rate, but there are only two cases.

Potential outliers in terms of magnitude of settlement of the upstream shoulder include Dixon Canyon, Spring Canyon and Horsetooth dams. Belle Fourche dam (within the closure section) is a clear outlier in terms of long-term settlement rate as is probably Dixon Canyon dam. The settlement rates are on the high side at Glenbawn Saddle and Dartmouth dams compared to other well-compacted rockfills. Several of these cases are discussed further in Section 6.0.

Table 4.15: Range of long-term settlement rate of the upper upstream slope and upstream crest region.

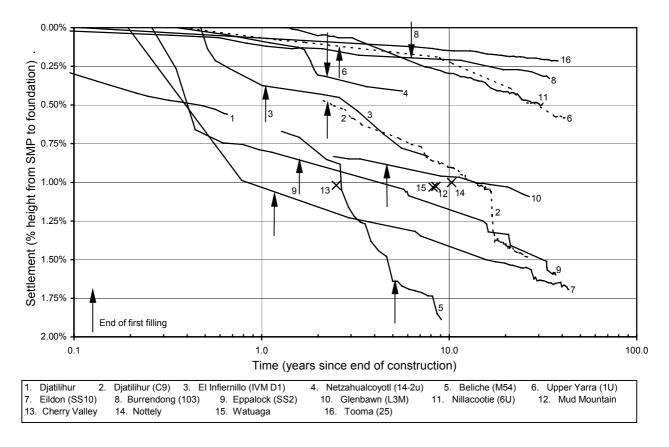
Material	Compaction	No.	Long-term Settlement Rate *1	Comments			
Type	Rating	Cases	(% per log cycle of time)				
Rockfill	well	8	0.05 to 0.70 (most < 0.50)	Dartmouth = 0.67%, Glenbawn Saddle = 0.675%			
	reas to well	9	0.10 to 0.56 (most < 0.50)				
	reas	3	< 0.55				
	poor	7	0.10 to 0.82 (most < 0.60)	Djatiluhur = 0.82%			
Gravels	-	2	< 0.21				
Earthfills	-	18	0.1 to 0.60	Belle Fourche (closure section) = 2%,			
				Dixon Canyon = 1.38%			

Note: *1 Excludes possible outliers.



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Figure 4.83: Long-term settlement rates of the upper upstream slope to upstream crest region of earthfill and earth-rockfill embankments.



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Figure 4.84: Post construction settlement of the upper upstream slope to upstream crest region for embankments with poorly compacted rockfill in the upstream slope.

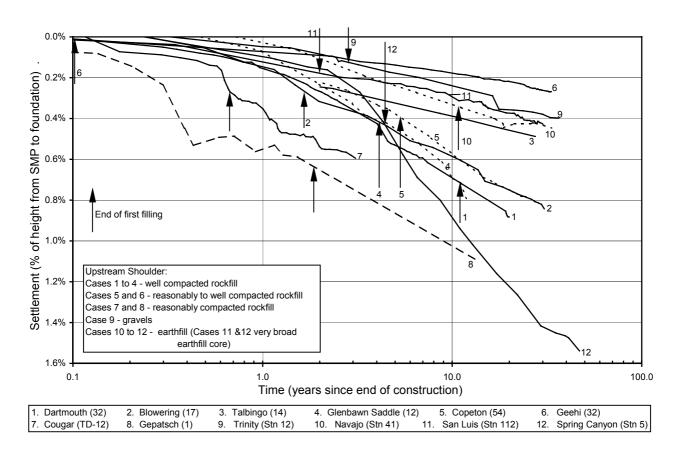


Figure 4.85: Post construction settlement of the upper upstream slope to upstream crest region for selected case studies (excluding poorly compacted rockfills).

The horizontal displacement versus time of the upper upstream slope and upstream crest region for selected case studies is presented in Figure 4.86 and Figure 4.87. The data for all case studies is presented in Figures A2.30 to A2.36 in Appendix A. As shown, there is a large variation in the magnitude and direction of the measured displacements. The data presented below has been sub-divided into two groups, those embankments with rockfills in the upstream shoulder, and those with earthfills, including gravels, in the upstream shoulder.

(a) Horizontal displacement for embankments with rockfill in the upstream shoulder

Figure 4.86 presents the horizontal displacement of the upper upstream slope and upstream crest region of selected case studies with rockfill in the upstream shoulder. The case studies have been selected covering the range of rockfill compaction rating from well compacted through to poorly compacted.

For rockfills that are well and reasonably to well compacted, displacements of the upstream shoulder during and after first filling are generally relatively small. For some case studies a large portion of the displacement occurs on first filling (e.g. for Talbingo dam), whilst for others first filling has a limited influence. Post first filling displacements are small in magnitude and long-term the displacement rate (per log cycle of time) approaches a small to near zero value, with variations about the trend due to reservoir fluctuation. The typical range of displacement for these case studies is:

- At more than 10 to 20 years after construction, displacements range from -0.10% to 0.15% of the embankment height (or -75 mm to 200 mm).
- Additional displacement post first filling ranges from -0.05% to 0.05% of the embankment height (-50 to 50 mm).

For rockfills that are reasonably to poorly compacted, displacements can be much larger reaching magnitudes of close to 1 metre, or close to 1% of the embankment height. The case studies with large displacements tend to be dry placed rockfills. A large portion of the displacement occurs on first filling and long-term the displacement rate (per log cycle of time) approaches a small to near zero value, with variations about the trend due to reservoir fluctuation. The typical range of displacement for these case studies is:

- At more than 10 to 20 years after construction, displacements range from -0.70% to 0.80% of the embankment height (-600 mm to 1000 mm for the case studies). For most of the case studies the long-term total displacement is less than about 0.2% to 0.3% of the embankment height, either upstream or downstream. Large displacements were measured at Eppalock, Gepatsch, Svartevann and Glenbawn dams, all dry placed rockfills.
- Additional displacements post first filling generally range from -0.15% to 0.05% of the embankment height.

Factors affecting the magnitude of displacement of the upstream slope are likely to include:

- The magnitude of differential deformation between the upstream shoulder and the core or upstream transition zone of the embankment. For rockfills susceptible to large deformations on collapse compression, differential deformations on first filling can be large, and the vector of deformation of the upstream shoulder will be affected by the internal zoning geometry of the embankment.
- Zoning of the rockfill in the upstream shoulder. Where placement methods, rock types or the degree of
 weathering differs for different rockfill zones in the upstream slope then differential deformations can
 develop between these rockfill zones and therefore influence the magnitude and direction of the
 displacement.
- The deformation of the core and downstream shoulder of the embankment. The closer the SMP on the upstream slope is located to the crest the greater the influence the displacement of the core and downstream shoulder will be. In the case of Svartevann and La Grande 2 dams large downstream displacement of SMPs on the crest (both the upstream and downstream edge) and downstream shoulder occurred on first filling.
- The slope of the upstream shoulder will have some influence.

For a number of embankments the long-term trend of displacement shows several trends that are potentially indicative of "abnormal" behaviour. These include, continuing high or increasing rates of displacement (rate per log of time), and non-recoverable displacements (usually upstream) on drawdown. Examples of this behaviour include Blowering, Eppalock and possibly Eildon dams as shown in Figure 4.86. Other examples of this type of behaviour are observed at Copeton and Wyangala dams. A number of these cases are discussed further in Section 6.3 and in Appendix B.

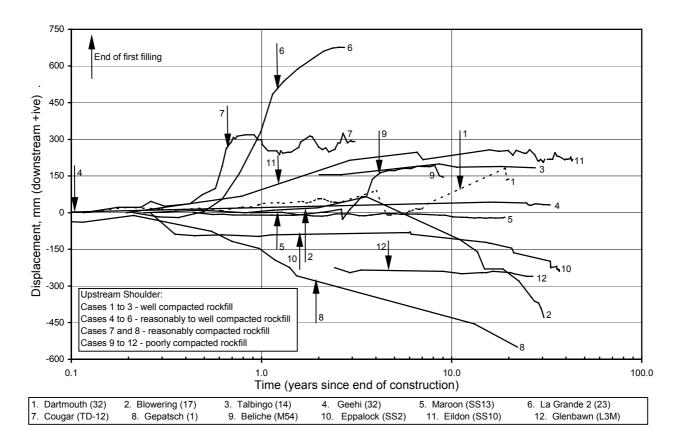


Figure 4.86: Post construction lateral displacement of the upper upstream slope and upstream crest region for selected case studies of embankments with rockfill in the upstream shoulder.

(b) Horizontal displacement for embankments with earthfills and gravels in the upstream shoulder

Embankments included in this sub-group include earthfill embankments, embankments with very broad core width, and zoned embankments of thin to thick core width with earthfills or gravels in the upstream shoulder. The displacement versus time for selected case studies is shown in Figure 4.87.

For most case studies the magnitude of post construction displacement is relatively small, and is either in an upstream direction or downstream direction. Long-term the displacement rate (per log cycle of time) approaches very small (either up or downstream) to near zero average values with fluctuation due to operation of the reservoir. At more than 20 to 30 years after construction, the magnitude of displacement is generally in the range -0.10% to 0.12% of the embankment height, or from 100 mm upstream (-100 mm) to 200 mm downstream.

The influence of the compressible foundation is evident at Horsetooth dam where a large downstream displacement occurred on first filling. The deformation of the compressible foundation has possible also influenced the displacement of the section of San Luis dam over the broad alluvial plain (San Luis – Stn 80 in

Figure 4.87), as well as Steinaker dam. For these two dams the displacements post first filling were larger than at other dams.

A number of case studies show the initial displacement on and after first filling to be upstream, and then long-term changes to downstream. These case studies include; Navajo, Dixon Canyon, San Luis (main section and slide area), Spring Canyon, Soldier Canyon and the right abutment embankment at Pueblo dam. This effect may be due to development of the phreatic surface or possibly softening within the embankment.

The high rate of displacement of the upstream shoulder within the closure section at Belle Fourche dam many years after construction is likely to be indicative of "abnormal" behaviour. The deformation behaviour at Dixon Canyon dam, showing a large upstream displacement on and after first filling followed by downstream displacement, is also potentially "abnormal" when compared to that at similar embankments.

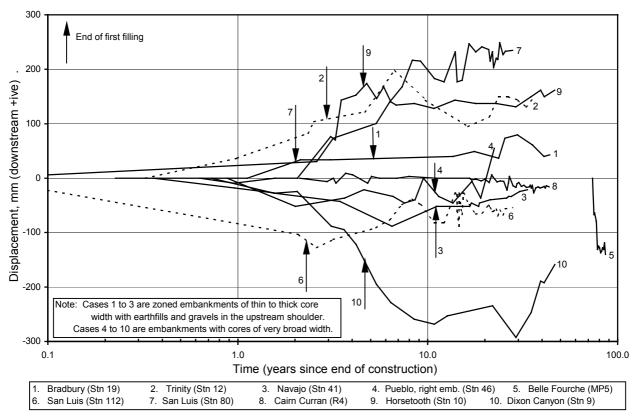


Figure 4.87: Post construction lateral displacement of the upper upstream slope to upstream crest region for selected case studies of embankment with earthfills and gravels in the upstream shoulder.

5.0 GENERAL DEFORMATION BEHAVIOUR OF PUDDLE CORE EARTHFILL EMBANKMENTS

5.1 Deformation During Construction of Puddle Core Earthfill Embankments

Selset Dam (Section 2.6 of Appendix D) is the only puddle dam for which deformation behaviour during construction has been found in the published literature. It was constructed in the late 1950's using relatively modern compaction techniques compared to the remainder of the data set. Therefore, the deformation behaviour of Selset Dam is not likely to be typical of most puddle dams. However, the deformation behaviour and pore water pressure response during construction at Selset dam does highlight several important aspects raised by Bishop and Vaughan (1962) fundamental to puddle cores:

- Arching develops across the core resulting in significant reductions in the total vertical stresses within the core.
- The deformation of the core occurs predominantly as yielding in undrained loading conditions, and the large observed settlements are primarily due to lateral deformation of the narrow puddle core.
- Consolidation type settlements during construction are not significant.
- Relatively high pore pressures are developed in the puddle core during construction and are still present at the end of construction.
- The internal settlement of the puddle core during construction at Selset Dam (Figure 5.1) is well in excess of that observed in wet placed, rolled clayey earthfills in zoned embankments. The vertical strains in Selset dam reached as high as 5 to 7% at 10 m depth below crest level. In comparison, vertical strains at a similar depth below crest were less than about 2% in zoned embankments with narrow rolled earthfill cores (Figure 4.22).

The issue of arching in embankments with narrow cores is discussed in Section 4.1.1.3 with reference to the literature on puddle core earthfill embankments.

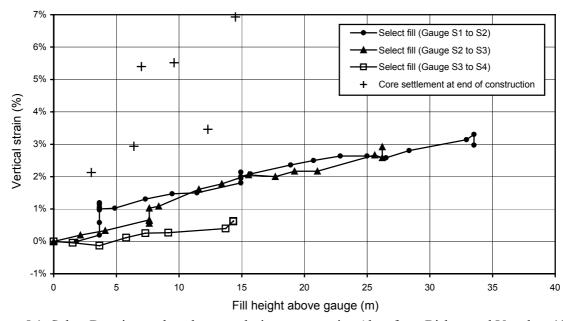


Figure 5.1: Selset Dam internal settlements during construction (data from Bishop and Vaughan 1962)

The rate of dissipation of pore pressures in the puddle core is likely to be affected by the permeability and width of the core, as well as the permeability of the adjacent shoulder earthfill if it is similar to that of the puddle core.

The amount of lateral deformation of the puddle core will primarily be influenced by the differential lateral stress conditions between the puddle core and supporting shoulders and the compressibility of the supporting shoulder earthfill. The lateral stresses that develop within the puddle core will depend on its shear strength and compressibility properties, as well as the stress reducing influence of arching of the core.

5.2 DEFORMATION DURING FIRST FILLING OF PUDDLE CORE EARTHFILL EMBANKMENTS

Charles (1998) identifies the poorly compacted upstream shoulder of the older puddle dams (presumably those embankments constructed prior to the 1900's without formal compaction) as prone to collapse compression on impounding.

Information on the susceptibility of earthfills to collapse compression from the case study data on rolled earthfills is discussed in Section 4.2.2. In the context of puddle dams, the susceptibility of materials to collapse compression and its effect on the deformation behaviour of the embankment is summarised as follows:

- For puddle dams constructed prior to 1900, typically without any formal compaction of the shoulder earthfill, collapse compression on inundation of the shoulders is likely to be significant.
- For dams constructed without a select zone adjacent to the core, which was typical before 1860, collapse compression is likely to be significant. From the available literature it is apparent that the select zone adjacent to the core was placed in thinner layers, thereby possibly reducing the amount of collapse compression adjacent to the puddle core.
- For puddle core earthfill dams constructed after the late 1930s, when soil mechanics principles were applied to embankment construction, moisture conditioning and good compaction of the shoulder fill could be expected (e.g. Selset Dam). As a result, collapse compression of the upstream shoulder earthfill on inundation is not likely to have occurred.

The puddle core itself is not susceptible to collapse compression. However, because of its very low undrained strength its post construction deformation behaviour is highly dependent (or largely controlled by) the deformation behaviour of the supporting shoulders. If the upstream (or downstream) shoulder region adjacent to the core settles due to collapse compression on wetting, then the puddle core will settle with the upstream shoulder. Therefore, the magnitude of crest settlement of the puddle core embankment on first filling will depend on the embankment zoning geometry and the susceptibility of the upstream shoulder earthfill zones to collapse compression on wetting. The mechanism of deformation of the puddle core on first filling is similar to that of zoned embankments with wet placed cores of low undrained strength and thin to medium width (Section 4.2.2, Figure 4.35).

For puddle dams with well-compacted earthfills either side of the puddle core that are not susceptible to collapse compression, crest settlements will be limited during first filling as observed at Selset and Burnhope dams (Figure 5.5). In the case of Burnhope dam (constructed in 1935) the upstream filling was placed in 450 mm layers and compacted by 10 to 17 tonne steamrollers. In the first four years after construction the settlement of the upstream slope was 104 mm (or 0.4% of the dam height) and 468 mm (or 1.2%) at the crest. For Selset dam, where the shoulder filling was well compacted with moisture content control, the crest settled 177 mm (0.45% of the dam height) on first filling. As shown in Figure 5.5 the magnitude of crest settlement for these dams is relatively small compared to other puddle core dams and the settlement versus time curve shows virtually no influence due to first filling.

Where the shoulder fill adjacent to the puddle core is dry placed and poorly compacted, and therefore susceptible to large magnitude collapse settlements on wetting, crest settlements will be large on first filling as

observed at Hope Valley dam (Figure 5.5). The mechanics of the deformation behaviour for this condition are discussed in more detail in Section 5.2.1.

5.2.1 Collapse Compression of Poorly Compacted Filling

The mechanism of deformation considered to be involved during first filling and first drawdown of an old puddle dam with no formal compaction of the shoulder filling is summarised as follows:

- On first filling:
 - Wetting of the poorly compacted upstream filling results in collapse of the soil structure and therefore large settlement of the upstream fill (Figure 5.2a to c). The strength and compressibility properties of the wetted earthfill will be significantly different to those of the "as placed" earthfill; its compressibility and undrained strength will be significantly reduced.
 - The effect of first filling on the deformation of the upstream shoulder is for large settlement as a result of the collapse of the soil structure on wetting.
 - At the upstream (and downstream) interface between the puddle core and shoulder, the core must settle with the shoulder. This is because the puddle core does not have the shear strength to support any significant levels of stress that would be transferred to it by differential settlement.
 - The total horizontal stress acting on the upstream face of the puddle core must at least equal the lateral stresses within the puddle core for equilibrium. Hydrostatic pressures acting on the upstream face of the puddle core increase with the rise in reservoir level, but the lateral stresses within the upstream shoulder decrease due to the substantial loss in strength and compressibility on collapse due to wetting (Figure 5.2d). The deformation of the upstream face of the core / shoulder interface is dependent on the net lateral stress acting. If the net lateral stress at any point increases then the interface at that point will deform downstream and horizontal stresses will increase in the puddle core and downstream shoulder. If the net lateral stress decreases then the interface will deform upstream until equilibrium conditions are re-established as lateral stresses increase in the upstream shoulder. It is possible that the interface can initially deform upstream at low water levels and then deform downstream as the reservoir approaches full supply level.
 - If upstream displacement of the core / shoulder interface occurs then lateral spreading of the core results. The deformation of the puddle core occurs as undrained plastic yielding and is therefore associated with vertical strain to maintain volumetric consistency. Further arching across the core is also likely where lateral spreading occurs, thereby decreasing the lateral stress required for equilibrium.
 - Once full supply level is reached, whether or not equilibrium pore water pressure conditions in the upstream shoulder have been reached is dependent on its permeability and the time for first filling. Free draining earthfills will effectively reach equilibrium conditions as the reservoir is filled. For the case studies analysed, in most cases the earthfill comprises a mix of clay to gravel (and larger) sized particles and its permeability is quite variable due to layering as well as material and density variations within and between layers. Therefore, it is likely that for most cases equilibrium pore water pressure conditions are not reached in the upstream shoulder when full supply level is reached (Figure 5.2b). As a result, the upstream shoulder will continue to wet up after first filling is completed and settlements can continue to occur post first filling within the shoulder and possibly the puddle core.

On first drawdown:

Further compression of the upstream shoulder occurs as effective stresses increase under the falling reservoir level. Because the upstream earthfill was not fully saturated at end of first filling, its drained compressibility will have continued to decrease as further softening occurred on wetting post first filling. Therefore, poorly compacted earthfills susceptible to collapse compression and softening on wetting are likely to be close to normally consolidated after the end of first filling, and increases in

- effective stresses on drawdown will result in large deformations due to yielding. The upstream shoulder can therefore experience large settlements on drawdown (Figure 5.3).
- The puddle core will also undergo significant settlement and displacement on initial drawdown (Figure 5.3). As the hydrostatic pressures acting on the upstream face of the puddle core decrease, total stresses in the puddle core and downstream shoulder initially decrease and the embankment deforms upstream. At some point, the lateral stresses in the upstream shoulder must increase to maintain equilibrium with those within the puddle core. Because of the large reduction in compressibility of the poorly compacted upstream shoulder earthfill on wetting, lateral spreading of the puddle core will occur as the upstream shoulder deforms under the increased lateral stresses imposed on it as hydrostatic pressures continue to decrease on drawdown. The deformation of the puddle core occurs as undrained plastic deformation (i.e. yielding) and the lateral spreading is associated with vertical strains to maintain volumetric consistency, and therefore crest settlements are potentially large.

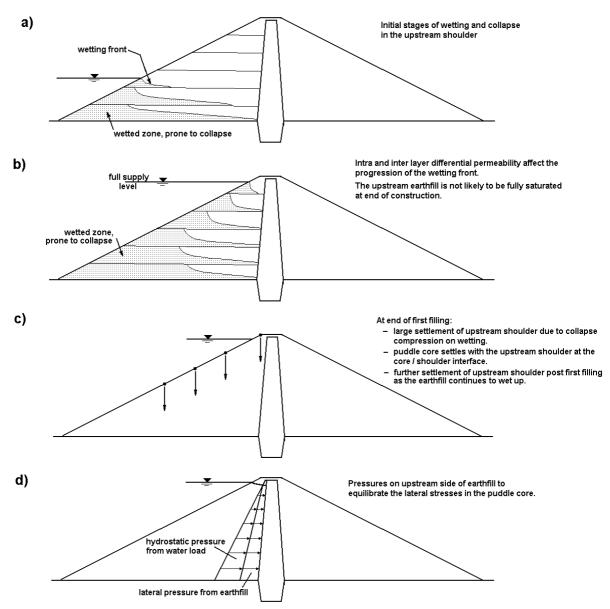


Figure 5.2: "Idealised" model of collapse compression of poorly compacted shoulder fill of puddle core earthfill dam on first filling.

Figure 5.3: "Idealised" model of yielding of poorly compacted shoulder fill and puddle core on first drawdown of a puddle core earthfill dam.

In summary, deformation of the upstream shoulder earthfill (where it is poorly compacted) will be significant on initial filling due to collapse compression. The settlement of the puddle core will also be large on first filling as it deforms with the adjacent upstream zone of the shoulder. On first drawdown large deformations can occur in the upstream shoulder due to drained yielding as effective stresses increase in the near normally consolidated water softened earthfill. Deformations of the puddle core will be large on first drawdown mainly due to lateral spreading as the now softened adjacent shoulder deforms under the increased lateral stresses applied as hydrostatic pressures decrease. Subsequent drawdowns that result in increases in effective stresses in the upstream shoulder above those previously experienced post first filling will also result in large deformations of the upstream shoulder and puddle core (this is discussed in Section 5.3.5).

In the case of wetting of part of the downstream filling (Figure 5.4), such as due to seepage through the foundation, or hydraulic fracture through the puddle core due in part to arching of the narrow core, the wetted zone of the filling would undergo collapse compression assuming it to be in a poorly compacted and dry condition. Consequently, deformations of the downstream shoulder and crest region are likely to be large.

In the case of Hope Valley dam (refer Section 2.3 of Appendix D), wetting of the downstream fill was observed shortly after the start of initial filling. This was probably mainly due to seepage in the Tertiary alluvial soils in the foundation. From Figure 5.5 it is evident that significant crest settlement (1250 mm or 5.5% of the embankment height) occurred in the period up to 11 years after construction. Collapse compression on wetting of both the upstream and downstream earthfill shoulders were likely to be significant contributing factors to the large crest settlement, in addition to the deformations due to yielding on drawdown. What influence wetting and collapse compression of the downstream shoulder earthfill had on the magnitude of crest settlement is not known, and the data in Figure 5.5 does not include many comparable dams. But, in comparison to the anecdotal information of the crest settlement for several old UK puddle dams the magnitude of crest settlement of Hope Valley dam is relatively high, possibly suggesting its influence was significant.

In the case of the Dale Dyke dam failure in 1864, Charles (1998) suspected that collapse compression on wetting of the earthfill in the downstream shoulder may have contributed to the loss of freeboard. Although, Binnie (1978) surmises that the large settlement was localised to a small portion of the crest and due to loss of material from piping, a view supported by Foster et al (1998, 2000). Instability of the downstream slope prior to overtopping as a result of strength loss within the downstream earthfill shoulder could also have been a factor.

It is notable that longitudinal cracking between the core and upstream filling has not been reported on any puddle dams as a result of collapse compression, as has been the case for a number of central core earth and rockfill dams. It is considered that this is due to the low undrained strength and plastic nature of the puddle clay core. Rather than a crack appearing, the core deforms with the upstream shoulder and may result in the ridge or bulge developed on the upstream face near to crest level that it evident on several puddle dams, or a narrow

wedge of upstream zone adjacent to the core dropping to form a "reverse" scarp as is the case for Happy Valley dam.

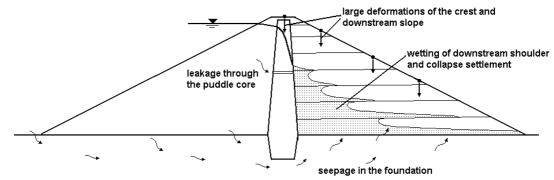


Figure 5.4: "Idealised" model of collapse compression on wetting of poorly compacted earthfill in the downstream shoulder of a puddle core earthfill dam.

5.3 Deformation Behaviour Post First Filling of Puddle Core Earthfill Dams

Section 2.3.3 summarises the research by BRE and Imperial College in terms of the factors they consider to affect the long-term deformation behaviour and the mechanism/s affecting the deformation behaviour of puddle core earthfill embankments.

The following sub-sections present an analysis of the deformation behaviour of the puddle core earthfill dam case studies in the context of defining "normal" deformation behaviour, and the various mechanisms associated with the observed deformation behaviour are discussed. Section 6.5 discusses the methods of identification of "abnormal" deformation behaviour of puddle core earthfill embankments and summarises several case studies considered to indicate "abnormal" deformation behaviour.

5.3.1 Available Deformation Records

The database comprises seventeen case studies of puddle core earthfill dams. The embankment details, reservoir operation, hydrogeology and monitoring for each case study are summarised in Table D1.1 of Appendix D. Table 5.1 presents a summary of the post construction surface deformation behaviour of the crest and slopes for each case study. Figure 5.5 presents the post construction crest settlement versus log time for those embankments with records available from end of construction (time is in years and zero time is the end of construction).

Figure 5.5 shows that the post construction crest settlement of puddle core earthfill embankments is significant, ranging from 1% up to 8 to 14% of the dam height after more than 100 years. It is generally the older (pre 1900) embankments that show the greater post construction deformations. The more recent puddle core earthfill embankments constructed in the 1930's to 1950's generally show significantly lower crest settlement (less than 1 to 4% at 10 to 50 years after construction) as expected given the differences in placement methods of earthfill and compaction equipment between the two periods.

Table 5.1: Summary of the post construction surface deformations of the puddle core earthfill dam case studies.

Name	Year Completed		Reservoir Operation	Response to Drawdown Upstream of Puddle Core	Period of Monitoring	Deformation of Embankment Crest				Upstream Slope			Downstream Slope		Downstream Toe	
		Height, H (m)				Settlement since EOC Long-term		n Deformation	Settlement		Displacement	Long-term	Long-term	Long-term	Long-term	
						mm	% of H	S _{LT}	Displacement Rate (mm/yr)	S _{LT}	Monitored	Rate (mm/yr)	Settlement	Displacement	Settlement	
Burnhope	1935	41	assume fluctuating	assume minor	EOC to 1957	702 (22 yrs)	1.8	0.80	-	0.14	130 mm in 22 yrs since EOC	-	-	-	-	-
Challocombe	1944	15	Steady	assume minor	EOC to 1981	520 (45 yrs)	3.45	0.80	-	-	-	-	-	-	-	-
Cwnwernderi	1901	22	Steady	assume full	1977 to 1989	-	-	0.90	negligible when at FSL (> 70 yrs after EOC)	-	-	-	-	-	-	-
Dean Head	1840	19	assume fluctuating	assume full	1977 to 1984	-	-	7.4	-	-	-	-	-	-	-	-
Happy Valley	1896	25	Fluctuating (15 to 30% of H)	minor	EOC to 1973 1982 to 1998	770 (76 yrs)	3.1	1.0	-	2.3 to 0.7 (varies with magnitude of drawdown)	29 mm (85 to 102 yrs after EOC)	up to 2.5 mm/yr (85 to 102 yrs after EOC), rate is dependent on magnitude of d'down.	11 mm in 10 yrs (92 to 102 yrs after EOC), SLT = 1.0%	1 mm/yr over 10 yrs (92 to 102 yrs after EOC)	5 mm over 10 yrs (92 to 102 yrs after EOC)	0.6 mm/yr (92 to 102 yrs after EOC)
Hollowell	1937	13.1	assume fluctuating	assume minor	EOC to 1954	817 (17 yrs)	6.2	0.70	-	-	-	-	-	-	-	-
Holmestyes	1840	25	assume fluctuating	negligible	1990 to 1991	-	-	2.3 (only 18 months data)	negligible (150 yrs after EOC)	-	-	-	-	-	-	-
Hope Valley	1872	21	Fluctuating (15 to 25% of H)	variable (minor to full)	EOC to 1983 1988 to 1999	1820 (125 yrs)	8.7	2.6	5 mm/yr at 118 to 128 yrs after EOC (downstream edge of crest)	-	-	-	negligible on mid slope (118 to 128 yrs after EOC)	0.5 mm/yr (118 to 128 yrs after EOC)	-	-
Ladybower	1945	43	assume fluctuating	assume minor	1950 to 1984	-	-	2.0	=	-	-	-	-	-	-	-
Langsett	1904	33	assume fluctuating	nk	EOC to 1907	124 (3 yrs)	3.8	-	-	-	-	-	-	-	-	-
Ogden	1858	25	assume fluctuating	assume full	1989 to 1993	-	1	10 (1 normal drawdown cycle only)	-	-	7 mm on 10 m d'down cycle in 1992/93	-	4 mm on 10 m d'down cycle in 1992/93	-	-	1
Ramsden	1883	25	assume fluctuating	assume full	1949 to 1985 1988 to 1990	(450 mm from 1949 to 1985)	-	6.6 (normal operating conditions)	3 mm/yr (1977 to 1985)	-	12 mm in 2 yrs (1988 to 1990), includes abnormally large d'down	-8 mm in 2 yrs (1988 to 1990)	8 mm in 2 yrs (incl. Extreme d'down)	-2 mm in 2 yrs (incl. Extreme d'down)	2 mm in 2 yrs (1988 to 1990)	negligible (1988 to 1990)
Selset	1959	39	assume fluctuating	minor	EOC to 1970	313 (10 yrs)	0.80	0.4	-	-	-	-	-	-	-	-
Walshaw Dean	1907	22	Fluctuating (50 to 60% of H)	assume full	some early data, 1990 to 1995	-	-	4.5 (1915 to 1943) to 6.7 (1988/90)	3 mm/yr (85 years after EOC)	1.7	2 mm on 13 m d'down in 1988 to 1990.	< 1 mm (1988 to 1990)	5 to 6 mm on 13 m d'down cycle in 1988/90	1 to 2 mm (1988 to 1990)	-	-
Widdop	1878	20	assume fluctuating	assume full	1988 to 1990	-	-	-	-	-	-	-	-	-	-	-
Yan Yean	1857	9.6	Fluctuating (15 to 25% of H)	minor	1986 to 1999	1205 (128 yrs)	12.5	0.5 (upstream edge of the crest) 11 (downstream edge of the crest)	negligible rate at upstream edge of crest. Up to 4.5 to 5.5 mm/yr (increasing rate) on downstream crest and slope -	2.0	4 mm in 6 yrs (> 135 yrs after EOC)	negligible (135 to 141 yrs after EOC)	max 8 - 10 mm/yr, increasing rate with time (> 128 years after EOC)	2 to 3 mm/yr (more than 128 years after EOC)	negligible (more than 130 yrs after EOC)	15 to 20 mm in 13 yrs (more than 130 yrs after EOC), stick - slip type displacement
Yateholme	1872	17	Fluctuating	assume full	1989 to 1992	-	-	5.4 (120 yrs after EOC)	-	-	-	-	-	-	-	-

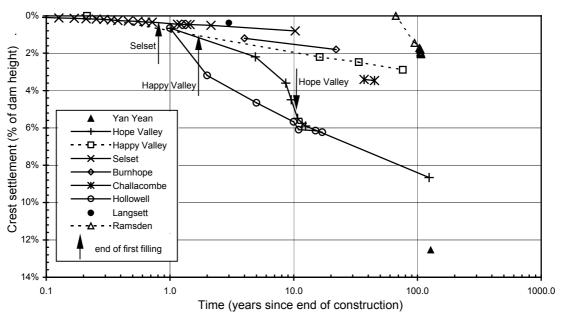
Note: $S_{LT} = \text{long-term settlement index (refer Section 5.3.2, Equation 5.1)}$.

EOC = end of construction

FSL = full supply level

Displacement is the deformation in the horizontal direction normal to the dam axis, positive is downstream and negative is upstream.

d'down = drawdown



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Figure 5.5: Post construction crest settlement versus log time of puddle core earthfill dams.

The understanding of "normal" long-term deformation behaviour is mainly aimed at three situations:

- Deformations during "steady state" conditions. This is basically deformation behaviour assuming the reservoir level is maintained at a constant level, and therefore the components of long-term deformation are foundation settlement, shrink-swell movements, and primary creep type movements under constant stress conditions. Clearly, the data set would be fairly limited given fluctuations in reservoir level occur in most dams, however, as will be discussed later, it is appropriate not only to dams with little fluctuation in reservoir level, but also to cases where the embankment design incorporates a relatively impermeable zone upstream of the core that effectively acts as a water barrier thereby limiting the changes in the phreatic surface in the central portion of the dam.
- Deformations during the normal cyclic operation of the reservoir for puddle core earthfill dams with permeable upstream earthfill.
- Deformations during a historically large drawdown event.

In Table 5.1 the reservoir operation for each case study is categorised as either fluctuating or steady, and the level of fluctuation, where it can be estimated, is given as a percentage of the embankment height. For those case studies where the reservoir operation is not known it has been assumed as fluctuating. Also presented in Table 5.1 is an estimate of the response of the phreatic surface in the zone upstream of the puddle core to changes in reservoir level (more details are given in Table D1.1 of Appendix D). The assessment is based on piezometer records where available, but in their absence has been assessed from the presence or otherwise of a select earthfill zone upstream of the core. The purpose of categorising the response of the phreatic surface in the zone upstream of the puddle core is because it has important implications on the deformation behaviour as discussed in Sections 5.3.2 to 5.3.6. Three categories of response have been used:

- Full or large pore water pressure response to reservoir fluctuation, indicating the earthfill in the upstream shoulder to be permeable.
- Minor response, indicating the earthfill zone upstream of the puddle core is of low permeability such that the change in pore water pressure recorded is a small percentage of the change in reservoir level.
- Negligible response, indicating virtually no change in pore water pressure in the earthfill zone upstream of the puddle core due to the reservoir operation. Only Holmestyes dam is in this category because it has an effective puddle clay blanket on the upstream face of the dam.

It should be noted that the categorisation of the response of the phreatic surface upstream of the puddle core to changes in reservoir level is based on a limited knowledge of the reservoir operation and assumptions regarding the permeability of earthfill zones in the upstream shoulder. For embankments constructed after 1930 it has been assumed that the select filling is well compacted and of low permeability. For Ramsden, Walshaw Dean (Lower), Ogden and Yateholme dams (all Yorkshire Water dams) the shoulder earthfill comprised weathered Millstone Grit and has been assumed as permeable based on the results of constant head tests at Ramsden dam (permeability varying from 2×10^{-5} m/sec to 10^{-7} m/sec). For the Australian dams Yan Yean, Hope Valley and Happy Valley the classification has been based on the pore water response from piezometers installed in the upstream shoulder.

It is recognised that the compressibility of the zones supporting the puddle core have a significant influence on the deformation behaviour of the core (and settlement of the crest) under cyclic reservoir conditions. The compressibility of the earthfill shoulders will generally be lower and not as susceptible to softening on wetting where it has been well compacted (i.e. post 1930) compared to poorly compacted earthfills. Material type will also influence the material compressibility.

5.3.2 Long-term Settlement Rate Index

The long-term settlement rate index, S_{LT} , is used as a guide for prediction of settlements (mainly crest settlement) of puddle core earthfill dams post first filling and as a comparative measure to assess the performance of a specific dam. The settlement rate index, S_{LT} , is calculated using Equation 5.1. It is the same equation as the settlement index proposed by Charles (1986) except that the settlement rate is expressed in units of settlement (as a percentage of dam height) per log cycle of time. The same form of equation has been used to calculate the post first filling long-term settlement rate for the crest and shoulders of rolled earth and earth-rockfill embankments.

$$S_{LT} = s * 100 / [1000 * H * \log(t_2/t_1)]$$
(5.1)

where s is the crest settlement in millimetres measured between times t_2 and t_1 (in years) after the end of embankment construction, and H is the height of the dam in metres.

 S_{LT} values for the puddle core earthfill dam case studies are given in Table 5.1 and have been calculated mainly for the crest of the embankments with some values given for the upper regions of the upstream and downstream shoulders. The quoted S_{LT} values are for periods of measurement during normal operation conditions (i.e. excluding abnormally large drawdown events) and are generally from deformations observed tens of years after construction.

The long-term settlement rate, S_{LT} , basically encompasses all of the mechanisms involved in the long-term deformation behaviour (summarised in Section 2.3.3) into one factor appropriate to analysis of crest settlement for periods of normal reservoir operating conditions. Analysis of the data indicates that a correlation exists between S_{LT} for crest settlement, the reservoir operation (under normal conditions) and the pore water pressure response of the earthfill zone upstream of the puddle core to changes in reservoir level (i.e. full, minor or negligible as discussed in Section 5.3.1). The correlation is summarised as follows:

• For conditions where the change in pore water pressure in the earthfill zone upstream of the puddle core is a small fraction of the change in reservoir level on drawdown or where the reservoir remains steady, S_{LT} generally ranges from 0.4% to 1% settlement per log cycle of time (settlement as a percentage of the embankment height).

• For fluctuating reservoir conditions and where the change in pore water pressure in the earthfill zone upstream of the puddle core closely follows the reservoir level, S_{LT} ranges from 2.6% to 7.4% settlement per log cycle of time.

5.3.3 "Normal" Deformation Under "Steady State" Conditions

The results of long-term post construction monitoring indicate the reservoir operation and the pore water pressure response of the earthfill zone upstream of the puddle core to changes in reservoir level have a significant influence on the long-term deformation behaviour. Case studies of "normal" deformation behaviour under "steady state" conditions (Figure 5.6) are consider to comprise those cases where the reservoir level is essentially maintained at a steady level (Cwnwerderi and Challocombe dams), or the pore water pressure response of the earthfill zone upstream of the puddle core is either negligible (Holmestyes) or minor (Burnhope, Hollowell, Ladybower, Selset, Yan Yean and Happy Valley dams) to changes in the reservoir level.

For these nine case studies the long-term crest settlement rate, S_{LT} , typically ranges from 0.4% to 1% settlement per log cycle of time.

Of these embankments both Ladybower and Holmestyes dams exceed an S_{LT} of 1%. For Holmestyes (S_{LT} of 2.3%) the period of monitoring of 1.5 years at 150 years after construction is considered too small to obtain a reasonable estimate, as only 3 mm of crest settlement occurred during this period. Ladybower dam (S_{LT} of 2.0%) was classified as "minor" response to reservoir level fluctuation based on the year of completion of construction (1945) and assumption that a zone of select earthfill of low permeability was used up and downstream of the puddle core, as was the general design practice used at the time. However, the embankment design details, the construction materials used and their method of placement were not known, and it is therefore possible that these assumptions are not correct for Ladybower dam.

Of the remaining seven cases considered as "steady state" conditions there is no clear correlation of the long-term crest settlement rate, S_{LT} , with embankment height or age. However, S_{LT} is be expected to be affected by:

- The age of the embankment, mainly due the difference in design and compaction techniques adopted at the time of construction. The S_{LT} of the older puddle dams would be expected to be greater than for newer dams constructed after the 1930's.
- The height of the embankment. It is not clear what effect the embankment height has because its influence appears to be relative minor and over-shadowed by other factors, as is the case with the earth and earth-rockfill embankments (Figure 4.62 and Figure 4.63). It would be expected that S_{LT} would be greater for dams of greater height.
- Pore water pressure changes in the embankment. Even though steady state conditions have been assumed, the pore water pressures in the puddle core and the zone immediately upstream would be expected to change with time. The available data does indicate some change, albeit minor, with reservoir level.
- The material type and placement method of the various zones within the embankment, and their effect on the time dependent creep coefficients. The data for rolled earthfill and earth-rockfill embankments (Section 4.3.3) indicates clayey earthfills generally have higher long-term settlement rates than dominantly sandy earthfills, and dams with poorly compacted rockfill shoulders higher long-term settlement rates than well-compacted rockfills.

Holmestyes Dam (constructed 1840) presents a slightly different case to most puddle core earthfill embankments classified as "steady state" conditions. A layer of puddle clay was placed on the upstream face of the dam in 1857 (some 17 years after construction) due to the observations of leakage (Tedd et al 1993). Installation of piezometers downstream of the upstream blanket indicated the phreatic surface in the upstream

shoulder and core to be well below the reservoir level, and on drawdown only minimal responses in piezometric level were recorded. On a large drawdown of approximately 10 m minor heave movements of the crest and upstream shoulder were observed (Tedd et al 1993).

In summary, it is considered based on the available records of the case studies analysed that under "steady state" conditions (or near steady state conditions) the expected long-term crest settlement rate, S_{LT} , for "normal" deformation of puddle core earthfill dams is less than about 1% settlement per log cycle of time. The mechanisms involved are considered to be time dependent primary creep and deformations associated with the small changes in effective stress following the small changes in pore water pressure in the puddle core and upstream shoulder.

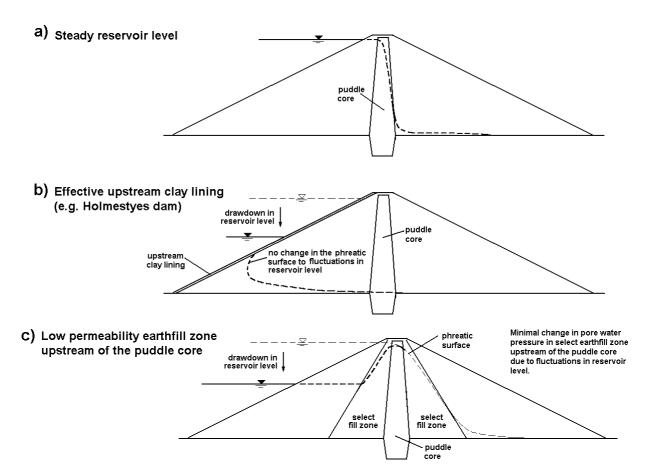


Figure 5.6: Definitions of "steady state" conditions during normal reservoir operating conditions.

5.3.4 "Normal" Deformation Under Normal Reservoir Operation and Fluctuating Phreatic Surface in the Upstream Shoulder

This section looks at case studies where large changes in pore water pressure are observed in the upstream shoulder (adjacent to the core) with fluctuations in reservoir level (Figure 5.7). In these cases cyclic changes in effective stress occur in the upstream shoulder. Cyclic changes in total stress acting on the upstream face of the puddle core also occur as hydrostatic pressure change with fluctuations in reservoir level. Long-term settlement rates of the crest and upstream slope for these cases are generally significantly greater than for "steady state" conditions as described in Section 5.3.3.

The case studies include Dean Head, Ramsden, Yateholme and Walshaw Dean dams where the long-term crest settlement rates, S_{LT} , under normal operating conditions range from 4.5 to 7.4 % per log cycle of time (Table

5.1). The rate of crest settlement of these dams is typically 5 to 10 times that under "steady state" conditions and confirms the significant influence of the permeability of the earthfill zone upstream of the puddle core under fluctuating reservoir conditions on the long-term crest settlement rate. Figure 5.5 graphically presents the difference in the long-term crest settlement rate between Ramsden dam and several "steady state" dams.

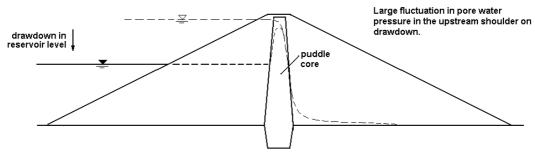


Figure 5.7: Fluctuation of pore pressures adjacent to and upstream of the puddle core with fluctuations in reservoir level.

For Hope Valley dam (S_{LT} of 2.6%) the pore water pressure response in the earthfill zone upstream of the puddle core is variable, ranging from 25% to 100% of the change in reservoir level. It has been included with this group of case studies even though it possibly lies somewhere between these case studies where large changes in pore water pressure upstream of the puddle core are observed and the "steady state" case studies. The reported long-term crest settlement rate for Hope Valley dam is from SMPs on the downstream edge of the crest, and the crest settlement rate above the puddle core is therefore possibly higher than 2.6% per log cycle time.

Ladybower dam possibly also falls into this category, however, no information was available on the pore water pressure response under drawdown or the reservoir operation to confirm or deny this.

The mechanism of deformation during the normal cyclic operation of the reservoir for embankments with permeable upstream earthfill is:

• On drawdown:

- Increasing effective vertical stresses in the upstream shoulder occur as pore water pressures dissipate relatively quickly in the permeable earthfill, resulting in settlement. Settlements are not expected to be overly high because the stress path is in the unloading / reloading range and the earthfill's compressibility will therefore be relatively low.
- The reduction in hydrostatic pressure acting on the upstream face of the puddle core results in a reduction in the total horizontal stresses within the puddle core and downstream shoulder, and upstream displacement occurs. This upstream displacement was observed for SMPs on the crest, upstream slope and upper downstream slope of Walshaw Dean and Ramsden dams (refer Appendix D) and the internal displacement of the centre of the puddle core at Ramsden dam (Figure 5.8) during drawdowns within the normal operating range for the reservoir.
- Some of the reduction in hydrostatic pressure acting on the upstream face of the puddle core is transferred onto the upstream shoulder in order to maintain equilibrium with the lateral stresses of the puddle core. This increase in horizontal stress in the upstream shoulder will result in a net upstream displacement of the upstream interface of puddle core and lateral spreading of the puddle core. Deformations within the puddle core occur primarily as undrained plastic yielding and therefore lateral spreading is accompanied by vertical strain to maintain volumetric consistency, and therefore settlement of the crest. Internal vertical strains measured in the puddle core during normal operating drawdown at Ramsden (Figure 5.9) and Walshaw Dean dams confirm this pattern of behaviour (refer Sections 2.4 and 2.5 of Appendix D).

- Vertical strains in the puddle core can also occur due to increases in effective vertical stress as a result
 of pore water pressure dissipation. The amount of pore water pressure dissipation will depend on the
 permeability of the puddle core and the time the reservoir is in a drawn down condition.
- On re-filling the stress paths are reversed and:
 - Decreases in effective vertical stresses in the upstream shoulder result in heave type deformations.
 - Increase in the horizontal stresses acting on the upstream face of the puddle core results in downstream displacement of the core, upstream slope and downstream slope.
 - Within the puddle core heave type internal strains (Figure 5.9) and heave of the crest are observed. The mechanism causing this will be a combination of reduction in lateral width of the core under the increased total stress acting (i.e. a plastic type response on re-filling) and heave due reduction in effective vertical stresses as pore water pressures increase. Deformations due to pore water pressure increase will show a time delay due to the low permeability of the core.

The net permanent vertical deformations following a reservoir cycle of normal magnitude are typically settlement of the crest, upstream slope and upper portion of the downstream slope. The magnitude of settlement is generally greatest for the crest. Net permanent crest displacements are generally small and in a downstream direction, but data records are limited. Tedd et al (1997a) attribute the net permanent settlement of the upstream shoulder to the difference in compressibility properties between unloading (lower compressibility on re-filling) and re-loading (higher compressibility on drawdown) as observed in cyclic laboratory oedometer testing (Holton 1992; Tedd et al 1997a). Kovacevic et al (1997) obtained good agreement between measured and predicted (from finite element analysis) settlements under normal reservoir drawdown cycle for several puddle core earthfill embankments when incorporating the differential compressibility between unloading and reloading in their analysis.

The net internal deformation of the puddle core at Ramsden dam shows upstream displacement (Figure 5.8) and permanent vertical strains (Figure 5.9) over the full height of the core. At Walshaw Dean dam the internal vertical strain shows a permanent increase in strain over the full height of the core (refer Section 2.5 in Appendix D). These permanent core deformations are most likely primarily due to undrained plastic deformations on drawdown.

The net permanent crest settlements (Figure 5.10) are typically 50 to 70% of the maximum settlement at drawdown, for drawdowns within the range of normal reservoir operation. Permanent settlements of greater magnitude are observed for abnormally large drawdowns, and are discussed in Section 5.3.5. The data in Figure 5.10 is mainly from puddle dams in Yorkshire, England where the shoulder earthfill is relatively permeable.

The permanent crest settlement on drawdown for puddle core earthfill dams with permeable earthfill in the upstream shoulder is likely to be affected by the magnitude of the drawdown, the length of time of the drawdown and the compressibility of the upstream filling. Ignoring the events associated with abnormally large drawdown events, Figure 5.10 shows some correlation of permanent crest settlement with drawdown height. It should be noted that the data set is relatively small and comprises UK puddle dams of similar height and constructed using similar materials for the shoulders. Therefore, the following preliminary predictive methods, which are based on the data presented by Tedd et al (1997b), should be used with caution:

- Estimation based on the S_{LT} rates of the reported cases within the data set, ranging from 4.5 to 7.4% vertical strain per log cycle of time.
- Estimation based on the plot of permanent crest settlement versus depth of drawdown (as a percentage of the embankment height) as shown in Figure 5.11. This plot is based on the data from Tedd et al (1997b) omitting those points associated with abnormally large drawdown events and normalising the drawdown height with respect to embankment height.

Where the pore water pressure response of the earthfill zone upstream of the core is in-between that of "permeable" and low permeability, such as at Hope Valley dam (and possibly Ladybower dam) the long-term settlement rate is also between that of the two categories. However, there is not sufficient data with which to provide a possible range of long-term crest settlement rate.

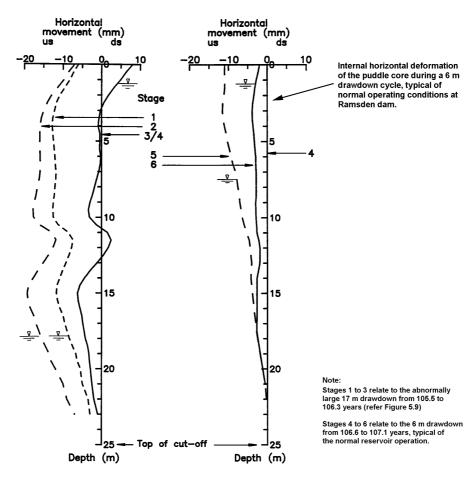


Figure 5.8: Ramsden Dam, internal horizontal deformation of the puddle core, 1988 to 1990 (Holton 1992).

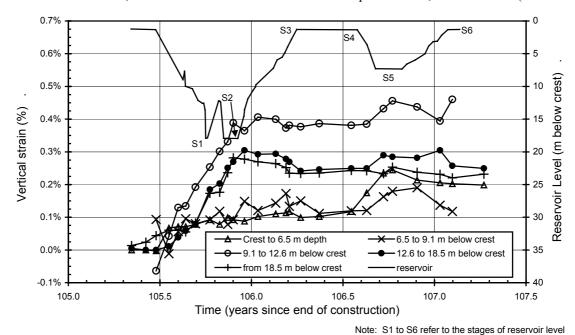
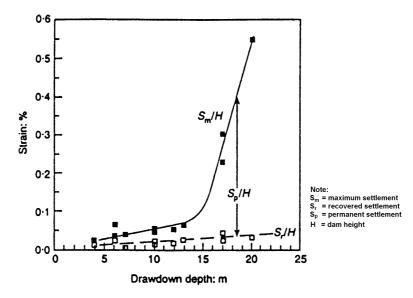


Figure 5.9: Ramsden Dam, internal vertical strains measured in the puddle core from 1988 to 1990 (data from Tedd et al 1997b and Kovacevic et al 1997).



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Figure 5.10: Vertical strain (or settlement) at the crest versus drawdown depth for specific drawdown and refilling cycles (Tedd et al 1997b).

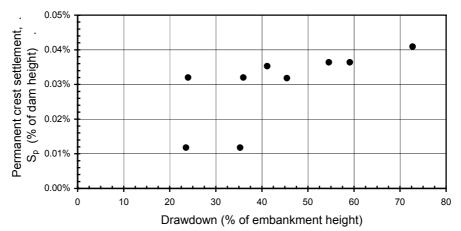


Figure 5.11: Permanent vertical crest strains during cyclic drawdown (in normal operating range) for puddle core dams with permeable upstream filling (Tedd et al 1997b).

5.3.5 Deformation During Abnormally Large Drawdown Events

Tedd et al (1997b) report the deformation behaviour during historically large drawdown events for several UK puddle core dams, including Ramsden, Ogden and Widdop dams. Permanent crest settlements under abnormally large drawdown for these three dams were:

- 51 mm at Ramsden dam (Figure 5.12), or 0.20% of the embankment height, as a result of the abnormally large drawdown cycle of 17 m at 106 years after construction. Further details are given in Section 2.4 of Appendix D.
- 130 mm at Ogden dam (Figure 5.13), or 0.52% of the embankment height, during the abnormally large 20 m drawdown event in 1990 to 1992.
- 52 mm at Widdop dam, or 0.26% of the embankment height, during the abnormally large drawdown cycle of 17 m at 110 years after construction.

The data records indicate large permanent crest settlements occur under abnormally or historically large drawdown events. At Ramsden dam a smaller magnitude, but still relatively large, permanent settlement was measured for the SMP on the upstream slope.

The initial deformation pattern on drawdown to a level commensurate with the normal operating conditions would result in the deformations as described in Section 5.3.4. On drawdown to abnormally or historically low levels drained yielding in the upstream shoulder and undrained yielding in the puddle core are considered to be the dominant mechanisms resulting in the large magnitudes of deformation observed.

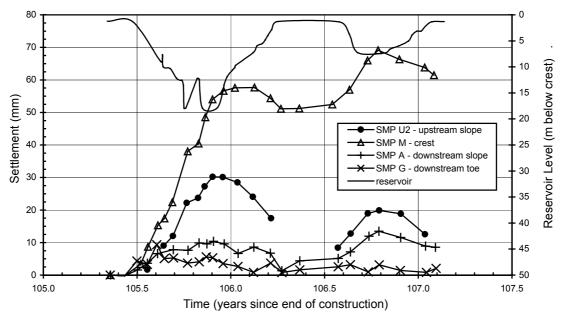


Figure 5.12: Ramsden Dam, settlement of SMPs versus time from 1988 to 1990 (data from Tedd et al 1990).

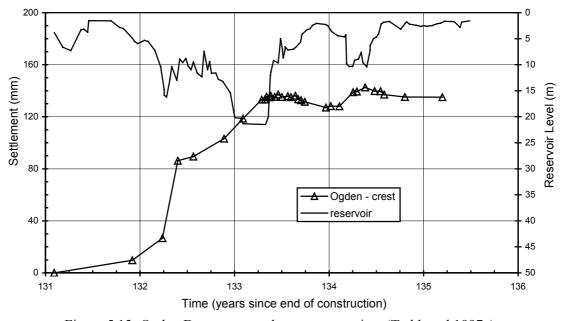


Figure 5.13: Ogden Dam, crest settlement versus time (Tedd et al 1997a).

In Section 5.2.1 it was considered that poorly compacted earthfills in the upstream shoulder, susceptible to collapse compression on wetting, were close to normally consolidated after the end of first filling due to the softening effects of collapse and wetting. Increases in effective stresses on drawdown post first filling were considered to result in large deformations due to yielding as vertical stress levels exceeded the pre-consolidation pressure in the near normally consolidated upstream earthfill. On subsequently larger drawdowns where the effective stresses in the upstream shoulder exceeded those previously experienced post first filling, further large deformation of the upstream shoulder due to yielding in drained conditions was also likely. It is on these abnormally or historically large drawdown many years after the end of construction that effective vertical stresses in the permeable upstream shoulder are considered to increase to levels greater than previously

experienced. The new high effective vertical stress levels will be experienced in the lower elevations of the upstream shoulder (Figure 5.14) and drained yielding will be confined to this region as stress levels exceed the pre-consolidation pressure and the earthfill's compressibility increases significantly above that for the recompression range of loading. In the mid to upper regions of the upstream shoulder it is likely that effective stress levels will equal those previously experienced and therefore yielding will not occur; smaller settlements are likely under the re-compression compressibility properties of the earthfill.

For the puddle core, the hydrostatic pressures acting on the upstream face of the puddle core will be reduced to levels not previously experienced. Therefore, the horizontal stress levels in the upstream shoulder required to equilibrate the lateral stress from the puddle core will exceed those levels previously experienced since first filling. The new high lateral stress levels will be in the lower region of the embankment where a net upstream displacement of the core / upstream shoulder will occur. Deformations in the puddle core will largely occur as undrained plastic lateral spreading and vertical compression (Figure 5.14). This pattern of deformation behaviour of the puddle core was observed at Ramsden dam where vertical strains in the mid to lower region were much greater than in the upper region (Figure 5.9) and the bulk of the lateral deformation was below about 15 m (Figure 5.8).

After the initial plastic deformation, consolidation type settlements within the puddle core may occur as pore water pressures dissipate. For Ramsden dam, Figure 5.8 and Figure 5.9 show significant lag in the deformation of the lower portion of the core. It is suspected that this is not only due to pore water pressure dissipation in the puddle core, but is also due to a lag in pore pressure dissipation in the upstream shoulder.

On-refilling after the historically large drawdown at Ramsden dam, a large amount of the lateral displacement of the core was recovered (Figure 5.8) as the hydrostatic pressures and horizontal stresses acting on the upstream face of the puddle core increased. However, only a limited amount of the vertical strain was recovered on refilling (Figure 5.9). At Ogden dam (Figure 5.13) only a small proportion of the maximum crest settlement (approximately 6%) was recovered on re-filling. The reasons for this are considered to be due to the large undrained plastic type deformation of the core and drained yielding in the lower upstream shoulder that is not recovered on re-filling.

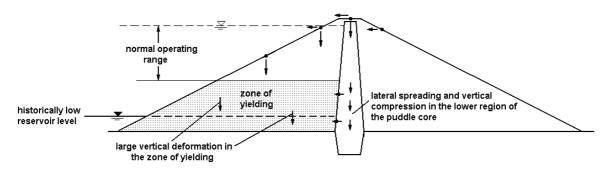


Figure 5.14: Idealised model of deformation during historically large drawdown.

Comparison of the crest settlement of Ramsden (Figure 5.12) and Ogden (Figure 5.13) dams with Walshaw Dean Dam (Figure 5.15) during a full drawdown provides some degree of verification of the mechanism discussed. As indicated, for both Ramden and Ogden dams the crest settled significantly on full drawdown, 51 mm and 130 mm respectively. In contrast, the full drawdown of Walshaw Dean dam over the period 86 to 88 years after construction resulted in a permanent crest settlement of only 9 mm, which is in line with the general settlement trend from the earlier drawdowns. The reason for the deformation behaviour at Walshaw Dean is considered to be because the normal reservoir operation is for very large seasonal drawdown, as shown in Figure 5.15, and the maximum effective stresses on full drawdown within the upstream earthfill have previously

been experienced. Therefore, the changes in stress are within the re-compression zone for which the compressibility of the earthfill is significantly lower.

In terms of prediction of deformation during abnormal drawdown events, the only available method would seem to be fully coupled finite element modelling (FEM) due to the complexity of the processes and interaction between the different elements within the embankment. Kovacevic et al (1997) attempted to model the deformation of Ramsden dam during the large drawdown event using finite element methods, but were unable to model the deformation with any accuracy. It is considered that they may not have been able to model the yielding that occurred in the core and upstream zone when the reservoir was drawn down to an all time low level. Therefore, undertaking FEM is not a simple procedure and to add to the complexity is the lack of known information on material properties and required assumptions necessary in the modelling.

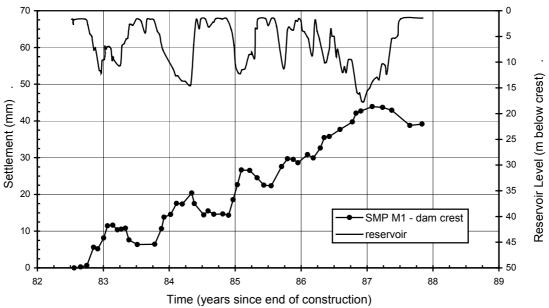


Figure 5.15: Walshaw Dean (Lower) dam, crest settlement versus time (Tedd et al 1997a).

5.3.6 Factors Affecting the Long-term Deformation Behaviour of Puddle Core Dams

In summary, from the above analysis and discussion on the "normal" deformation behaviour of puddle dams, the factors affecting the long-term deformation behaviour are considered to be:

- Under normal reservoir operating conditions the permeability of the earthfill zone upstream of the puddle core has a significant effect on the long-term deformation behaviour of puddle core earthfill embankments. Long-term crest settlement rates, S_{LT} , under "steady state" conditions (or near steady state conditions) are typically less than about 1 % per log cycle of time (settlement as a percentage of the embankment height). For embankments where the earthfill upstream of the puddle core is permeable and the seasonal reservoir fluctuation is large, the S_{LT} rates are significantly higher (2.6 to 7.5 % per log cycle of time).
- The historical operation of the reservoir also has a significant effect on the deformation behaviour. Historically large drawdown events result in large, permanent undrained plastic deformations of the puddle core, and yielding (and possibly undrained plastic deformation) of the lower portion of the upstream filling. This type of deformation behaviour could be classified as "abnormal".
- The method of placement and moisture content control of the shoulder filling, particularly, the upstream shoulder. The poorer the quality of compaction and drier the moisture content at placement, the greater the potential for collapse compression on saturation. The older (pre 1900) puddle core earthfill embankments were more prone to collapse compression.

- Primary creep (or creep under constant stress), secondary consolidation and foundation compression are encapsulated in the long-term settlement rate. Given the age of most puddle core earthfill embankments these effects are considered to be relatively minor compared with the effect of the permeability of the earthfill zone upstream of the puddle core.
- Shrink-swell movements are likely to influence those sites for which plastic clays are present and seasonal moisture content changes occur.
- Seasonal creep movements of the outer downstream slope. This is creep of the outer surface of the downstream slope due to seasonal factors such as moisture content profile changes. It is dependent on the plasticity of the soil and potential for shrink-swell related movements, the steepness of the slope and the soil strength. The effect of seasonal creep movements is usually offset in the method of installation of surface measurement points on the slopes of the embankment.

6.0 "ABNORMAL" EMBANKMENT DEFORMATION BEHAVIOUR – METHODS OF IDENTIFICATION AND CASE STUDIES

6.1 METHODS OF IDENTIFICATION OF "ABNORMAL" DEFORMATION

"Abnormal" deformation behaviour of embankment dams is deformation that cannot readily be explained as being due to "normal" mechanisms. The purpose of attempting to identify abnormalities in the deformation behaviour is for the early detection of potential problems such as instability or internal erosion. Often what may initially be termed "abnormal" may later be proven by investigation, analysis and additional monitoring to infact be "normal". Identification of "abnormal" deformation behaviour is basically from outliers to the "normal" deformations trends from similar embankment types taking into consideration the embankment zoning geometry, material types, placement methods and foundation conditions. Outliers are identified from magnitude of deformations as well as trends in the direction and rate of deformation.

During construction, identification of "abnormal" deformation is from outliers to the total settlement and vertical strain profiles in the core of earthfill and zoned earth and earth-rockfill embankments (Section 4.1.3). The data on lateral core deformations in zoned earthfill embankments with thin to medium core widths (Section 4.1.2) from the case studies is limited and therefore not sufficient for assessment of "abnormal" deformation unless measured core deformations are larger than or the trend different to the case study data presented.

Deformation of SMPs on the shoulders of embankments during construction is available for some embankments in the literature, but it has not been considered here due to inconsistencies in timing and variation in embankment type for which the data is available. Penman (1986) provides some guidelines for the magnitude of lateral displacement, but the data is from a variety of embankment types and should be used with caution.

Identification of "abnormal" deformation post construction for earthfill and zoned earth and earth-rockfill embankments is from outliers in terms of magnitude and rate of deformation of surface monitoring points on the crest and slopes of the embankment taking into consideration the zoning geometry and material properties of the core and shoulders. "Abnormal" trends in the deformation behaviour, such as increased rates of deformation on first filling, and internal localised zones of high strain in the core are other means of identification of potentially "abnormal" deformation behaviour. For puddle core earthfill dams outliers are more readily evaluated from the rate and trend of the long-term deformation behaviour given the age and period of monitoring of these structures.

It is important to note that for embankments where the deformation is considered to be "abnormal" may be, and in most instances is, explainable on consideration of the type and placement method of the earth or rockfill, or the reservoir operation. For example, embankments with high vertical core strains during construction may be related to plastic core deformations due to low undrained strength of the core and corresponding large lateral strains of the supporting poorly compacted rockfill shoulders. Whilst the core strain may be large in comparison to similar embankments and therefore identified as "abnormal", it may in-fact be "normal" given the reasons for the behaviour.

It is emphasised that the identification of a dam as exhibiting "abnormal" deformation behaviour in no way indicates there is any issue of dam safety with that dam.

In the following sections various methods of identification of "abnormal" deformation behaviour are discussed with reference to and specific discussion on those case studies within which the "abnormal" deformation was observed.

The "abnormal" deformation behaviour may be specific to one part of the embankment whilst the overall stability of the embankment is not in question. For example, movement along a shear plane in the core on a large drawdown event for a zoned earth and rockfill dam would be considered as "abnormal" behaviour of the core, but the upstream rockfill zone provides adequate overall stability of the embankment.

Where failures due to slope instability have occurred in embankment dams there is often limited, if any, information of the deformation behaviour leading up to failure. Of the embankments considered in this report only at Carsington, Belle Fourche, San Luis and Steinaker dams did a slope failure occur, and only at Carsington was there any formal monitoring from within the failed region prior to the failure. Hunter and Fell (2002b) present information on the characteristics of pre and post failure deformation behaviour from analysis of some 53 case studies of slope failures in embankment dams with discussion on the mechanism causing the deformation behaviour. Most of the data presented relates to the post failure deformation behaviour, but there is some useful information on deformation behaviour leading up to failure that is supplementary to the several case studies discussed within this report. More generally, the basic concept of the creep model under constant deviatoric stress conditions (Singh and Mitchell 1968; Mitchell 1993) is useful for assessment of the deformation behaviour leading up to a failure condition. Primary creep, or deformation at a decreasing rate with time under constant stress conditions, is applicable to "normal" type deformation behaviour. Tertiary creep, or deformation at an increasing rate with time, under constant deviatoric stress conditions is indicative of the onset to failure.

6.2 "ABNORMAL" DEFORMATION BEHAVIOUR DURING CONSTRUCTION OF EARTH AND EARTH-ROCKFILL EMBANKMENTS

From Section 4.1 the embankments highlighted as outliers to the general deformation trend of the core showed up in both the plots of vertical strain profile and of total settlement. All cases are from zoned embankments and mostly of thin to medium core widths (combined core width less than 1H to 1V) with the core placed close to or wet of Standard optimum moisture content.

The outliers to total core settlement at the end of construction (Figure 4.23 of Section 4.1.3.3) include Hirakud, Beliche, Blowering, Nurek and Tedorigawa dams. For Hirakud, Beliche, Blowering and Tedorigawa the difference in total settlement from the trendline representative of the core type and width (Table 4.6) was 3 to 10 times the standard error and clearly very much greater than the estimated mean for "normal" behaviour. For Nurek dam the difference above the mean was 2.7 times the standard error, however, at a height of 290 m Nurek is the highest dam in the data set and therefore the settlement estimate based on the trendline is not accurate.

The outliers to vertical strain profile (Figure 4.18, Figure 4.21 and Figure 4.22) include Hirakud, Beliche, Blowering, Tedorigawa, Chicoasen and possibly Maroon dams. For these embankments the vertical strain at end of construction for portions of the core zone are greater than for the majority of case studies taking into consideration core material type, moisture content at placement and core width.

6.2.1 Plastic Deformation of the Core During Construction

For Beliche, Tedorigawa, Chicoasen and Maroon dams large plastic deformation of the wet placed core is considered to be the main reason for high vertical strains and/or high total settlement at end of construction. At Nurek and Blowering dams plastic deformation of the wet placed core is also considered to be a significant factor, but other factors are also potentially significant. Beliche, Tedorigawa, Chicoasen and Nurek are all zoned earth and rockfill dams with thin cores of clayey gravels placed at about Standard optimum or wetter.

At Beliche dam (Figure 6.1) high vertical strains at end of construction, in the range 5 to 7%, were measured for a large portion of the core from 16 m depth below crest to foundation level at 55 m depth (Figure 4.21a). The vertical strains are well in excess of those measured in similar thin, wet placed clayey sand to clayey gravel cores, as well as those within thin, wet placed clay cores (Figure 4.22a) over a similar depth range. Large lateral displacements of the core, as indicated by distortion of the inclinometer tubes up and downstream of the core (Naylor et al 1997), are considered to be a significant factor in large vertical strains measured. The high compressibility of the lightly compacted, highly weathered schists and greywackes used as the inner rockfill zone is likely to have been a contributing factor to the large lateral displacements of the thin core. Another contributing factor to the potentially large lateral displacement could have been collapse compression of the upstream rockfill on saturation following partial impoundment of the reservoir to elevation 29 m (i.e. above the crest level of the upstream cofferdam) following heavy rainfalls in January 1985 prior to completion of construction. However, no significant acceleration was recorded in the vertical strain rate for the cross-arms represented in Figure 4.20b. Further details of the deformation behaviour at Beliche dam are given in Section 1.3 of Appendix B.

At Chicoasen dam (Figure 6.2), a 260 m high central core earth and rockfill dam constructed in a broad gully with near vertical abutment slopes, high vertical strains within the core at IVM I-B4 of 5.5 to 6.5% were recorded over the depth range 85 to 105 m depth below crest level (Figure 4.21a). This region of the core is located within the plastic region identified by Moreno and Alberro (1982). In comparison to Beliche and Tedorigawa dams the vertical strains at Chicoasen dam are not overly high and are isolated to a small region of the core. This is possibly reflective of smaller lateral displacements of the core due to the likely low compressibility of the well-compacted, moderately wide to wide gravelly filter / transition zones and well-compacted inner rockfill shoulders. It is notable that the region of high vertical strain in IVM I-B4 at 85 to 105 m depth below crest level (elevation 295 to 315 m) is coincident with the base of the core region where the filter / transition width is relatively thin (Figure 6.2). Further details are given in Section 1.4 of Appendix B.

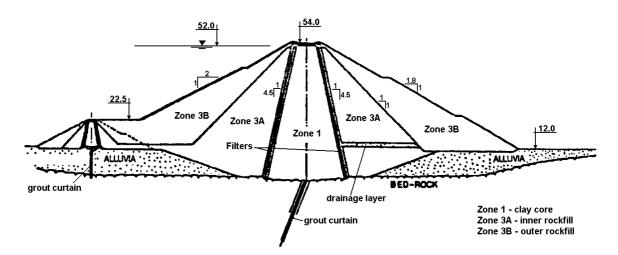


Figure 6.1: Cross section through Beliche dam at the main section (Maranha das Neves et al 1994)

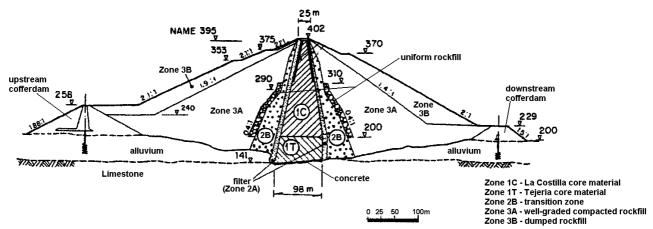


Figure 6.2: Cross section of Chicoasen dam at the main section (Moreno and Alberro 1982)

For Tedorigawa dam (Kawashima and Kanazawa 1982), a 148 m high zoned earth and rockfill dam, limited information was available from the cited references on the properties and placement methods of earth and rockfill materials. The high vertical strains at end of construction (5 to 8%) over a large section of the core (from 50 to 125 m depth below crest level) as shown in Figure 4.21a are considered indicative of plastic deformation of the core.

At Maroon dam, a central core earth and rockfill dam of 52 m height with thick core of medium plasticity sandy clays to clayey sands placed on the wet side of Standard optimum, relatively high vertical strains of 4.5 to 6.5% were measured in the depth range 26 to 33 m below crest level at end of construction (Figure 4.22b) in all three IVMs (ES1, 2 and 3). Given it is one of very few case studies in the database of zoned embankments with wet placed thick clay cores (combined core width greater than 1H to 1V) and is only marginally outside a nominal limit established from five zoned embankments with wet placed clay cores, it is difficult to really classify the vertical strain as "abnormally" high.

At Blowering dam (Figure 6.22), a 112 m high central core earth and rockfill dam with medium sized core of sandy clays placed on average from 0.2% dry to 0.3% wet of Standard optimum, vertical strains in the core at end of construction ranged from 6 to 12% (Figure 4.22b) in the depth range 32 to 76 m below crest level. This region of the core coincides with core placement on the wet side of Standard optimum. The lateral displacement ratio at 73 m depth below the crest (Figure 4.12) was estimated at 2.3% at end of construction. At the corresponding depth the vertical strain at end of construction was almost 7% indicating the lateral displacement contributed to about 33% of the measured vertical strain at this depth. The relatively low modulus of the supporting rockfill (estimated from HSG records in the rockfill) is thought to be significant in the relatively high vertical strains measured in the core. Blowering dam is discussed further in Section 6.3.3 and Appendix C.

In summary, embankments with high vertical strains within the core where plastic deformations associated with lateral spreading are considered to be significant contributing factor, have the following properties:

- Cores generally comprise clays, clayey sands and clayey gravels placed at moisture contents at or wet of Standard optimum.
- The region of high vertical strain is generally observed over a large portion of the core (e.g. Beliche, Tedorigawa and Blowering). Changes in the embankment zoning geometry or moisture placement conditions of the core may restrict the region of the core where high vertical strains are measured.
- Zoned embankments with relatively compressible rockfill shoulders, such as observed at Beliche and Blowering dams.

6.2.2 Collapse Compression of the Central Earthfill Zone

The vertical deformation behaviour of the core of Hirakud dam stands out as an outlier in terms of total settlement (Figure 4.23) and vertical strain in the mid to lower portion of the core (Figure 4.18b).

The very broad central earthfill zone at Hirakud dam (Figure 6.3) comprised mostly low to medium plasticity clayey gravels to clayey sands placed 1% to 3% dry of Standard optimum. Vertical strains in the central earthfill zone at IVM C at end of construction, installed in the deeper gully section of the embankment, reached values in the range 7.5% to 13.5% in the lower 15 m of the core (35 to 49 m) as shown in Figure 4.18b. These values are well in excess of the typical vertical strains recorded at end of construction for similar dry placed earthfills.

During the shutdown period of the 1952 monsoon season high water levels were impounded in the reservoir and backwaters engulfed the embankment section within the gully region (about 12 to 13 m height at this time) from July to December 1952 (Rao 1957). The large settlement during the 1952 shutdown period (Figure 6.4) is likely due to collapse settlement of the dry placed earthfill on saturation during inundation. Subsequent large settlements during the following two construction periods in 1953 and 1954 were predominantly within the lower 15 m of the core and likely due to the low moduli of the wetted earthfill. At end of construction the total settlement was in the order of 2170 mm, 1430 mm (or 66%) of which occurred within the lower 15 m or lower 30% of the embankment. Investigations at end of construction (Rao 1957) encountered "soft soil patches" in the lower 21 m where moisture contents were as high as 10% above Standard optimum. Further details are given in Section 3.2 of Appendix B.

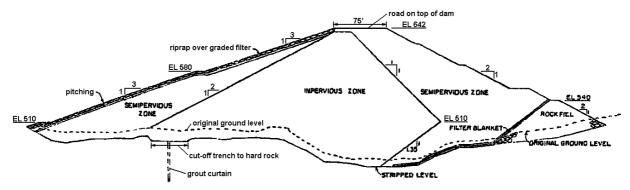


Figure 6.3: Section at Hirakud dam (Rao and Wadhwa 1958)

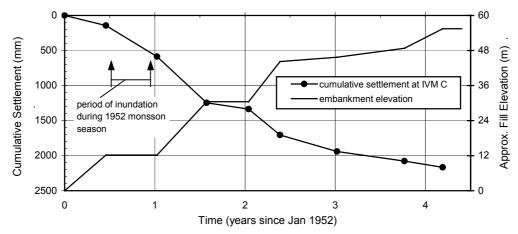


Figure 6.4: Cumulative settlement during construction at Hirakud dam (adapted from Rao 1957)

6.2.3 Reservoir Filling During Construction

Partial impoundment during construction can affect the vertical strains and total settlement of the core for zoned embankments with free draining upstream shoulders that are susceptible to collapse compression on wetting. As the monitoring records at several dams (Section 6.3.2) and results of the finite difference modelling (Section 4.2.2) indicate, embankments with wet placed cohesive earthfill cores of low undrained shear strength are susceptible to high vertical strains in the core resulting from collapse compression of the upstream rockfill on first filling.

At Nurek dam (Figure 6.5), a 290 m high zoned earth and rockfill dam with thin central core zone, the total settlement of the core during construction is considered to have been affected by the reservoir impoundment during construction. The thin, sandy clayey gravel core was likely to have been placed close to or wet of Standard optimum given the high pore water pressures developed (Sokolov et al 1985). Reservoir filling was undertaken at construction proceeded. Sokolov et al (1985) report total settlements at end of construction of 13.7 m for the core, 11.9 m for the upstream shoulder and 4 to 6.5 m for the downstream shoulder. They comment that collapse compression on wetting in the gravel to bouldery upstream shoulder fill contributed to its large settlement relative to the downstream shoulder.

The large settlement of the thin, wet placed sandy clayey gravel core is likely to have been affected by the collapse compression within the upstream shoulder. As discussed in Section 4.2.2 (Figure 4.35), collapse compression in the upstream shoulder can result in plastic deformation from lateral spreading in cores of low undrained shear strength with subsequent large vertical strains in the core.

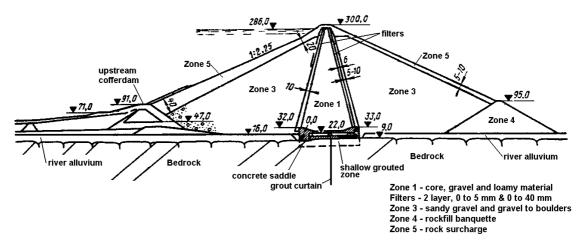


Figure 6.5: Cross section of Nurek dam (adapted from Borovoi et al 1982)

6.2.4 Shear Surface Development in the Core During Construction

High localised vertical strains in the core can be an indicator of possible development of a shear surface. The clearest example of this in the literature is probably the failure at Carsington dam (Skempton and Vaughan 1993; Rowe 1991; Potts et al 1990).

Carsington dam, a zoned earthfill embankment of 36 m maximum height, failed during construction in early June of 1984. The central core, including its unusual "boot" structure on the upstream side, comprised high plasticity clays that were placed well wet of Standard optimum moisture content and heavily rolled. The outer earthfill zones were of weathered mudstone placed in thin layers and well compacted.

The deformation records from IVM gauges installed in the central core highlighted the development of the shear surface in the core prior to the failure. At IVM C, located 4 m upstream of the axis at chainage 850 m, the vertical strain profile (Figure 6.7a) shows an increase from 7 to 9% at about elevation 180 m from early to late October 1983 during the early stages of the winter shutdown. After this date, and during the shutdown period, the tube at IVM C became constricted and then blocked at this elevation indicating the continuance of shear type deformation. This zone of high vertical strain, at about elevation 180 m, is coincident with the surface of rupture through the clay core (Figure 6.6) determined from investigation after the failure.

At IVM B, located 4 m upstream of the axis at chainage 705 m, the vertical strain profile (Figure 6.7b) shows a large increase in vertical strain between elevations 185 and 192 m in the days immediately prior to and during the development of the failure.

Figure 6.8 (from Rowe 1991) highlights the large shear strain development in IVMs B and C leading up to the failure. This type of plot of vertical strain versus bank level or height above the gauge is useful in identification of potential shear development as it highlights the development of large localised strains that are not necessarily attributable to increasing total vertical stress or to consolidation type settlements. This behaviour contrasts that for a number of the case studies from the database (Figure 4.20), which typically show a constant or decreasing rate of vertical strain with increasing fill height above the gauge.

Further details on the deformation behaviour from IVM gauges leading up to the failure at Carsington dam are given in Section 3.1 of Appendix B.

At Blowering dam the vertical strain deformation behaviour in the core during construction possibly also shows the development of a shear surface. The deformation behaviour of Blowering dam is discussed in Section 6.3.3.1 and Appendix C.

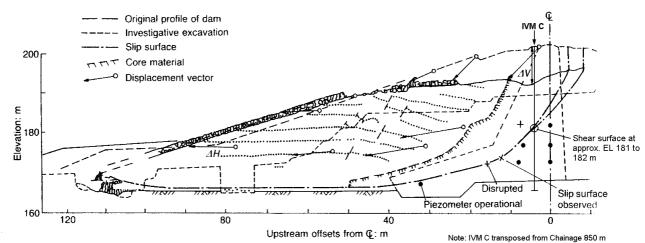


Figure 6.6: Carsington dam, section at chainage 825 m after failure (adapted from Skempton and Vaughan 1993).

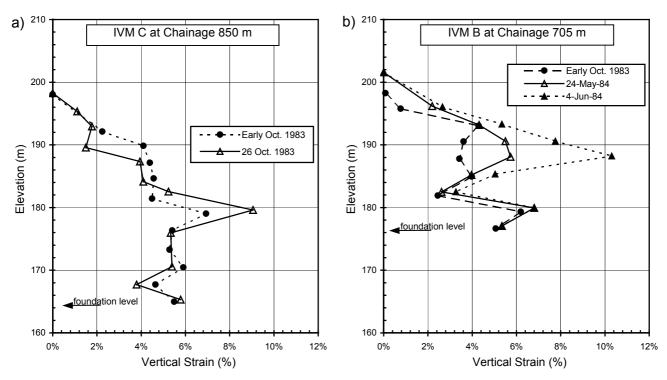


Figure 6.7: Carsington dam, vertical strain profiles from IVM during construction, (a) IVM C at chainage 850 m, and (b) IVM B at chainage 705 m (adapted from Rowe 1991).

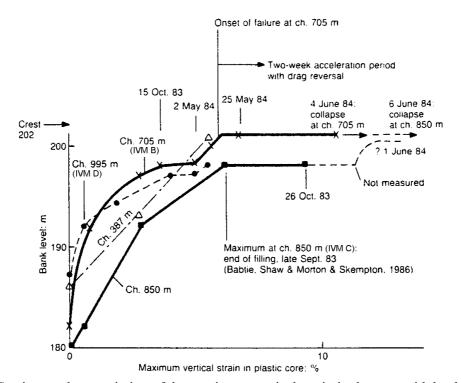


Figure 6.8: Carsington dam, variation of the maximum vertical strain in the core with bank level (Rowe 1991).

6.3 "ABNORMAL" DEFORMATION BEHAVIOUR POST CONSTRUCTION OF ZONED EARTH AND ROCKFILL EMBANKMENTS

In Sections 4.2.3 and 4.3 aspects of the post construction deformation behaviour of a large number of case studies of zoned earth and rockfill dams, mainly central core earth and rockfill dams, were labelled as "abnormal" or possibly "abnormal". Of these, the deformation at several embankments was clearly an outlier to

the general trend including Ataturk, Beliche, Svartevann, Eppalock and Djatiluhur dams. At Eppalock and Djatiluhur dams consultants to the owners (Woodward Clyde 1999; Sowers et al 1993) indicated that the upstream slope under a drawn down reservoir condition was approaching a marginal stability condition.

The other case studies where one or more aspects of the deformation behaviour were labelled as possibly "abnormal" included Gepatsch, South Holston, Glenbawn, Srinagarind, Canales, El Infiernillo, Eildon, Upper Yarra, Matahina, Blowering, Wyangala, Bellfield and Cougar dams. Others named for unusual internal deformation behaviour in the core include Copeton and La Grande 2 dams. In total this is approximately 20 of 75 zoned earth and rockfill embankments with thin to thick core widths, most of which are central core earth and rockfill dams. For virtually all but Eppalock (which has been remediated with upstream fill to improve stability) and Djatiluhur the overall stability of the embankment is not in question. The potentially "abnormal" deformation trends include:

- Accelerations in settlement rate over short periods of time of SMPs on the crest and sometimes the upstream slope, often, but not always, post first filling on drawdown. This is an attribute of quite a number of the case studies, including Eppalock and Djatiluhur and therefore potentially indicative of marginal stability conditions (refer also to San Luis and Belle Fourche dams discussed in Section 6.4.2).
- Non-recoverable displacements over short time periods post first filling (usually upstream on drawdown), a change in the direction of the displacement trend, and high rates of displacement long-term. Once again some of these trends are evident in the case studies approaching marginal stability.
- Large magnitudes of settlement or displacement compared to similar types of embankments. The direction of displacement is also a factor.
- High long-term settlement rates.

Of the case studies named, a large number incorporate rockfills that are susceptible to large deformations due to collapse compression on wetting and/or the rockfill is of high compressibility. These include rockfills that are poorly compacted, dry placed and poorly to reasonably compacted, dumped and sluiced rockfills, weathered rockfills, and rockfills of rock type susceptible to large loss in unconfined compressive strength on wetting. The implication is that the rockfills susceptible to large deformations after the end of construction can and do have a significant influence on the post construction deformation behaviour of the overall embankment. Conversely, the data indicates that, in most cases, sound rockfills that are wetted and well and reasonably to well compacted are not susceptible to large collapse compression on first filling and the overall post construction deformation behaviour of the embankment is "normal" and generally of limited magnitude.

6.3.1 Rockfill Susceptibility to Collapse Compression

Most rockfills in the upstream shoulder undergo some collapse compression on wetting during first filling. In most cases the amount of collapse compression and its effect on the overall deformation behaviour of the embankment is limited to negligible. As previously discussed (Section 4.2.2), the factors affecting the susceptibility of a rockfill to collapse compression include the method of placement (layer thickness, roller type, number of passes), moisture content at placement, effective stress level, particle size distribution, particle shape (angular versus rounded), degree of weathering of the rock, and the rock type itself (its loss in unconfined compressive strength on wetting).

Table 6.1 presents a list of embankments for which collapse compression of the rockfill resulted in moderate to large settlements of the upstream shoulder. From the data, and other information from the published literature, it is evident that:

- Dry dumped rockfills are susceptible to very large settlements due to collapse compression when wetted. At Cogswell dam the dry dumped rockfill settled up to 6% due to wetting from heavy rainfall and later sluicing (Baumann 1958).
- Dry dumped and poorly sluiced rockfills are also susceptible to large settlements when wetted. Howson (1939) describes the large settlements that occurred at Strawberry and Dix River dams on partial flooding of the dumped and poorly sluiced rockfill. At Dix River dam the settlement amounted to 0.2 to 0.3% of the height of rockfill, but was more likely closer to 1% in the flooded portion of the rockfill.
- Dry placed and poorly to reasonably compacted rockfills are susceptible to very large settlements due to collapse compression when wetted. Settlements of the upstream shoulder at Eppalock, Eildon and Gepatsch, and the downstream shoulder at Svartevann were all close to or greater than 1% on first filling. At Eildon and Eppalock dams settlements much greater than the measured 1.23% and 0.90% respectively were likely to have occurred because the monitoring missing about the first half of filling. Internal vertical strains in the rockfill at Svartevann were as much as 1.4 to 2.1% at four years after construction.
- Dumped and sluiced rockfills are susceptible to large settlements due to collapse compression when wetted. At Cherry Valley and Mud Mountain dams measured settlements of the upstream slope were close to 1%. At Watuaga, Nottely and South Holston large settlements of the upstream rockfill also occurred on first filling.
- Some weathered compacted rockfills are susceptible to very large settlements due to collapse compression when wetted, presumably if they are placed with limited quantities of water. At Beliche dam, vertical strains of up to 2.1% were measured on first filling within the "lightly compacted" rockfill of weathered schists and greywackes. At Ataturk dam the settlement of the upstream weathered rockfill was potentially very large as indicated by the very large post construction settlement of the crest (4% at almost 7 years after construction).

Table 6.1: Embankments for which collapse compression caused moderate to large settlements.

	Height	First Filling	Maximum Internal Strain (%)	Rockfill - Upstream Shoulder			
Dam Name	(m)			Туре	Compaction Rating	Water at Placement	Comment
Ataturk	184	very high		weathered basalt & limestone	reasonable (?)	2 to 6%	suspect very high because of very large settlement of the crest
Beliche	55	1.15	2.1	weathered schists & greywacke	poor	unknown	very high strains in weathered rockfill
Blowering	112	> 0.32 * ³		quartzite & phyllite	good (Zone 3A) reas (Zone 3B)	yes	monitoring missed first 50 m of filling
Canales	156	0.64		limestone	unknown	unknown	settlement of upstream edge of crest
Cherry Valley	100	1.05		granite & granodiorite	dumped	sluiced	
Copeton	113	> 0.5 * ³		granite	reas to good (Zone 3B) poor (Zone 3C)	dry	greater because cofferdam was overtopped prior to end of construction
Eildon	79	> 1.23 * ³		quartzitic sandstone	poor	dry ?	greater because reservoir within 33 m of FSL before monitoring started.
El Infiernillo	148	0.38	0.7	diorite & silicified conglomerate	poor to reasonable	dry	lower 50 to 75m saturated prior to end of construction.
Eppalock	47	> 0.90 * ³		basalt	poor	dry	monitoring missed filling of first 20 m.
Gepatsch	153	0.8 to 1.0		gneiss	reasonable	dry	some influence of the alluvial foundation
Glenbawn - main dam	76.5	0.83		limestone	poor	dry	large collapse settlement on first filling
Mud Mountain	128	1.05		andesite & tuff	dumped	sluiced	
Svartevann	129	1.0 (downstream)	1.4 to 2.1 (downstream)	granitic gneiss	reasonable	dry	settlement in downstream shoulder in first 4 years after construction.
Vatnedalsvatn	121	> 0.37 *3		quartz, granite & gneiss	reasonable	dry	suspect large, missed filling to within 10 m of FSL

Note: *1 settlement of upstream shoulder on first filling unless stated

The use of watering during placement of rockfill reduces the susceptibility of the rockfill to collapse compression as implied in the above summary. Rock type does not appear to stand out as a significant factor,

^{*2} internal vertical strains measured in the upstream rockfill during first filling, unless stated

 $[\]star^3$ > indicates settlement likely to have been greater than that stated because part of settlement on first filling was not measured

being over-shadowed by placement method and variability in the data. However, the large settlements attributed to collapse compression in the weathered rockfills at Ataturk and Beliche tends to indicate that reduction in the rock substance strength due to wetting is a significant influence on susceptibility to collapse compression.

For well and reasonably to well compacted rockfills collapse compression, in most cases, has a limited to negligible influence on the settlement of the upstream shoulder on first filling, particularly for rockfills sourced from sound rock types and watered during placement. But, for several embankments collapse compression was considered to have had some influence on the settlement of the upstream shoulder on first filling, as indicated by either the relatively large magnitude of settlement (greater than about 0.3% on first filling) and/or a small but significant differential settlement between the up and downstream shoulder (great than about 0.1% difference to upstream). Examples from the database include:

- Several dams with dry placed (and well or reasonably to well compacted) rockfill sourced from sound rock types, including LG-2, Round Butte and possibly Dartmouth and Thomson dams. These embankments are all greater than 130 m in height. Rock types varied from granitic gneiss to basalt to sedimentary sandstones and siltstones.
- Several dams with lesser quality rockfills (including rockfills sourced from weathered rocks or rock types of medium to high unconfined compressive strength, or rockfills with high fines content), including:
 - The 146 m high La Angostura zoned earth and rockfill dam. Much greater settlements were measured for the upstream shoulder on first filling (0.38% compared to 0.10 to 0.18% for the downstream shoulder). The embankment zoning geometry may have partly contributed to the greater settlement, but collapse compression in the well-compacted but poor quality (as described by Benassini et al (1976)) limestone rockfill is likely to be the main reason. Benassini et al (1976) indicate the rock used as rockfill was highly contaminated and susceptible to particle breakage on trafficking, and the rockfill as compacted was of high compressibility and low shear resistance.
 - The 90 m high Dalesice dam where much greater settlements were measured for the upstream shoulder than the downstream shoulder (0.39% compared to 0.12 to 0.18%) in the first 15 years after construction. Brousek (1976) indicates that on opening the quarry a considerable proportion of the rock material was of worse quality that originally presupposed, and might have been a factor in the greater settlement of the reasonably to well compacted upstream rockfill shoulder. The rock type and whether or not water was added during placement are not known.
 - The 112 m high Blowering dam where greater settlements on first filling were measured for the upstream than downstream shoulder (0.32% compared to 0.13%). Settlement of the upstream shoulder is likely to have been greater as post construction monitoring did not commence until the reservoir was more than 50% filled. Collapse compression within the reasonably compacted outer Zone 3B and possibly well compacted inner Zone 3A rockfill due to large loss in rock strength on saturation was considered to contribute to the greater settlement of the upstream shoulder, even though large volumes of water were used during placement. The source rock, particularly the phyllite rock used in the Zone 3B rockfill, was susceptible to large strength loss on wetting, up to 60% of its dry strength.
- At the 35 m high Glenbawn saddle dam (Saddle Dam A) relatively large settlement of both shoulders occurred during first filling, but the settlement of the upstream shoulder was greater (0.40% compared to 0.28%). Possible reasons for the likely collapse compression are not known as only limited details were gathered on the material type and placement methods of the rockfill.

6.3.2 Deformation on First Filling in Embankments where Collapse Compression occurs in the Upstream Rockfill Shoulder

The influence of collapse compression on wetting of the upstream rockfill shoulder during first filling on the deformation behaviour within zoned earth and rockfill embankments was discussed in Section 4.2.2. Two bounds of behaviour were defined:

- (i) Embankments where the core is of high undrained strength and relatively low drained compressibility, such as a partially saturated clayey core of very stiff to hard strength consistency or well compacted sandy to gravelly core. On first filling the upstream rockfill settles relative to the core and the down drag due to differential settlement results in the development of high shear stresses at the upstream core / shoulder interface as described by Squier (1970).
- (ii) Embankments with wet placed clayey cores of low undrained strength. On collapse compression in the upstream shoulder the core settles with the upstream shoulder and large differential settlements occur at the downstream core / shoulder interface (Figure 4.35). Deformations within the core are largely plastic.

The actual deformation behaviour is much more complex than the idealised simplified models, but these two bounds of behaviour are observed in a number of the case studies where collapse compression of the upstream rockfill is significant during first filling. Examples from the database are discussed below.

An important aspect of the deformation behaviour on first filling shown in the monitoring records at the well instrumented El Infiernillo dam (refer Section 6.3.2.1 below) and Chicoasen dam (refer Section 1.4 of Appendix B) is the deformation behaviour of the upstream filter zones when collapse compression occurs within the upstream rockfill. For both embankments localised regions of high vertical strain developed within the upstream filters indicating the formation of shear surfaces. The mechanism for development of the shear surfaces is considered to be due to the high shear stresses that develop within the filter as a result of the differential settlement associated with collapse compression of the upstream rockfill shoulder and shedding of load onto the relatively high modulus filters.

6.3.2.1 Collapse Compression on First Filling and its Influence on the Deformation Behaviour of Wet Placed Clayey Cores.

The following discussion presents a summary of case study evidence of wet placed clayey cores where significant collapse compression occurred within the upstream rockfill shoulder on wetting and the core largely deformed with the upstream shoulder. In several of the cases there is evidence that differential settlements were concentrated at the downstream interface of the core, and at others the internal deformation records indicate large differences in settlement between the core and downstream rockfill shoulder suggesting likely concentration at the downstream interface. The case studies discussed below include El Infiernillo, Djatiluhur, Canales and Beliche dams. A similar mechanism is considered to have occurred at La Angostura and Netzahuacoyotl dams, but data records found in the published literature were limited.

The deformation behaviour at Chicoasen dam, also discussed below, indicates that this behaviour can occur where settlements due to collapse compression of the upstream rockfill are relatively small.

(a) Deformation Behaviour at El Infiernillo Dam During First Filling.

Marsal and Ramirez de Arellano (1967), Squier (1970) and Nobari and Duncan (1972b) discuss the deformation behaviour on first filling at El Infiernillo dam. In the following summary only the vertical deformation behaviour measured from internal deformation gauges and the interaction between various zones in the embankment is discussed. Further details are presented in Section 1.9 of Appendix B.

El Infiernillo dam (Figure 6.9), constructed in the early 1960's, is a central core earth and rockfill dam of 148 m maximum height located in a narrow valley with steep abutment slopes. Well-compacted filter / transition zones are located either side of the wet placed, high plasticity sandy clay core. Rockfill, of quarried diorite and silicified conglomerate, was placed dry and track rolled by bulldozer; Zone 3A in 0.6 to 1.0 m lifts and Zone 3B in 2.0 to 2.5 m lifts.

The section of the upstream rockfill zone between the embankment and the upstream cofferdam was flooded during construction to reduce the impact of collapse compression on first filling.

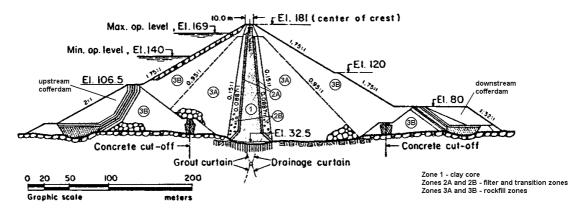


Figure 6.9: Maximum section at El Infiernillo dam (Marsal and Ramirez de Arellano 1967).

The measured internal settlements of the rockfill, filters and core at Station 0+135 at selected time intervals over the period of first filling are presented in Figure 6.11 (instrument locations are shown in Figure 6.10). The plots highlight several important aspects of the deformation behaviour:

- The settlement profiles of the core, upstream filter and upstream rockfill are similar, and are different to that of the downstream rockfill.
- At end of first filling, large and uniform vertical strains were measured in the upstream rockfill between elevation 80 and 125 to 130 m. Similarly, large vertical strains were measured in the core, but at a higher elevation, from 105 to 150 m.
- In the upstream filter (IVM I4) localised zones of high vertical strain were measured at about elevation 102 m and 132 m.
- Low vertical strains were measured below about elevation 80 m in the upstream rockfill. This is likely to be because due to the pre-saturation during construction and therefore this region is not susceptible to collapse settlement post construction.

The settlement of the core and upstream filter / transition on first filling is largely controlled by the collapse compression in the upstream rockfill. Differential settlements between the upstream rockfill and filters will occur above about elevation 80 m and, as a result of the down-drag effect, high shear stresses are developed on the upstream side of the upstream filter zone. The deformation response within the upstream filter / transition zone to these high shear stresses is the development of shear surfaces, as evidenced by the localised regions of high vertical strain within gauge D1 at elevations 102 m and 132 m.

The vertical strain in the core between elevation 105 and 150 m is almost three times the strain below elevation 105 m (average of 0.66% compared to 0.24% in February 1966). It is considered that the greater vertical strain in the mid region of the core is largely due to plastic type deformations in undrained loading as the core deforms with the upstream shoulder. The strains in the core are not localised as they are in the upstream filter because of the low undrained strength and plastic nature of the wet placed clay core.

The vertical strain profile in the downstream rockfill contrasts that of the core, upstream filter and upstream rockfill. Differential settlements between the core and downstream filter above elevation 105 m are large at more than 200 mm, and indicate a likely concentration at the downstream interface of the core.

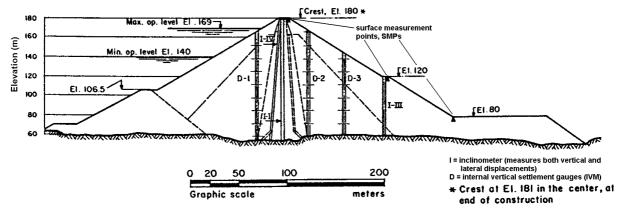


Figure 6.10: El Infiernillo dam, deformation instrumentation on station 0+135 at the lower left abutment (Marsal and Ramirez de Arellano 1967).

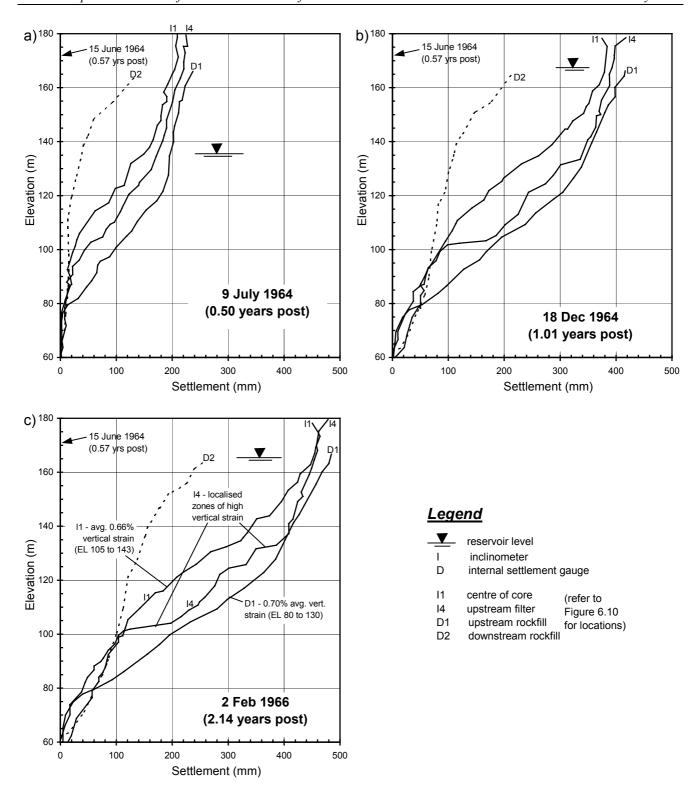
(b) Deformation Behaviour at Chicoasen Dam During First Filling (Section 1.4 of Appendix B).

Chicoasen dam (Figure 6.12), constructed in the late 1970's, is a central core earth and rockfill dam of 261 m maximum height located in a narrow valley with near vertical abutment slopes. The core comprises a well-compacted clayey gravel placed at close to Standard optimum moisture content. Filter / transition zones of well-compacted gravels are located either side of the narrow core and the main rockfill zone (Zone 3A) consists of well-compacted quarried limestone.

The embankment was well instrumented at the main section (Figure 6.13) with numerous inclinometers and cross-arms installed in the core, filters and rockfill zones. Moreno and Alberro (1982) present a selection of the instrumented deformation behaviour on first filling and a summary of the vertical deformation is presented below.

The internal vertical settlement profiles for the period of first filling are presented Figure 6.14. Note that the dates of the monitoring period are different between gauges, but cover most of the period of first filling from 1 May 1980 to late July 1980. As shown, a region of high vertical strain developed in the upstream rockfill between elevations 260 m and 300 m (strains of 0.4 to 0.5% in IVM I-A5 and I-A6), possibly due to collapse compression on wetting in the well-compacted limestone rockfill, although Moreno and Alberro (1982) comment that it may be due to plastic type behaviour on reduction in effective horizontal stresses.

Vertical strains in the upstream filter zones are much greater above elevation 270 m and are most likely in response to the higher stresses developed within the filter as a result of differential settlement following collapse compression in the upstream rockfill. In the Zone 2A filter (IVM D5) localised concentrated zones of vertical deformation were evident at elevations 285 m and 317 m, and are likely to be due to shear type deformations. The vertical settlement profile in the core shows that vertical strains are much greater above about elevation 290 m (0.5% to 0.75% compared to 0.3% below elevation 290 m), which is above the base elevation of high strains in the upstream rockfill and upstream filters. The broad zone over which high vertical strains developed in the core suggests it is largely due to plastic type deformations.



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Figure 6.11: El Infiernillo dam, internal settlements during first filling at Station 0+135.

The deformation behaviour suggests that the collapse type deformation of the upstream rockfill has a controlling influence on the deformation in the upstream filter / transition zones and core.

Moreno and Alberro (1982) comment that no concentration of deformation was measured in the downstream rockfill and it is assumed that the deformations would have been relatively small. Given the high vertical strains in the core it is likely that differential settlements were concentrated between the core and downstream shoulder, but Moreno and Alberro (1982) give no indication that cracking or differential vertical displacements were evident at the crest.

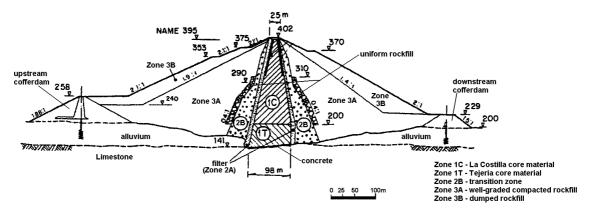


Figure 6.12: Main section at Chicoasen dam (Moreno and Alberro 1982).

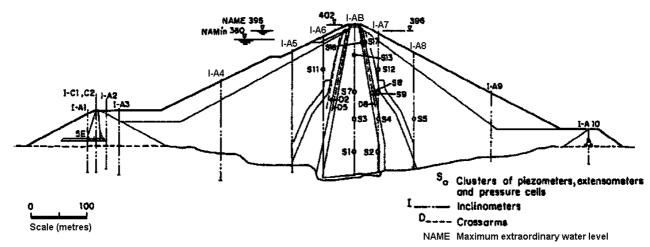


Figure 6.13: Chicoasen dam, inclinometer and cross-arm locations at the main section (Moreno and Alberro 1982).

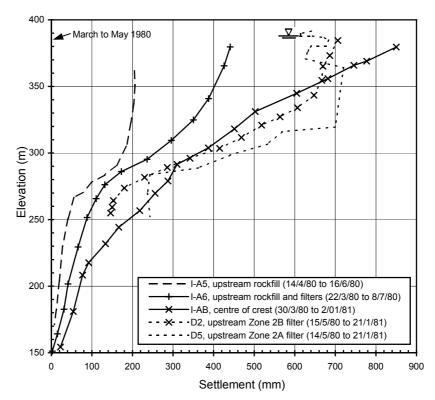


Figure 6.14: Internal settlement profiles during first filling at Chicoasen dam (adapted from Moreno and Alberro 1982).

(c) Deformation During First Filling at Beliche Dam (Section 1.3 of Appendix B)

Beliche dam (Figure 6.15), Portugal is a central core earth and rockfill embankment of 55 m maximum height that was completed in 1986. The compacted earthfill core of clayey sandy gravels was placed at moisture contents close to Standard optimum moisture content. Naylor et al (1997) indicate that the inner rockfill zone (Zone 3A) of highly weathered and fractured schists and greywackes was placed in 1.0 m layers, "relatively lightly compacted" and comprised a "significant proportion" of fines. The outer rockfill zone (Zone 3B) was of "good quality" greywackes placed in 1.0 m layers and also "relatively lightly compacted". Water is indicated as being added to the rockfill, but in what proportion or to which zones is not clear.

During construction the reservoir level exceeded the height of the upstream cofferdam and saturated the upstream rockfill to elevation 29 m.

The internal vertical settlement profiles of the core and up and downstream rockfill shoulders during the period of first filling after construction are shown in Figure 6.16. Very large vertical strains (average of 2.1%) were measured on first filling above elevation 27 m in the inner upstream rockfill zone of poorly compacted weathered rock, largely due to collapse compression on wetting. Large vertical strains were not measured below about elevation 27 m because the upstream rockfill was saturated to this elevation during construction. The vertical settlement profile in the core is similar to that of the inner upstream rockfill shoulder. Relatively low vertical strains (average 0.7%) were measured below about elevation 29 m and very large vertical strains above elevation 29 m, particularly in the upper 10 to 12 m of the core where they averaged 3.2%. The deformation of the wet placed clayey gravel core is considered to be largely due to undrained plastic type deformation, and to be largely controlled by collapse compression in the upstream rockfill.

Relatively large settlements also occurred in the downstream rockfill shoulder over the monitored period, possibly due to rainfall induced collapse compression.

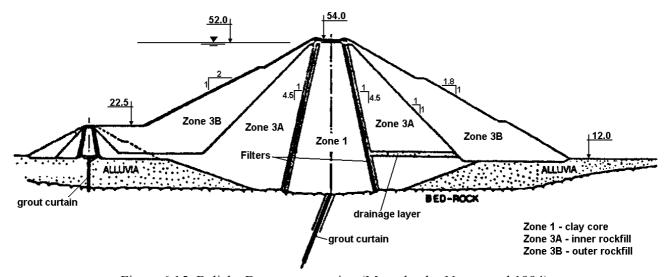
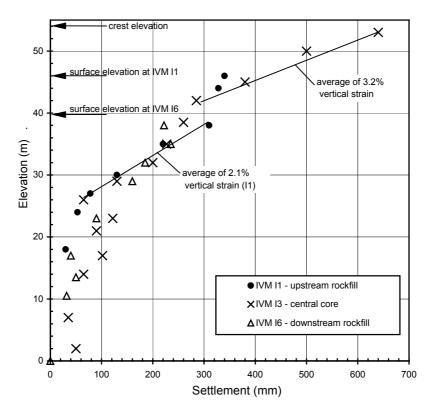


Figure 6.15: Beliche Dam, cross section (Maranha das Neves et al 1994)



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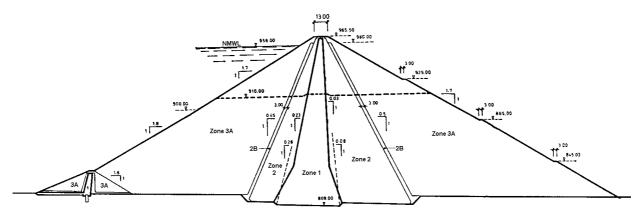
Figure 6.16: Beliche dam, post construction internal vertical settlement profile within the embankment for the period from end of construction to 2.75 years post construction.

(d) Deformation During First Filling at Canales Dam (Section 2.1 of Appendix B)

The deformation behaviour on first filling at Canales dam is a clear example of the development of differential settlement at the downstream interface of the core. Canales dam (Figure 6.17) in southern Spain is a zoned earth and rockfill embankment constructed in a narrow, steep sided valley. The embankment is of 156 m maximum height and was constructed in two stages; the first stage of 100 m (to elevation 910 m) from 1979 to 1981, and the second stage to crest level from 1985 to 1986.

The thin core (Bravo 1979) consisted of high plasticity silty clays placed at moisture contents on the wet side of Standard optimum (from OMC to 2% wet of OMC). Very broad transition zones of clayey to silty gravelly sand are located either side of the core and the rockfill shoulders are of quarried limestone. Bravo (1979) indicates the embankment materials were compacted to "the highest possible density", but it is not known what layer thickness was used or if water was added during placement.

During the period of first filling a substantial longitudinal crack developed in the crest between the core and downstream transition (Figure 6.18a). Giron (1997) indicates that the crack was first observed in 1989 (2 years after end of construction) when the recorded differential settlement of the crest was about 200 mm (Figure 6.18b). On reservoir raising to elevation 930 m at 3.8 years post construction a vertical slump had clearly developed across the downstream core / shoulder interface with a differential settlement of 405 mm. On reservoir raising to full supply level (elevation 958 m) for the first time, the rate of settlement of the mid to upstream portion of the crest increased significantly and differential settlement across the crack increased to about 1000 mm. Giron (1997) and Bravo et al (1994) attribute the crest settlement behaviour to collapse settlement of the upstream shoulder fill on saturation.



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Figure 6.17: Typical section of Canales dam (Bravo 1979).

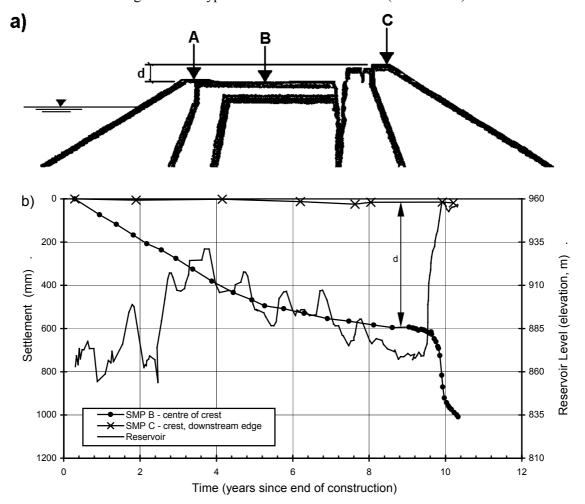


Figure 6.18: Canales dam, (a) cracking and differential settlement at the crest, and (b) post construction settlement at the crest (Giron 1997)

Several possibilities could explain the observed deformation behaviour:

- The low undrained strength of the wet placed, high plasticity clay core and consequent plastic deformation as the core deforms with the upstream shoulder as it settles due to collapse compression.
- A shear type movement in the core, along a defined plane of shearing with backscarp at the downstream core / transition interface.
- A combination of the above.

The first explanation is considered more feasible than the second mainly because the period of rapid crest settlement (after 9.6 years) occurs when the collapse settlement of the upstream rockfill is localised to the upper

20 to 30 m of the upstream shoulder, and when the reservoir level is close to full supply level where it would provide a high level of support to the upstream face of the core. For this reason, shear type movements on a preexisting shear surface in the core seem less likely than plastic deformation of a wet placed high plasticity clay core. However, a combination of both is considered possible.

(e) Deformation at Djatiluhur Dam (Section 1.7 of Appendix B)

Aspects of the deformation behaviour and observations during and after construction at Djatiluhur dam in Indonesia indicate that the core largely deforms with the upstream shoulder and large differential settlements occur at the downstream interface of the core.

Diatiluhur dam (Figure 6.19) is a central core earth and rockfill dam of 105 m maximum height constructed in the early to mid 1960's. The thin core was of high plasticity clays derived from weathered claystone, placed at moisture contents on the wet side of Standard optimum. Investigations after construction indicated the core to be of low undrained strength, particularly above elevation 65 m (Sowers et al 1993). Details on the placement methods of the rockfill, sourced from quarried andesite, are not precisely known but are thought to include both roller compaction and placement without formal compaction in layer thicknesses ranging from 0.5 m up to 2 m (Farhi and Hamon 1967; Sherard 1973; Sowers et al 1993). Farhi and Hamon (1967) comment that most of the rockfill in the mid to lower elevation was well sluiced (300% water by volume), and in the upper section the rockfill was placed in 1 to 2 m lifts, 30% by volume water added and trafficked by trucks and bulldozer.

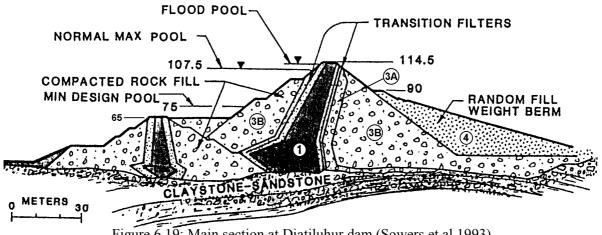


Figure 6.19: Main section at Djatiluhur dam (Sowers et al 1993)

Because first filling largely occurred during the period of construction it is difficult to gauge the magnitude of influence of collapse compression on the settlement of the upstream rockfill. But, important aspects of the deformation behaviour were revealed from monitoring during a shutdown period in construction and from test pits excavated after construction as described by Sherard (1973).

In early January 1965 construction was halted when the embankment had reached elevation 103 m. As described by Sherard (1973), shortly after construction was stopped a longitudinal crack appeared at the boundary between the core and downstream filter (Figure 6.20a), reaching a total length of some 500 m. Monitoring points were then established at elevation 103 m and the measured deformation records (Figure 6.20b) showed much larger settlements of the core compared to the rockfill shoulders, and limited settlement of the downstream shoulder. Differential displacements indicated lateral spreading of the core amounted to some 400 mm over the period January to July 1965. On raising the embankment to design level in August 1965 the core settlement at elevation 103 m totalled some 800 mm, well in excess of that measured on the downstream

slope at a similar elevation, and relatively large settlements were also recorded on the upstream slope at elevation 100 m of about 400 mm over this period.

The very large settlements of the core relative to the shoulders, particularly the downstream shoulder, are considered to be largely due to undrained plastic type deformations from lateral spreading of the core. The large differential settlements between the core and both the upstream and downstream shoulder indicates that arching or stress transfer from the core to the shoulders would have occurred. The low vertical stresses within the core were confirmed by water pressure testing after construction (Sherard 1973), which showed that under relatively low water pressures the horizontal cracks present in the upper region of the core would opened up resulting in high rates of water leakage.

Soon after embankment construction was completed a longitudinal crack (300 m in length and 25 to 40 mm in width) developed on the crest. Several deep test pits were excavated within the core to investigate the cracking, and Sherard (1973) describes the findings. An important observation was that the no cracks were visibly evident in the pit excavated within the upstream portion of the core, yet numerous horizontal cracks were exposed in the pit excavated in the downstream portion of the core. The greater number of horizontal cracks observed in the downstream portion of the core is considered to indicate that differential settlements at the downstream interface of the core were large. This suggests that, whilst lateral spreading is contributing to the greater settlement of the core, the deformation of the core is also controlled to some extent by the settlement of the upstream shoulder. The upstream orientation of the core would also contribute to the large differential settlement with the downstream shoulder.

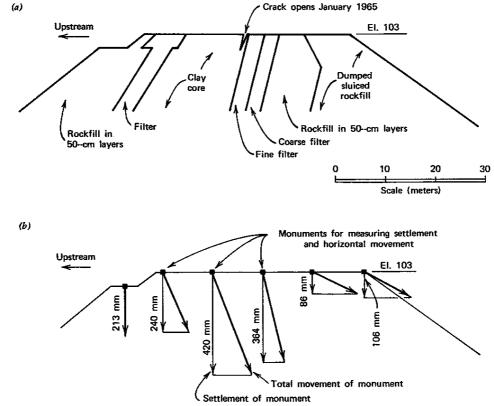


Figure 6.20: Djatiluhur dam; (a) location of crack observed during construction, January 1965; and (b) deformation of monuments at elevation 103 m, January to April 1965 (Sherard 1973).

6.3.2.2 Collapse Compression on First Filling and its Influence on the Deformation Behaviour of Cores of High Undrained Strength and Low Compressibility.

The deformation behaviour during first filling where the settlement of the upstream shoulder (due largely to collapse compression) is much greater than that of the core is observed for a number of case studies within the database. The core types for these embankments include:

- Dry placed and well-compacted clay cores of medium to thick width. Examples include Eppalock and Eildon dams. At Eppalock dam very large settlements of the upstream slope occurred on first filling compared to much smaller settlements of the crest (Figure 6.21), close to five times smaller when compared on a percentage height basis.
- Embankments with well-compacted silty sand to silty gravel cores. Examples include Cherry Valley, Cougar, Round Butte, Mud Mountain and LG-2 dams, which range in core width from thin to thick.
- Embankments with well-compacted clayey cores of thick width placed at close to Standard optimum moisture content. This would include the series of dams owned by the Tennessee Valley Authority including South Holston, Watuaga and Nottely (Leonard and Raine 1958).

For the silty sand to silty gravel cores, the magnitude of crest settlements (as a percentage of dam height) on first filling generally decreased with increasing core width. For the clayey cores, the magnitude of crest settlement was greater for the core placed close to Standard optimum than for cores placed dry of Standard optimum, possibly reflecting the likely lower undrained strength of the wetter placed cores and a larger component of plastic type deformation.

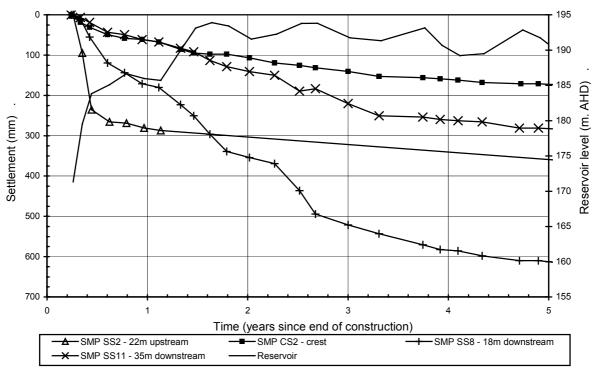


Figure 6.21: Eppalock dam, post construction settlement of SMPs on the crest and slopes for the first five years after construction.

The differential settlement between the upstream shoulder, due to collapse compression on wetting, and the core usually results in the observation of longitudinal cracking on the crest and visibly greater settlement of the upstream edge of the crest. But, this is dependent to some extent on the embankment zoning geometry. In some embankments the surface expression of the differential settlement may be masked within the riprap zone on the upper upstream slope. Examples of the surface expression of differential settlement on first filling are:

- At the 100 m high Cherry Valley dam (Lloyd et al 1958) longitudinal cracking with differential settlement across the crack was observed along the crest at the junction between the core and transition at both the upstream and downstream edges, as idealised in Figure 4.30. On first filling the amount of cracking was greater on the upstream edge of the crest.
- On first filling at the 159 m high Cougar dam (Pope 1967) longitudinal cracks with vertical offset along the crack were located at the upstream and downstream edges of the core. For both crack locations, the vertical settlement on the upstream side of the crack was greater, possibly indicating some localised shear development in the narrow core (refer Section 6.3.5, item b).
- At the 160 m high La Grande No. 2 dam (Paré 1984) longitudinal cracking over a length of 350 m was observed along the upstream edge and centre of the crest. Visibly greater settlements up to 300 mm were evident along the upstream edge of the crest. The differential settlement to upstream across the crack in the centre of the core reached a maximum of 500 mm and Paré (1984) indicates the crack to be associated with a shear surface developed within the core.
- At the 128 m high Mud Mountain dam longitudinal cracking was observed along the crest. Cary (1958) comments that the cracking was associated with the differential settlement between the dumped and sluiced rockfill and the well-compacted core.
- On first filling at the 134 m high Round Butte dam longitudinal crest cracking developed in the centre of the crest above the core to a maximum length of 150 m and width of 15 mm (Patrick 1967). There was also some indication of differential movement between the core and upstream transition due to differential settlement and lateral spreading.
- At the 97 m high Watuaga dam longitudinal cracking was observed on the edge of the crest coincident with the upstream and downstream edges of the core due to the greater settlement of the rockfill than the core, approximately 150 to 200 mm greater (Leonard and Raine 1958).
- At the 56 m high Nottely dam Leonard and Raine (1958) describe the longitudinal cracking on the upstream side of the crest coincident with the upstream edge of the core as severe. The vertical difference in settlement at the crest was greater than 250 mm.

Much of the observed longitudinal crest cracking on first filling occurs along the upstream edge of the crest, or along both edges indicating both the upstream and downstream shoulders settle relative to the core. In several cases, particularly at LG-2, the crack development was indicative of shear type deformation in the core. This is discussed further in Section 6.3.3.

6.3.3 Development of Shear Surfaces Within the Earthfill Core

Included in the database are a number of zoned earth and rockfill embankments for which shear surfaces (at least one, possibly multiple) are known to or thought to have developed in the earthfill core. The timing of the initial shear surface development can be during construction (refer Section 6.2.4), on first filling (e.g. LG-2 dam) or post first filling. Case studies where shear deformation was evident or considered to have most likely occurred are discussed in the following sub-sections. The summary is generally brief and further details are given in Appendix B for most case studies.

For a number of these embankments, it is "abnormal" trends in the deformation behaviour post first filling for which the a case study has often been identified as clearly "abnormal" or potentially "abnormal" compared to other case studies, and in a number of cases has involved shear type deformations in the core. These trends are summarised in the case study discussions.

6.3.3.1 Shear Surface Development in the Core on First Filling

The deformation behaviour on first filling at LG-2, Ataturk and Copeton dams is considered to be indicative of shear type development in the core of the embankment during first filling. At Blowering dam a shear surface is thought to have initially developed in the core during the latter stages of construction, with further movements along the shear surface on first filling.

(a) Blowering dam (Appendix C)

A detailed analysis of the deformation behaviour at Blowering dam is presented in Appendix C. A summary of the findings is presented here.

Blowering dam (Figure 6.22) is a 112 m high central core earth and rockfill embankment that was constructed in the mid to late 1960's. The medium width core was of well-compacted medium plasticity clayey sands to sandy clays placed at moisture contents ranging from slightly dry to slightly wet of Standard optimum (see below). Either side of the core the filter / transition zones were of well-compacted gravels. The rockfill comprised slightly weathered to fresh quarried phyllite, meta-siltstone and quartzite. Placement of the weaker phyllites was limited to the outer Zone 3B. The rock types were susceptible to large loss in unconfined compressive strength when wetted (35 to 62% reduction) and as a consequence high volumes of water were used during placement to offset as far as practicable collapse type settlements post construction. The Zone 3A rockfill was placed in 0.9 layers and the Zone 3B in 1.8 m layers, and compacted by 4 passes of an 8.1 tonne smooth drum vibratory roller.

The moisture content specification for the core was adjusted at various stages during construction as follows:

- Initially the specification was for placement in the range 1.3% dry to 0.7% wet of Standard optimum moisture content (OMC), and averaged 0.3% dry.
- When the embankment height was about 33 to 37 m the specification was adjusted to 0.7% dry to 1.3% wet of OMC, and averaged 0.3% wet.
- When at about 67 to 74 m the specification was adjusted to 1.0% dry to 1.0% wet of OMC, and averaged 0.1% wet.

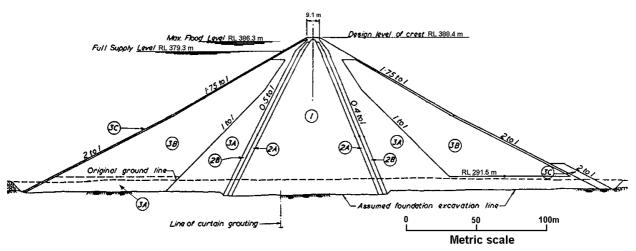


Figure 6.22: Main section at Blowering dam (courtesy of NSW Department of Public Works and Services, Dams and Civil Section).

During the latter stages of construction very high vertical strains were measured within the core at 60 to 71 m depth below crest level (between cross-arms 13 and 14 in IVM A) as shown in Figure 6.23. During construction of the last 17.5 m to crest level the vertical strain at this depth range increased from 4.3% to 11.9%, which was far in excess of the magnitude of strain at other cross-arm intervals in the wet placed region of the core. In addition, the stress-strain trend for this cross-arm interval is "abnormal" when compared to that observed in

other embankments (Figure 4.20). Development of a shear surface within the core was considered a possible explanation for the deformation behaviour.

In the early stages of first filling shortly after the end of construction very high vertical strains were concentrated between cross-arms 13 and 15 (Figure 6.24). A constriction or kink in the inclinometer tube developed below cross-arm 14 shortly after construction and several months after the start of first filling, when the reservoir level was still 30 to 35 m below full supply level, the measuring torpedo became blocked between cross-arms 13 and 14. The deformation behaviour was considered to indicate further movement on the shear surface in the core. Collapse compression of the upstream rockfill on first filling was thought to trigger the addition deformation, either due to development of high shear stresses at the upstream interface between the filters and rockfill as a result of differential settlement, or due to reduction in the lateral stresses acting on the upstream face of the core.

In 1982/83 (14 to 15 years after construction) the reservoir was subjected to a large drawdown of 57 m to an elevation more than 70 m below full supply level. This was the largest drawdown in the dam's history. On drawdown, acceleration in the rate of settlement of SMPs on the upstream shoulder and crest was measured, with a crest settlement for the period of about 60 to 80 mm. The internal vertical settlement of the core either side of the drawdown (only the upper 17 cross-arm intervals could be measured due to the earlier constriction) indicated that the upper 55 m of the embankment virtually settled as a block (i.e. only 3 mm cumulative settlement in the upper 55 m), indicating most of the settlement on drawdown occurred in the lower 50 to 55 m of the core. It is possible that most of this settlement represents shear type deformation on the existing shear surface reactivated on large drawdown. The trigger for the movement was possibly the reduction in lateral stress acting on the upstream face of the core on drawdown.

Further acceleration in the rate of settlement of SMPs on the upstream slope occurred during the next large drawdown of 53 m in 1997/98 (29 to 30 years after construction).

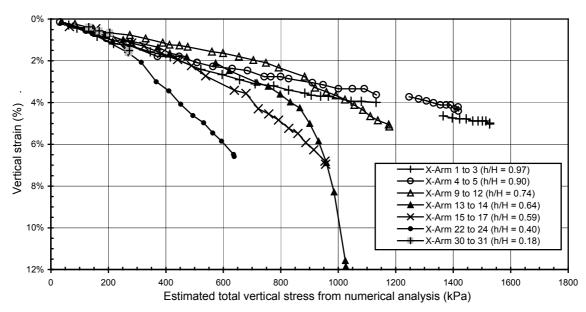
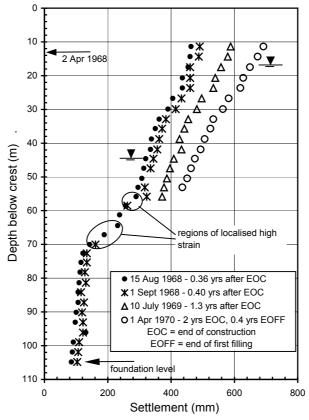


Figure 6.23: Blowering dam, vertical strain during construction for selected cross-arms intervals in the core.



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Figure 6.24: Blowering dam, post construction internal settlement of the core from IVM A during first filling.

(b) Ataturk dam (Section 1.1 of Appendix B)

Ataturk dam, Turkey (Figure 6.25) is a 184 m high central core earth and rockfill dam constructed in the late 1980's. The medium width central core is of high plasticity, reasonably to well compacted clays to sandy clays placed at moisture contents on average 1.5% dry of Standard optimum. Rockfill was placed in 0.6 m to 1.5 m layers and compacted by vibratory rollers (Cetin 2002), but no specific details are available for each zone. Moisture contents at placement were in the range 2 to 6%. The inner upstream rockfill consisted of weathered, vesicular basalt and outer upstream rockfill of sound basalt. The downstream shoulder consisted of sound basalt and an encapsulated zone of pliacated limestone having a sand to clay sized fraction of 50%. Poor quality materials were therefore used for the inner rockfill zones up and downstream of the core.

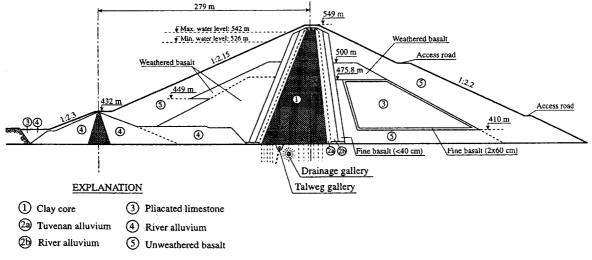


Figure 6.25: Cross section of Ataturk dam (Cetin et al 2000)

The post construction crest settlement at Ataturk dam was of very large magnitude. More than 7 metres settlement was measured in less than 7 years (close to 4% of the dam height) and clearly stands out as "abnormal" in comparison to similar type embankments (Figure 4.46a and Figure 4.53a).

Cetin et al (2000) indicate that large settlements occurred in June to December 1990 and again in early 1992 (1.5 to 2 years post construction) during periods of relatively rapid rise in reservoir level to new high levels. In the early stages of reservoir filling in June 1990, several months before the end of construction, a number of internal monitoring gauges in the lower elevations were lost. Cetin et al (2000) also refers to "landslides" occurring in the upstream slope in May 1992. It is possible that they are referring to the surface expression of differential settlement between the upstream shoulder and core. During reconstruction of the upper 6 to 7 m of the crest in 1997 slickensided surfaces were observed in the core at close to the interface between the core and downstream filters.

Cetin et al (2000) considered slaking of the vesicular basalt in the upstream shoulder and poor placement of the core to be significant factors in the very large settlement of the crest post construction. Degradation of the basalt has since been discounted as a possible cause of the large deformation (Riemer 2001).

It is difficult to surmise the potential cause/s and mechanics controlling the deformation behaviour of the embankment given the limited information available. Notwithstanding this, it is suspected that collapse compression of the upstream rockfill on wetting is likely to be a significant factor. On first filling it is suspected that very large settlements occurred in the upstream shoulder due to collapse compression on wetting, the weathered basalt rockfill possibly being particularly susceptible. The observations of differential settlement between the upstream shoulder and core, and the very large magnitude of settlement of the crest tend to confirm this.

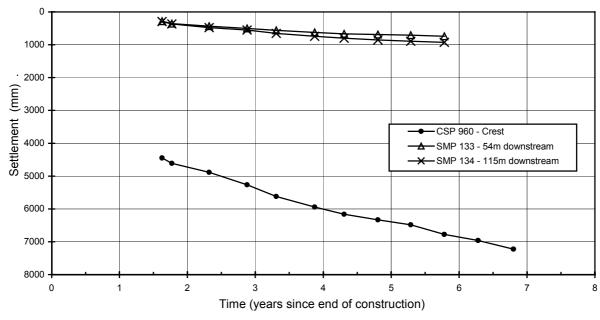
The most likely explanation for the very large settlement of the crest, large differential settlement between the crest and downstream shoulder (Figure 6.26), and observation of slickensided surfaces in the core at close to its downstream interface is considered to be the development of a shear surface and shear deformations in the core toward upstream. The available information would suggest that a shear surface (or surfaces) formed within the core during a rising reservoir condition in the early stages of first filling and prior to the end of construction, and that further shear type deformations occurred during the early part of 1992 on a rising reservoir. It is possible that localised instability developed in the core due to high shear stresses at the upstream interface as a result of the collapse compression of the upstream rockfill on wetting.

How extensive the surface of rupture might be is not known, but the loss of internal instruments in the lower elevations of the embankment may be related to shear displacements, indicating the shear surface (or surfaces) is at depth.

(c) La Grande No. 2 dam (Section 1.12 of Appendix B)

The LG-2 dam in Quebec is a 160 m high central core earth and rockfill dam, with the core slightly inclined to upstream, that was completed in October 1978. The thin core is of well-compacted non-plastic gravelly silty sand moraine deposits and supported by moderately wide and well-compacted filter / transition zones of gravelly sands to sandy gravels. The rockfill shoulders are of quarried granitic gneiss dry placed in 0.9 to 1.8 m thick layers and compacted with 4 passes of a 9 tonne smooth drum vibratory roller.

The reservoir was filled to close to maximum water level over the period October 1978 to December 1979 (0.1 to 1.2 years after construction). Since first filling the reservoir operation is not known.



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Figure 6.26: Post construction settlement of the crest and downstream shoulder at Ataturk dam.

Extensive cracking of the crest, described by Paré (1984), occurred during the latter stages of first filling. One of the longitudinal cracks was located close to the centreline of the crest, across which differential settlement to upstream was about 500 mm after first filling (Figure 6.27). Investigation undertaken after first filling found that this crack was near vertical and in the order of 150 to 200 mm wide decreasing to 50 mm wide at 3.5 m depth. Paré (1984) also refers to a "sharp tilt" that developed in September 1980 within an inclinometer located in the upstream portion of the core (Figure 6.27), which became blocked at about 18 m depth in November 1980. The timing of these observations is 1.9 to 2.2 years after end of construction, almost 1 year after completion of first filling.

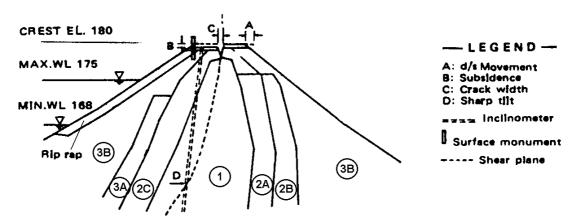


Figure 6.27: LG-2 dam; crack in crest and possible shear plane in core (Paré 1984)

Paré (1984) considered the localised straining in the inclinometer as development of a shear plane in the core. He attributes the longitudinal cracking and shear formation to a combination of the large downstream displacement of the core on first filling and collapse settlement of the upstream rockfill on wetting.

An interesting aspect of the deformation behaviour is the timing of the "sharp tilt" and blockage in the inclinometer. Paré (1984) indicates that this occurred almost 1 year after completion of first filling. But, the shear surface in the core developed during the latter stages of first filling as indicated by the timing of the crack and the large settlement (close to 600 mm) of the upstream edge of the crest on first filling. It would have been

expected that some indication of the shear formation would have been identified in the deformation of the inclinometer during first filling, however, there is no indication from Paré (1984) that any tilt was recorded during first filling. Maybe the inclinometer was not installed until after first filling when a potential slip surface was identified.

The likely mechanism of the initial shear formation in the core is considered to be a result of high shear stresses on the upstream interface of the core that developed due to the differential settlement between the core and the upstream rockfill shoulder. The observation of further shear deformation from September to November 1980 is possibly drawdown related and due to the reduction in lateral support on the upstream face of the core as the reservoir level was lowered. This reduction in lateral support possibly led to a locally unstable condition of the already sheared upstream wedge of core, which then deformed to upstream until adequate lateral support was provided by the upstream shoulder.

(d) Copeton dam (Section 1.5 of Appendix B)

The 113 m high Copeton dam in New South Wales, Australia (Figure 6.28) is a central core earth and rockfill dam that was constructed in the early 1970's. The medium width core is of well-compacted clayey sands of medium plasticity placed at moisture contents in the specified range of 1% dry to 1% wet of Standard optimum. The low pore water pressures that were developed during construction suggest the core was placed on the dry side of Standard optimum. The rockfill shoulders are of quarried granite placed in 1.2 m (Zone 3B) to 3.7 m (Zone 3C) thick layers and compacted with 4 passes of a 9 tonne smooth drum vibratory roller. No water was added during construction.

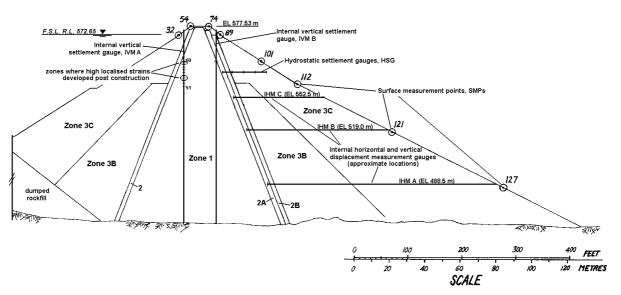


Figure 6.28: Main section at Copeton dam (courtesy of New South Wales Department of Land and Water Conservation)

The post construction deformation behaviour of the SMPs on the crest and slopes of the embankment was "normal" in comparison to similar type embankments. However, the internal core settlement within the internal settlement gauges located slightly upstream of dam axis (IVM A and IVM C) indicated the development of a possible shear zone in the core at close to its upstream interface. The internal deformation of the core during construction in these IVMs was considered "normal" indicating the shear surface did not develop until first filling.

Figure 6.29 shows the development of localised zones of high strain in IVM A, located upstream of the dam axis, at depths of 20 to 30 m below crest level. Similar localised zones of high strain were measured in IVM C

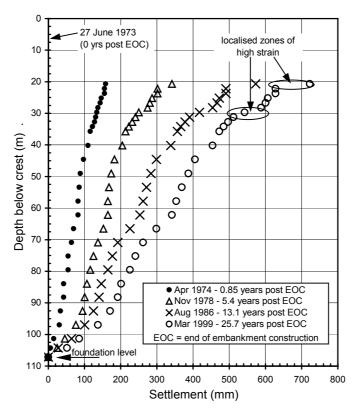
at 20 m depth below crest level. In both IVMs the highest region of strain is between the two top cross-arms and close to the upstream interface of the core with the upstream filter as shown in Figure 6.28. The regions of high shear strain are at elevations where the Zone 3C rockfill (dry placed in 3.7 m lifts) is located immediately upstream of the Zone 2 filter. By March 1999 post construction vertical strains between the upper cross-arms was 6.3% in IVM A (cross-arms 59 to 60) and 8.5% in IVM C (cross-arms 43 to 44), or 95 and 129 mm respectively. In contrast the settlement profile at IVM B, located downstream of the dam axis, shows no localised region of high strain.

The settlement versus time plot of the upper cross-arms intervals in IVMs A and C (Figure 6.30) shows that increases in settlement between the cross-arms occur at similar time periods; during first filling, sometime between 9 and 13 years, and sometime between 20 and 26 years. The localised settlement in the latter periods possibly occurs during rising reservoir level. This is confirmed from the settlement records of SMPs on the upstream edge of the crest which show a small but perceptible increase in the rate of settlement between 10.35 to 11.2 years post construction coincident with the rise in reservoir level to full supply level.

The possible cause and mechanism associated with the development of the localised regions of high strain can only be surmised from the data. It is reasonable to conclude that localised straining between the upper cross-arms in IVMs A and C is concentrated during periods of rising reservoir level, most likely when raised above the elevation of these regions of high strain at 545 to 560 m. A likely explanation is that a shear zone developed within the upstream region of the core on first filling due to high stresses at the upstream core / filter / rockfill interface developed from differential settlement associated with collapse compression on wetting of the Zone 3C dry placed and poorly compacted rockfill. It is notable that the regions of localised high strain in the core are located 5 to 15 m above the elevation where the Zone 3C rockfill was placed immediately upstream of the upstream filter.

The subsequent shear displacements during rising reservoir at about 10.5 to 11 years, and then again at 22 or 25 years, are most likely due to further differential settlement at the upstream interface as indicated by the SMP and IVM data records. A possible explanation for the differential settlement post first filling is softening or degradation of the upstream rockfill over time.

The presence of transverse and longitudinal cracking in the bitumen seal on the crest (first observed in June 1977, 4 years after construction), and the "visually evident" greater settlement of the upstream side of crest noted in surveillance reports (LWC NSW 1995a) give further support to the mechanism of shear development.



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Figure 6.29: Copeton dam, post construction internal settlement profile in the core at IVM A.

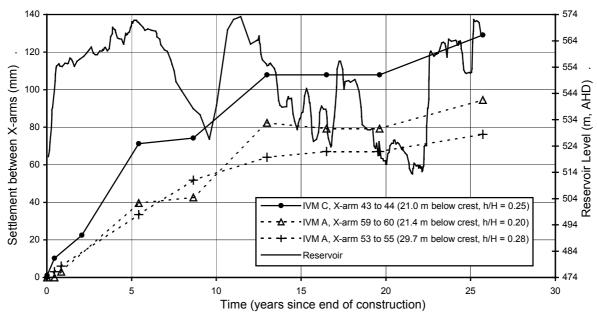


Figure 6.30: Copeton dam, post construction settlement between cross-arms versus time.

6.3.3.2 Shear Surface Development in the Core Post Filling

There are a number of case studies for which a shear surface has or is thought to have developed in the core post first filling. At Eppalock and El Infiernillo dams internal monitoring records indicate that a shear developed in the core, at Djatiluhur dam it is most likely and for Eildon, Bellfield, Cougar and several other dams possibly developed. Details for each of these case studies are presented in Appendix B referencing the sources of data, and they are summarised below (except for Cougar dam which is summarised in Section 6.3.5).

As previously indicated, for virtually all but Eppalock and Djatiluhur dams the overall stability of the embankment is not in question. Therefore these two cases will be dealt with first and in more detail.

(a) Eppalock Dam (Section 1.10 of Appendix B)

The 47 m high Eppalock dam (Figure 6.21), located in central Victoria, Australia is a central core earth and rockfill embankment that was constructed in the early 1960's. The central core of medium plasticity sandy clays were placed in 380 mm loose thickness layers and compacted by sheepsfoot rollers at moisture contents on average 0.8% dry of Standard optimum. The Zone 2A gravel filters were lightly roller and the crushed basalt rock Zone 2B filter zone was end dumped in high lifts. Rockfill was of quarried basalt was dry placed in 2 to 4 m lifts (and one 10 m lift) and spread by tractor.

The reservoir was first filled over the period from May 1962 to November 1963 (0.2 to 1.65 years after construction) and since then is subjected to a seasonal drawdown of typically 3 to 5 m. Larger drawdowns of 7 to 10 m occurred at 5 to 6 years (1967/68), 14 to 16 years (1976/78), 20 to 21 years (1982/83), 32 to 33 years (1994/95) and 36 to 38 years (1997/99) after construction.

On first filling large settlement of the upstream shoulder occurred due to collapse compression on wetting of the dry placed and poorly compacted rockfill. Similar large settlements also occurred for the downstream shoulder in the first 3 to 4 years after construction. Comparatively, the magnitude of crest settlement was much smaller (Figure 6.32). Internal settlements within the core (measured in the internal settlement gauge) over this period were "normal".

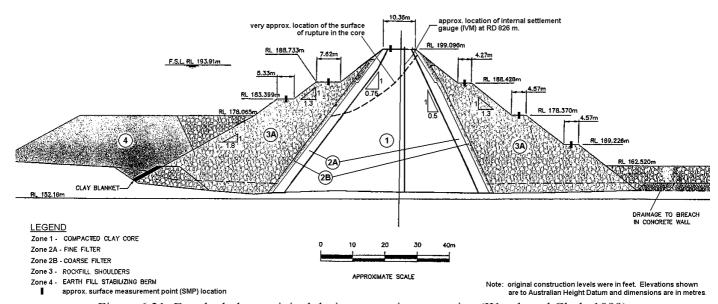


Figure 6.31: Eppalock dam, original design at maximum section (Woodward Clyde 1999)

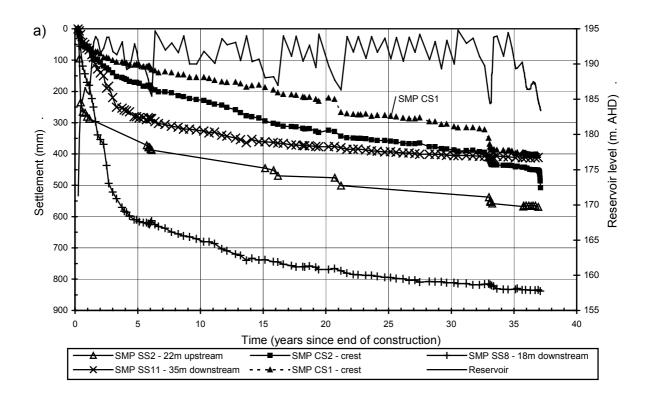
During the larger drawdowns several "abnormal" trends were evident in the deformation behaviour, including:

- Accelerations in the settlement rate of SMPs on the crest, in particular during the drawdowns at 20 to 21 years (SMP CS1 only), 32 to 33 years and 35 to 38 years after end of construction (Figure 6.31). It is notable that the influence of the first large drawdown in 1967/68 (5 to 6 years) on the crest settlement is negligible, but then much larger settlements occurred during the drawdown at 20 to 21 years and increasing settlement magnitude during later large drawdowns. This deformation pattern suggests softening of material strength parameters with time.
- The non-recoverable upstream crest displacement on large drawdown at 32 to 33 years and 37 years (Figure 6.33).

 Acceleration of settlement and non-recoverable upstream displacement of SMP SS2 on the upstream shoulder.

Inclinometers were installed in the crest of the embankment in 1997. On drawdown in 1998 (35 to 36 years) a localised shear type displacement of 1 to 2 mm was observed at 11 m depth below crest level in the inclinometer located next to SMP CS1. The localised displacement occurred between mid March and mid April 1998 when the reservoir was drawn down below 186.8 m AHD to a low at 186.4 m AHD. Further localised shear type displacement at this depth and also at 4 m depth were detected during (and following) placement of additional rockfill on the upper berm of the upstream slope at the time of the remedial works in 1999. Davidson et al (2001) refer to observation of shear type deformations along existing longitudinal cracks in the core and to the surface expression of the shear type deformation on the downstream batter of the exposed core during the remedial works.

These observations are clear evidence of the presence of a surface of rupture within the core oriented to upstream. The surface of rupture has formed because of a lack of support of the relatively stiff core by the rockfill and is not indicative of an overall low factor of safety, which was about 1.4. The accelerations in deformation of SMPs on the crest and upstream slope during large drawdown at 20 years (1982/83) are possibly the first indication of shear development in the core. Significant shear type deformations are unlikely to have occurred before this because they are not evident in the larger drawdown at 5 to 6 years (1967/68) and nor are they evident in the internal deformation of the core (IVM 1) during construction and the first three years after construction. The increasing magnitude of crest settlement and lateral spread of the shear zone in the core on subsequent large drawdown at 33 years and 37 years is considered to be indicative of a gradual softening and possible reduction in factor of safety with time. In comparison to other embankments this trend of increasing magnitude of deformation on subsequent similar sized drawdowns is unusual.



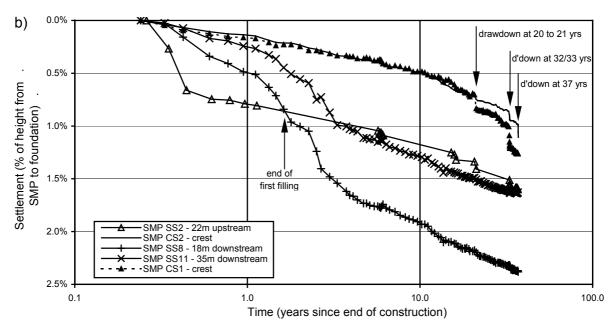
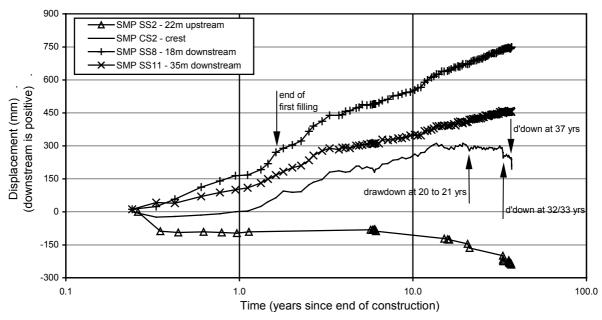


Figure 6.32: Eppalock dam, post construction settlement at main section and CS1.

It is notable that no indication of shear development was observed in the first 3 to 4 years after construction when the settlement of the rockfill shoulders was very much greater than the core. After this period the settlement of the core has been of similar magnitude to that of the rockfill shoulders. The longitudinal cracking on the crest observed from 1973 (11 years after construction) is therefore unlikely to be caused by the differential settlement between the shoulders and core alone, although it is a contributing factor that led to the initial crack development, softening of the core and subsequent shear type deformations within the core. The initial cause of the cracking is not precisely known, but the ongoing cracking is considered to be reflective of differential deformation between the upstream and downstream portions of the core, and not just as shear type deformations. There are not the monitoring records to confirm such behaviour, but the change in displacement trend of SMP CS2, which started in about November 1975 (13.65 years), from being similar to that of the downstream slope to being similar to that of the upstream slope provides some indication of the differential deformation behaviour of the core. The timing of the change is shortly after the initial observation of longitudinal cracking in 1973 suggesting that development of significant cracking in the core initially occurred from about 1973 to late 1975, and preceded the shear deformation in the early 1980's.

Another significant observation at Eppalock dam is the softening that has developed in the core, particularly the upper 5 to 6 m, confirmed from piezocone testing and pressuremeter testing in boreholes. Test pits and boreholes have revealed a series of softened zones at angles of 45 degrees to near vertical extending beyond 4 m depth to about the full supply level.



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Figure 6.33: Eppalock dam, post construction horizontal displacement at the main section.

(b) Djatiluhur dam (Section 1.7 of Appendix B).

The materials and placement methods of the 105 m high Djatiluhur dam (Figure 6.19) and the surface and internal cracking in the core that developed from the latter stages of construction were previously discussed in Section 6.3.2.1, item e.

The post construction settlement of the crest (Figure 6.34) shows acceleration in the settlement rate of the core and upstream edge of the crest occur post first filling on large drawdown to below about elevation 80 m. In comparison to other embankments that show acceleration in the settlement rate on large drawdown, the magnitude of settlement during drawdown at Djatiluhur dam is large (in the order of 120 to more than 300 mm). In addition, the magnitude of settlement of the core between the drawdowns in 1972 (7 years) and 1982 (17 years) is similar, which is an unusual observation and has only been observed at Djatiluhur and Eppalock dams. The general trend is for either negligible or reduced magnitude settlement on a second large drawdown of similar magnitude, or a larger magnitude second drawdown is required for an increase in the settlement rate.

Sowers et al (1993) comment that the continuing deformation of the central and upstream region of the crest is reflective of the "highly" stressed state of the upstream slope as indicated by its marginal factor of safety under static loading. They add that the accelerations in settlement on large drawdown are potentially indicative of shear type displacements. The authors agree with Sowers et al (1993) but add that the settlement data on drawdown also indicates a softening in the material strength properties over time. This would suggest that the development of a shear surface and strength loss due to shearing on this surface of rupture, at least in the core.

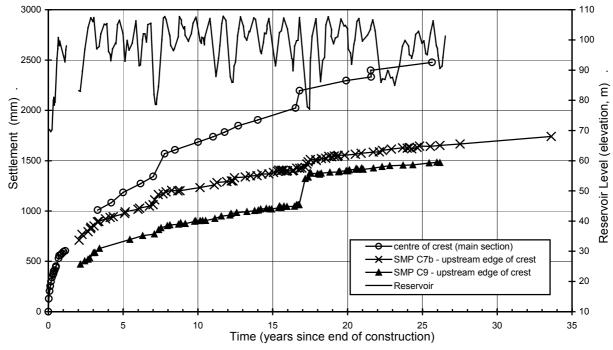


Figure 6.34: Djatiluhur dam, post construction crest settlement.

(c) Bellfield Dam, Victoria, Australia (Section 1.2 of Appendix B).

Bellfield dam is a 55 m high central core earth and rockfill dam constructed in the mid 1960's. The medium width core of sandy clays to clayey sands was compacted in 380 mm layers (loose thickness) at a specified moisture content range from 1.5% dry to 1.5% wet of standard optimum. The thin filter zones were compacted with steel flat drum rollers and the rockfill shoulders dry placed/dumped in 1.2 to 9.1 m lifts sloped at the angle of repose. The rockfill was sourced from mostly quarried sandstones but with some siltstones and mudstones.

The design and construction methods are very similar to those at Eppalock dam.

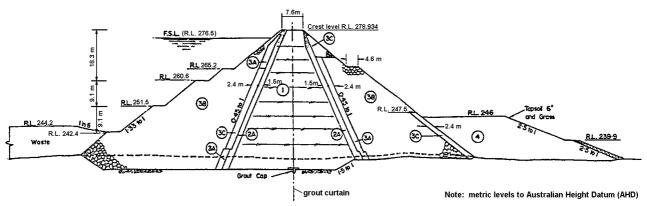


Figure 6.35: Bellfield dam, main section at chainage 701 m (courtesy of Wimmera Mallee Water)

The data records obtained (SMEC 1998a) for the SMPs and IVM in the core only cover the post construction period from 1987 to 1997 (21 to 31 years after construction). The IVM records from end of construction to February 1987 (Figure 4.45a) show two regions of high localised vertical strain within the core, one at 14 to 15 m below crest level and the other at 35 m depth. The timing and cause of the concentrated settlement is not known. It may represent local yielding or possible localised shear type displacements in the core.

The records from February 1987 to November 1997 (Figure 4.45b) show a localised zone of higher vertical strain developing at 28 to 30 m below crest level, inconsistent in elevation with those developed prior to 1987.

(d) El Infiernillo Dam, Mexico (Section 1.9 of Appendix B).

The materials and placement methods of the 148 m high El Infiernillo dam (Figure 6.9) and the internal vertical deformation behaviour during first filling were previously discussed in Section 6.3.2.1, item a. The post first filling internal settlement records in the core over the period mid 1966 to 1972 (Marsal and Ramirez de Arellano 1972), 2.6 to 8 years after construction, indicate a region of high vertical strain developed in the core at about 45 m depth below crest level (Figure 6.36). The region of high strain is evident in October 1967 and continued to progressively develop into the early 1970s. Over the corresponding period the displacement shows a block type deformation to downstream above elevation 135 m, and possibly a small reverse displacement to upstream between elevations 130 and 135 m.

At the time of the formation of the localised region of high strain a sustained period of accelerated settlement of the crest and downstream shoulder, and downstream displacement was occurring (Figure 6.37). Marsal and Ramirez de Arellano (1972) comment that the increased rate of deformation was coincident with the flooding of the lower portion of the downstream rockfill due to high tail water levels in October 1966 (2.8 years), January 1967 (3.1 years) and September 1967 (3.8 years) and periods of heavy rainfall. Collapse compression in the downstream rockfill due to wetting from inundation and rainfall is suspected as the cause of the increased rate of settlement and downstream displacement from late 1966 to 1968 of the crest and downstream shoulder.

Interpretation of the deformation behaviour and mechanism leading to development of the localised region of high vertical strain is not clear. It may be related to the deformation of the downstream shoulder, but it could be related to localised instability of the core under the larger drawdowns from 1968 onward. The internal core displacement to upstream at the location of high strain may indicate it is drawdown related.

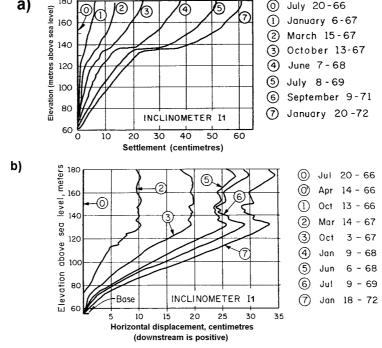
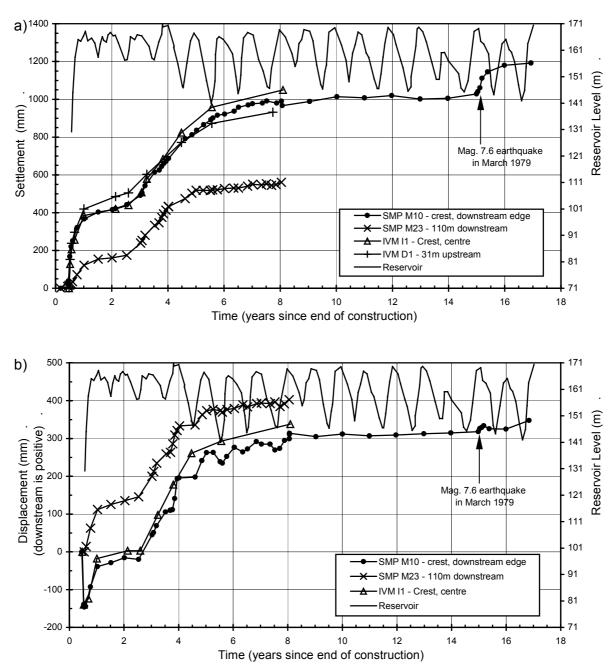


Figure 6.36: El Infiernillo dam, internal (a) settlement and (b) displacement in the core from 1966 to 1972 (Marsal and Ramirez de Arellano 1972)



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Figure 6.37: El Infiernillo dam, post construction (a) settlement and (b) displacement of surface markers.

Marsal and Ramirez de Arellano (1972) comment that the "abnormalities" in the post construction deformation behaviour are largely controlled by collapse type deformations of the dry and poorly placed rockfill, and due to incompatibility of the stress-strain characteristics between the materials used in the embankment. They add that the deformation of the core is very sensitive to interactions with the surrounding granular mass.

(e) Eildon Dam, Victoria, Australia (Section 1.8 of Appendix B)

Eildon dam (Figure 6.38) is a central core earth and rockfill embankment of 80 m maximum height that was constructed in the early to mid 1950's. The central core consisted of an inner zone (Zone 1A) of medium plasticity silty to sandy and gravelly clays and outer zone (Zone 1B) of clayey sands to silty sands. Both zones were placed on the dry side of Standard optimum and well compacted. The sandy gravel filter / transition zone was generally placed by end dumping without compaction, and the rockfill shoulders were placed in at least 2 m thick layers, probably without the addition of water and not formally compacted. The rockfill was sourced from

quartzitic sandstone for Zones 3A and 3B, and the random rockfill zone (Zone 3C) consisted of unsuitable rock that was poorly graded and contained a high fraction of finer sized rockfill.

Aspects of the post construction deformation indicate the possibility that a shear formed within the core sometime after the end of construction and that deformations on the shear surface occur during periods of large drawdown. They are:

- The settlement records from the internal vertical measurement gauges (IVM) installed in the core (Figure 6.40). The records for the period from 26 years after end of construction show localised zones of high vertical strain (IVM ES2 and ES3) at 17 to 20 m depth below crest level, a large portion of which occurred during the latter stages of the large drawdown in 1982/83 at 27 years. These zones are located in Zone 1A within 1 to 2 m of the upstream interface with Zone 1B. At IVM ES3 a blockage or constriction in the tube has limited measurements to the upper 20 m of the gauge.
- SMPs on the crest and upstream slope show an increase in settlement rate on large drawdown at 13 years (1968) and 27 years (1982/83) (Figure 6.39), and also show a non-recoverable upstream displacement at 27 years. Most of the other SMPs on the crest and upstream slope (SMEC 1999a) displayed a similar increase in settlement rate and non-recovered upstream displacement during these drawdowns.

The cause of the possible shear within the core at Eildon dam is not known. It could be due to poor support from the rockfill shoulders as in the case of Eppalock dam or due to differential settlement between the upstream Zone 1B outer sandy loam core and the inner Zone 1A clay core.

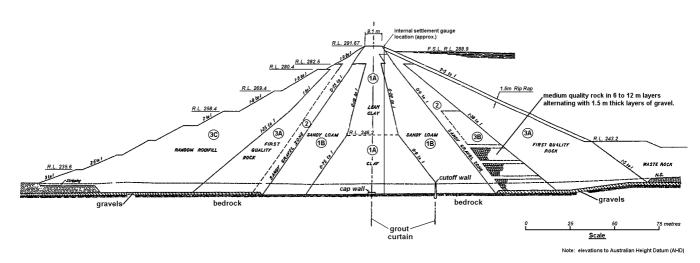


Figure 6.38: Main section at Eildon dam (courtesy of Goulburn Murray Water).

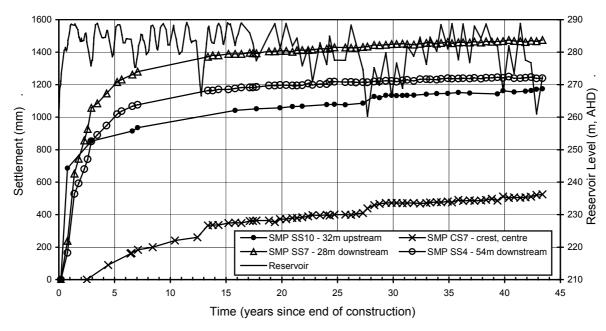


Figure 6.39: Eildon dam, post construction settlement of SMPs at chainage 685 m.

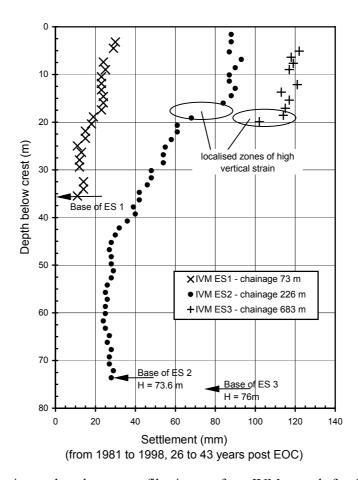


Figure 6.40: Eildon dam, internal settlement profiles in core from IVM records for the period 1981 to 1998.

6.3.4 Post First Filling Acceleration in Deformation that is Not Known to be Shear Related

A number of central core earth and rockfill dams show periods of acceleration in the deformation rate post first filling that may not be related to shear deformation in the core. Some of these have already been discussed, including the influence of earthquake such as at Matahina dam (Figure 4.64) and El Infiernillo dam (Figure 6.37a), and the influence of tail water impoundment of the lower downstream shoulder or heavy rainfall, such as at El Infiernillo dam (Figure 6.37a).

In a number of other dams a short period of acceleration and small settlement of SMPs on the crest or upstream shoulder is observed during the first and occasionally the second drawdown. Case studies include, but are not limited to Beliche, Dartmouth, Geehi, Parangana, Cherry Valley, Cougar, Wyangala and Gepatsch dams. The case studies include dams where collapse compression of the upstream rockfill on wetting is potentially significant, and others where it is less significant such as for well and reasonably to well compacted dry placed rockfills (e.g. Dartmouth and Wyangala dams). The deformation behaviour is not considered related to shear type deformation within the core since there are no localised zones of high strain in the IVM data. Although not well understood, several possible explanations for the deformation behaviour have been considered most of which relate to softening of the strength and/or compressibility properties of the materials on wetting.

For rockfills susceptible to collapse compression on wetting, the greatest effective vertical stress acting on the saturated rockfill occurs immediately on wetting. Once the reservoir level is raised slightly the vertical and lateral effective stresses in the upstream rockfill will decrease. If the reservoir is then lowered, the maximum vertical effective stress will increase, but will not exceed the previous maximum level at initial saturation assuming the rockfill is free draining. With respect to the deformation behaviour after saturation and collapse compression, the expectation is that a slight heave may occur on decreasing effective vertical stress and a slight settlement may occur on increasing effective stress as the stress path oscillates along an unloading / reloading path for which the rockfill modulus would be high. A large increase in settlement on increasing vertical stress would therefore be unexpected.

At Gepatsch dam, Schober (1967) attributed the acceleration in settlement of the SMP on the upstream shoulder on drawdown to increased effective vertical stresses in the upstream shoulder due to completion of the embankment construction at high reservoir levels (refer Section 1.11 of Appendix B). This is a reasonable argument for this case study. At Dartmouth dam this reasoning would partly explain the settlement of the upstream shoulder on drawdown, but part of the settlement occurred before the reservoir had been drawn down to below the level at end of construction. At Geehi dam, where first filling did not start until after the end of construction, the acceleration in settlement of the upstream shoulder on large drawdown cannot be explained by this reasoning. Nor can it explain the acceleration in deformation at Copeton and Wyangala dams.

Settlements of the crest of these dams are probably more readily explainable by softening of the shear strength and compressibility of the earthfill. Equilibrium pore pressure conditions after construction can take many years to develop in the core. It is conceivable therefore that on drawdown effective vertical stresses for the near saturated earthfill may exceed those previously experienced on first filling and an acceleration in settlement may occur under the "softened" compressibility properties of the earthfill. This may explain the observed increase in settlement rate on first drawdown for the more permeable silty earthfills such as at Cherry Valley and Parangana dams.

For the wet placed clayey earthfills (e.g. at Beliche and Dartmouth dams) softening of the strength and compressibility properties of the earthfill is not a valid explanation for the acceleration in deformation because the earthfills are already close to saturation. In these cases it is likely that the reduction in hydrostatic pressures

acting on the upstream face of the core as the water level is drawn down influences the deformation behaviour of the core. On drawdown, the reduction in hydrostatic stress will initially cause the embankment crest to deflect upstream as total stresses are reduced in the core and downstream shoulder. At some point though, lateral stresses must increase in the upstream shoulder to equilibrate the lateral stresses in the wet placed core. For rockfills where collapse compression and reduction of the compressibility occurred during first filling, it is conceivable that this increase in lateral stress will be associated with a net upstream displacement of the core / upstream shoulder interface, resulting in lateral spreading of the core. For wet placed cores of low undrained strength, the lateral spreading is likely to largely occur as undrained plastic deformation and will be associated with vertical compression to maintain volumetric consistency, and hence settlement of the crest.

6.3.5 Other Case Studies with Potentially "Abnormal" Deformation Behaviour Post Construction

Further discussion on potentially "abnormal" aspects of the deformation behaviour at Svartevann, Cougar and Wyangala dams is presented due to their informative nature. The deformation behaviour relates to:

- The very large magnitude settlement and displacement of the crest and downstream shoulder at Svartevann dam in the first four years after construction.
- The timing of the large deformation and longitudinal crest cracking at Cougar dam.
- The internal settlement of the earthfill core at close to its upstream interface at Wyangala dam.

A summary of each case study is presented below. Additional details are provided in Appendix B and in the references.

(a) Svartevann dam, Norway (Section 1.13 of Appendix B)

The 129 m high Svartevann dam (Figure 6.41) is a central core earth and rockfill dam with a thin, slightly upstream sloping core. It was constructed in the seasonally warmer months between 1973 and 1976 and stored water during construction. The core of silty sand to silty gravel moraine deposits was placed at an average moisture content of 0.4% wet of Standard optimum and well-compacted by heavy vibrating rollers. The filter / transition zones of gravels and crushed rock were sluiced and well-compacted. Rockfill of quarried granitic gneiss was dry placed and reasonably compacted (2 metre lifts compacted with 8 passes of a 13 tonne smooth drum vibrating roller).

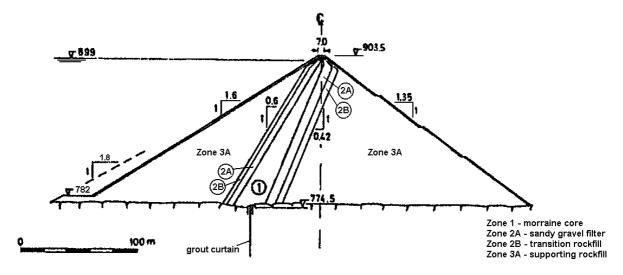


Figure 6.41: Main section at Svartevann dam (Kjærnsli et al 1982).

The post construction deformations at Svartevann dam, particularly during the period of first filling, were relatively high. The magnitude of the crest displacement on first filling at 1100 mm was a clear outlier to other case studies (Figure 4.38 and Figure 4.72) and the crest settlement and deformation of the downstream shoulder very high, but possibly not "abnormally" so. Compared to the case study data for zoned earth and rockfill dams with moraine cores (Dascal 1987) the crest deformation at Svartevann is clearly very large.

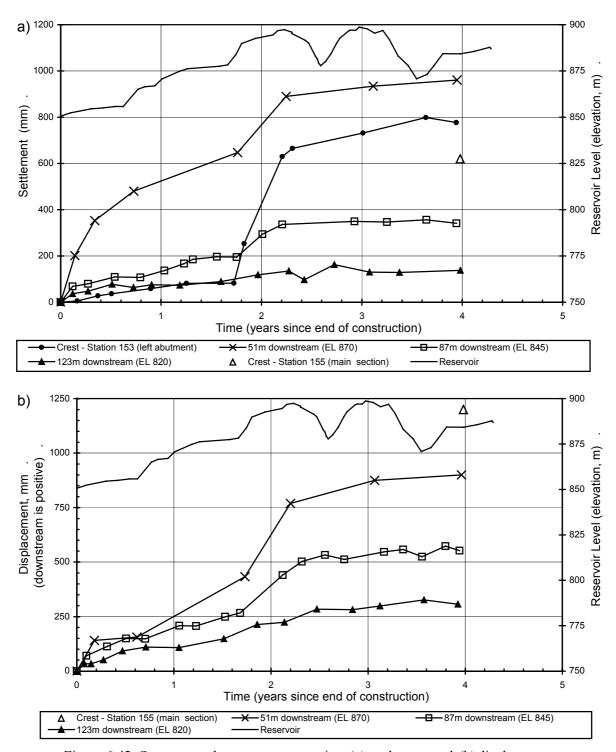
As shown in Figure 6.42, a large portion of the surface deformation at the crest and downstream slope occurred during the final 20 m raising of the reservoir to full supply level, although significant settlements were also measured for the upper downstream slope shortly after the end of construction. Finite element analysis by Dibiagio et al (1982) was unable to accurately model the deformation behaviour within the downstream shoulder, it significantly over-predicted the horizontal displacements and significantly under-predicted the settlements.

The post construction internal deformation of the downstream shoulder (Kjærnsli et al 1982) shows that large settlements and displacements occurred in the mid to upper region of the shoulder in the first 4 years after end of construction. Vertical strains were estimated at 1.4 to 2.2% in the mid to upper region of the downstream rockfill for this post construction period, with much lower vertical strains, 0.5 to 1.0%, estimated for the lower 45 to 50 m (below elevation 820 m). The deformation time plots show that very large settlements and downstream displacements of the mid to upper slopes occurred during the latter stages of first filling, but not on the lower downstream slope.

The large crest deformations on first filling observed at Svartevann dam appear to be related to the deformation of the downstream shoulder. In effect, the core deforms with the downstream shoulder.

The inability of the finite element analysis (Dibiagio et al 1982) to model the deformation behaviour on first filling, particularly the vertical component, suggests that the application of the water load and the associated changes in stress conditions on its own does not account for the actual deformation behaviour. In addition, time dependent or creep related deformations would not account for the vertical deformation because the magnitudes are too large.

It is considered that the large deformations of the mid to upper region of the downstream shoulder are due to collapse compression in the dry placed and reasonably compacted rockfill. The trigger for the collapse type settlement must be moisture related, so possibly either heavy rainfall or snowmelt is the source of water and the timing during the latter stages of first filling is somewhat coincidental. It is notable that the accelerations in deformation rate of SMPs on the mid to upper downstream slope in the first 2 years post construction occur at the same time period each year, from 0.0 to 0.2, 0.8 to 1.2 and 1.8 to 2.2 years corresponding to the period from June to September, or summer. This is the seasonally wettest and warmest period for the western coast of Norway. Therefore, heavy rainfall together with snowmelt may sufficiently wet the downstream rockfill for collapse settlement to occur. Possibly 1978 was a relatively wet summer or the winter one of high snowfall.



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Figure 6.42: Svartevann dam, post construction (a) settlement and (b) displacement.

(b) Cougar Dam, Oregon, USA (Section 1.6 of Appendix B)

The 159 m high Cougar dam (Figure 6.43) is a central core earth and rockfill embankment that was constructed in the early 1960's (Pope 1967). The slightly upstream sloping narrow core of silty gravels was placed at an average moisture content of 1% wet of Standard optimum and well-compacted. The filter / transition zones were also well compacted. Rockfill of quarried basalt and andesite was used. Zone 3A consisted of well-compacted sound rock, Zone 3B of sound rock placed in 900 mm layers and tracked by 2 passes of a D8 bulldozer, and Zone 3C was of lesser quality rock comprising up to 25% weathered rock placed in 600 mm layers and tracked by D8 bulldozer.

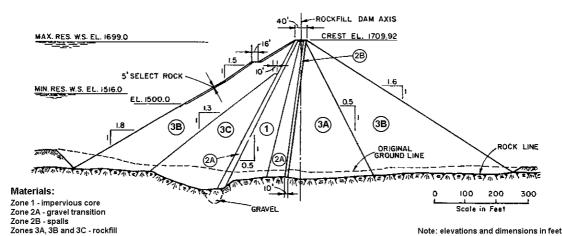


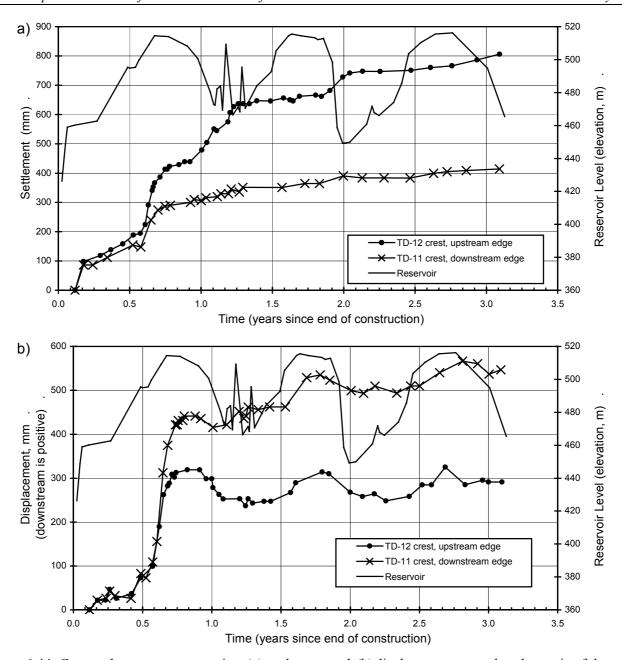
Figure 6.43: Cross section at Cougar dam (Cooke and Strassburger 1988).

On first filling the post construction crest deformations at about the maximum section, Figure 6.44, show that during the last 20 m to full supply level the settlement and downstream displacement of the crest increased significantly in magnitude. This latter part of the deformation was not uniform, settlements were greater at the upstream edge of the crest and displacements greater at the downstream edge of the crest. On the first and second drawdowns acceleration in the settlement rate of the SMP on the upstream edge of the crest occurred, resulting in a marked increase in the magnitude of settlement. At 3 years after construction the differential settlement of the crest had increased to about 400 mm and the lateral spreading to about 250 mm.

Cracking of the embankment was a consequence of the differential deformation between the upstream and downstream edges of the crest. As described by Pope (1967), longitudinal cracking of the crest was observed at the end of first filling and within several days had extended over a length of almost 300 metres. The cracking was mostly near the downstream core / transition interface. The cracking re-appeared in January 1965, shortly after the end of the first drawdown, located at the up and downstream core / filter interfaces and the downstream Zone 2A / Zone 2B interface (Figure 6.45). Cracks widths were up to 150 mm and differential vertical displacement (to upstream) across the crack occurred at the up and downstream edges of the core of 300 mm and 150 mm respectively.

Pope (1967) considered collapse type settlement of the upstream rockfill on wetting, particularly within the lesser quality track rolled Zone 3C rockfill, to be a significant factor in the observed deformation behaviour and cracking. But, what is interesting with the deformation at Cougar dam is that a large proportion of the differential deformation at the crest occurred during drawdown and not on first filling. This would suggest that collapse settlement of the upstream rockfill on initial saturation, whilst significant, is not the major cause of the differential deformation. A possible reason for the large differential settlements post first filling is that the lesser quality Zone 3C rockfill lost additional strength whilst saturated, and then under the increasing effective stress conditions on drawdown further settlement of the upstream rockfill occurred. The second drawdown was to a lower level than the first resulting in higher effective stress conditions in the upstream rockfill than at the end of the first drawdown, which may explain the further increase in settlement on the second drawdown.

The vertical offset at the downstream core / filter interface could indicate a potential shear development in the silty gravel core caused largely as a result of the high shear stresses at the upstream core / filter interface due to differential settlement between the upstream shoulder and the core. The fact that no cracking was observed at the upstream filter / rockfill interface is interesting because these materials probably have the greatest difference in compressibility properties. It is possible that a shear surface has developed in the upstream gravel filter at some point and as a result the upper part of the gravel filter deforms with the upstream rockfill.



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Figure 6.44: Cougar dam; post construction (a) settlement and (b) displacement normal to dam axis of the crest at the maximum section (adapted from Pope 1967).

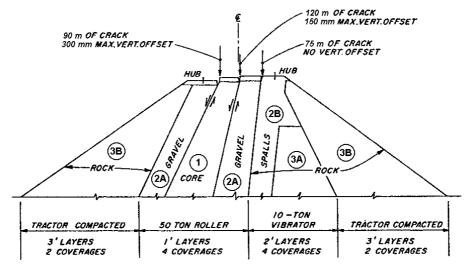


Figure 6.45: Cougar dam; differential settlement and cracks on crest (Pope 1967)

(c) Wyangala dam, New South Wales, Australia (Section 1.16 of Appendix B).

Wyangala dam (Figure 6.46) is a central core earth and rockfill dam of 85 m maximum height constructed in the 1960's. In the deeper valley section the old concrete dam forms the upstream toe of the earth and rockfill dam. The core (Zone 1) of silty to clayey sands was placed at an average moisture content of 1% dry of Standard optimum and well compacted. The filters were also well compacted. Quarried porphyritic gneiss was the main source of rockfill and for the most part was placed without the addition of water. Zone 3A was well-compacted in 0.9 m lifts, Zone 3B reasonably to well compacted in 1.2 m lifts and Zone 3C reasonably compacted in 2.4 m lifts.

During construction, the deformation behaviour of Wyangala dam was considered "normal" in comparison to other similar dams

Post construction, the deformation behaviour of Wyangala dam was, for the most part, considered "normal". An interesting aspect of the deformation behaviour is the very high vertical strains developed in the core at IVM B (Figure 6.47) between 23 and 32 m depth below crest level (1.85% at 21 years after end of construction). IVM B is located slightly upstream of the dam axis (Figure 6.46) and the region of high vertical strain is located near to the upstream core interface. The vertical strains in this region are very much higher than the average vertical strain of 0.48% in IVM A at a similar depth (located downstream of the dam axis) and in the mid to lower core region at IVM B (average less than 0.15%). The localised high vertical strains developed steadily over the first 11.5 years (1.2% vertical strain at 11.5 years), but increased in rate in the period from 11.5 to 17 years (0.5% vertical strain over this period for a total of 1.7% to 17 years).

The magnitude of settlement between the cross-arms over the region of high strain totals 155 mm in 17 years, which is approximately equivalent to the differential settlement between the up and downstream edges of the crest (about 100 to 120 mm) over this period plus the "normal" settlement over the depth range (estimated at 20 to 30 mm from IVM A). This would indicate that the region of high strain relates to differential settlement between the core and upstream shoulder. Whilst this is a shear type movement in itself, a single distinct shear surface does not appear to have developed as indicated by the broad width of the region of high strain. It is possible that a series of shears may have developed or that general softening of the earthfill may be a factor within the zone of high strain.

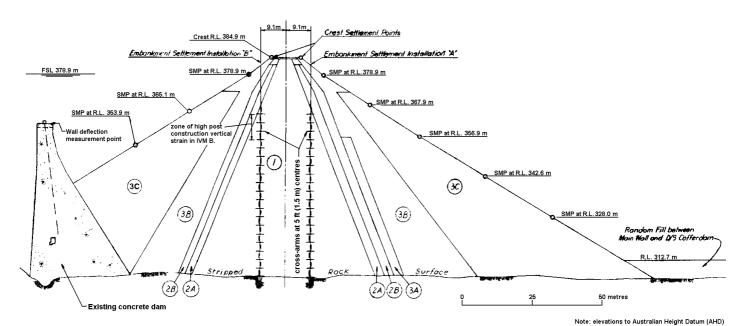
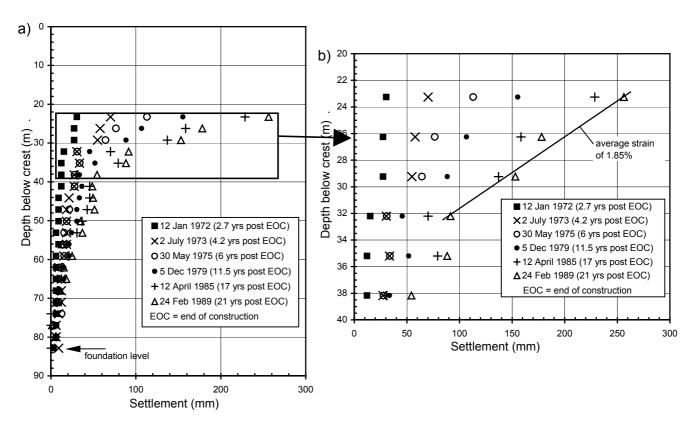


Figure 6.46: Section at Wyangala dam (courtesy of New South Wales Department of Land and Water Conservation).



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Figure 6.47: Wyangala dam, post construction internal settlement in IVM B located 9 m upstream of dam axis.

6.4 "ABNORMAL" DEFORMATION BEHAVIOUR POST CONSTRUCTION OF EARTHFILL AND ZONED EMBANKMENTS WITH VERY BROAD CORE WIDTHS

In Sections 4.2.3 and 4.3 the post construction deformation behaviour of a number of rolled earthfill embankments and zoned embankments with very broad earthfill cores were identified as "abnormal" or possibly "abnormal". In the following sub-sections the possible mechanisms explaining the deformation behaviour are discussed with reference to the case study data. The association of a case study to a particular mechanism has, in some cases, previously been identified by the owner, their consultants or other authors, and in other cases has been considered appropriate by the authors of this report. For some cases the association is speculative. Further details on the embankment and its deformation behaviour for most of the case studies discussed in this section are presented in Appendix B.

6.4.1 Collapse Compression of the Earthfill on Wetting

At Hirakud dam the large settlements in the lower portion of the embankment that occurred during construction were attributed to the effects of collapse compression of the earthfill on wetting (refer Section 6.2.2). Two factors were significant in the deformation behaviour; the deformation that occurred on initial wetting of the dry placed clayey sand to clayey gravel earthfill under the stress conditions existing at the time, and the large deformations as the embankment was constructed to design level after the shutdown period when the wetting occurred. The large deformations during the subsequent construction were indicative of the low shear strength and compressibility properties of the "collapsed" and softened earthfill. The influence of change in material strength and compressibility properties after wetting for embankments that are initially saturated on first filling after construction is important and its possible influence is discussed further for some of the case studies.

Rector Creek, Mita Hills and Roxo dams are considered examples of collapse compression of dry placed earthfills that occurred due to wetting after construction. Collapse compression possibly also occurred in the upstream shoulder to central region of Dixon Canyon, Spring Canyon and Horsetooth dams. All these cases are discussed further in Appendix B.

"Abnormal" deformation behaviour due to collapse compression on wetting is generally identified by excessive settlements that largely occur on wetting. This is particularly evident at Rector Creek, Mita Hills and Roxo dams. Another pattern sometimes observed is an "abnormally" large upstream displacement of the crest on first filling and later downstream displacement as the wetting front progressively develops through the embankment.

The deformation behaviour at Rector Creek dam (Sherard et al 1963; Sherard 1973; ICOLD 1974) presents probably the clearest example of collapse compression. The embankment (Figure 6.48) is a 61 m high zoned earthfill embankment that was completed in January 1947. The central earthfill region (Zones 1 and 2) was of silty to clayey sands with low plasticity fines, the finer materials being used in the central core region (Zone 1) and the coarser earthfills in the outer Zone 2 region. Compaction was in 150 mm layers by heavy sheepsfoot roller and moisture contents at placement were 2 to 4% dry of Standard optimum.

The post construction deformation of SMPs on the crest near to the main section (Figure 6.49) shows the large magnitude of crest settlement (almost 1.8% at 10 years after construction) and the large upstream then downstream displacement of the crest. In comparison to similar embankments the magnitude of settlement and long-term settlement rate of the crest are "abnormally" high (Figure 4.46, Figure 4.60a and Figure 4.63), and the magnitude and direction of the crest displacement during and post first filling clear outliers to the general behaviour (Figure 4.38, Figure 4.75 and Figure 4.77). ICOLD (1974) consider that collapse settlement on wetting of the dry placed earthfill and the gradual development of the phreatic surface within the embankment contributed to the observed crest settlement and displacement behaviour, and to the observed cracking. Initial wetting and collapse settlement of the upstream shoulder resulted in the upstream displacement of the crest, and subsequent wetting and collapse settlement in the central to downstream portion of the embankment resulted in the change in direction of displacement to downstream some 2 to 2.3 years after construction. The high long-term settlement rate of about 1.65% per log cycle of time maintained for at least 10 years after construction is possibly reflective of the large reduction in material strength and compressibility properties after wetting.

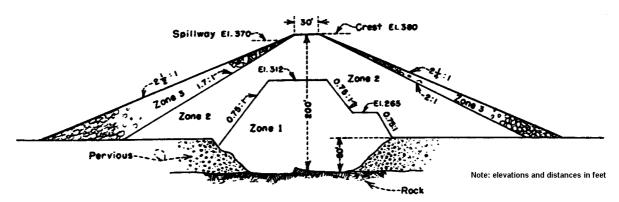
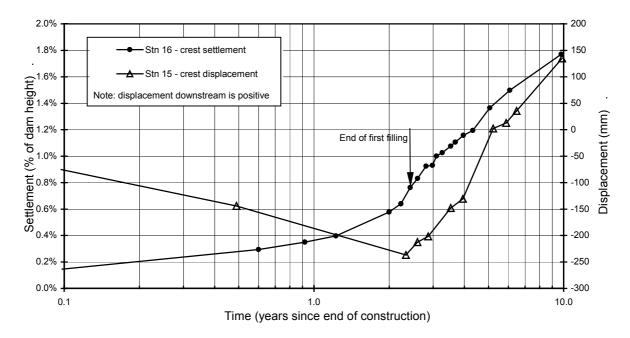


Figure 6.48: Rector Creek dam main section (Sherard 1953).



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Figure 6.49: Rector Creek dam; post construction settlement and displacement versus log time (adapted from Sherard et al 1963).

Collapse compression is considered a significant factor for the large crest settlements measured at Roxo dam. The dam, constructed in the 1960's, is part concrete and part earthfill embankment. The earthfill embankment is of 27 to 32 m maximum height and is constructed of medium plasticity clayey sands to sandy clays derived from weathered schists placed at moisture contents in the range 2% dry of OMC to OMC (OMC = Standard optimum moisture content). Large post construction crest settlements, up to almost 2% of the embankment height at 8 years (Figure 6.50), were measured over a large portion of the 80 to 100 metre long closure section of the earthfill embankment located at the interface between the two structures. In comparison to similar embankments, the magnitude and rate of the crest settlement is "abnormally" high (Figure 4.46, Figure 4.60a and Figure 4.63). At 95 metres from the interface the post construction crest settlement is more typical of "normal" type behaviour.

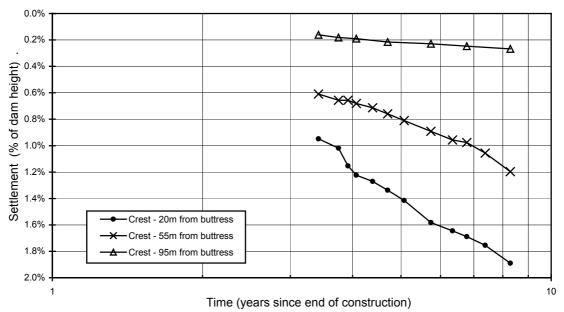


Figure 6.50: Roxo dam, settlement versus log time (adapted from De Melo and Direito 1982).

The cause of the large settlement is considered due possibly to collapse type settlements on wetting of dry placed and potentially poorly compacted earthfill in the closure section. Investigations and excavation (De Melo and Direito 1982) showed the presence of wetted and softened layers close to those in the as placed condition, but it is not clear to what extent these layers were observed.

De Melo and Direito (1982) do not consider collapse compression as a cause of the large settlements. Instead, they consider it due to a combination of factors including arching across changes in foundation geometry near to the abutment and the formation of horizontal cracks due to differential settlement and stress transfer between the earthfill and concrete structures. But these factors alone would only influence the deformation in the vicinity of the interface, and, as the settlement records show, the "abnormally" large settlements were measured more than 50 to 60 metres from the interface. Hence, collapse compression on wetting is considered a potentially significant factor.

Large reductions in the shear strength and compressibility properties of the earthfill may be contributing to the "abnormally" high long-term rate of crest settlement.

At Mita Hills dam (Figure 6.51) collapse compression on wetting of the outer upstream earthfill zone (Zone 3) is considered a potentially significant factor in the "abnormally" large magnitude of crest settlement and the upstream crest displacement on first filling. The embankment (Legge 1970), constructed in the late 1950's, is of 49 m maximum height. The earthfill was placed in 150 mm layers and compacted by heavy sheepsfoot rollers. The specified range in moisture content for the outer earthfill zones was 2% dry of OMC to OMC (OMC = Standard optimum moisture content) and 1% dry to 1% wet of OMC for the internal Zone 2 upstream of the chimney filters.

As shown in Figure 4.60a and Figure 4.46 the magnitude of crest settlement at Mita Hills dam of more than 1% is "abnormally" high, and the magnitude of upstream displacement (Figure 4.75 and Figure 4.38) possibly "abnormal" in comparison to similar type embankments. Most of the deformation occurred on first filling, and post first filling the crest deformation has been "normal" in comparison with that of other embankments. In addition, very high magnitude settlements were measured on the mid abutment slopes (up to 2.3% of the embankment height at these locations) and the deformation of SMPs on the downstream shoulder were very small (settlement of 0.27% of the height from the SMP to foundation level at the main section) in comparison to those on the crest.

The possibility that the deformation is related to marginal stability of the upstream slope has been considered, however, an instability mechanism seems unlikely as no acceleration in settlement rate or further upstream displacement is observed on drawdown (refer Section 4.2 of Appendix B).

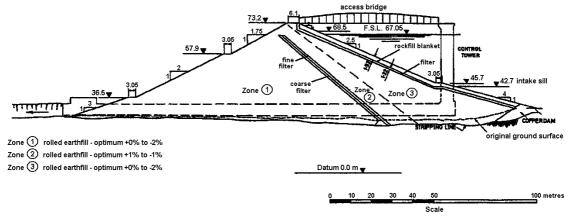


Figure 6.51: Main section at Mita Hills dam (Legge 1970)

For the Horsetooth Reservoir embankments Dixon Canyon, Spring Canyon and Horsetooth dams collapse compression is considered a possible cause of the large magnitude of settlement of SMPs on the upper upstream slope and crest. The Horsetooth Reservoir embankments were constructed in late 1940's to maximum heights of 48 to 74 m. A similar design was adopted for all of the main embankments (Figure 6.52 is of Dixon Canyon dam) consisting of a very broad central earthfill zone with thin outer zones of gravelly earthfill or rockfill. In the valley sections a cut-off was excavated through the over-burden soils to bedrock with most of the embankment founded on the over-burden soils. The cut-off design was similar for all embankments except Horsetooth dam, where it was located upstream of the dam centreline.

The earthfill zone was constructed of mainly low plasticity sandy clays to clayey sands and clayey gravels of alluvial origin. Finer materials were placed in the central region of the core and coarser materials toward the outer slopes. The earthfill was placed in 150 mm layers and compacted by tamping rollers. Moisture contents were well dry of Standard Proctor optimum, on average 2.2% to 2.9% dry.

The post construction deformation behaviour of SMPs on the upper upstream slope near to the main section for the Horsetooth Reservoir embankments is shown in Figure 6.53. Possible "abnormal" aspects of the deformation behaviour in comparison to similar embankments are:

- The large magnitude of settlement of the upstream slope for Horsetooth, Dixon Canyon and Spring Canyon dams where settlements were in the range 1.35 to 2.0% (Figure 4.52 and Figure A2.28 of Appendix A).
- The high rate of long-term settlement of the upstream slope for Dixon Canyon dam of 1.38% per log cycle of time (Figure 4.83). At Spring Canyon dam the settlement rate was high over the period from 8 to 30 years, but has since reduced from 30 to 45 years.
- For Dixon Canyon dam, the large upstream displacement on and after first filling followed by the change to downstream displacement after 30 years for the upstream slope (Figure 4.87). The trend of displacement at Spring and Soldier Canyon dams is similar, but at a much reduced magnitude and rate of displacement.
- The crest settlement behaviour of Dixon Canyon and Spring Canyon dams is also potentially "abnormal" in terms of the large magnitude (Figure 4.48 and Figure 4.61) and high long-term settlement rate (Figure 4.63).

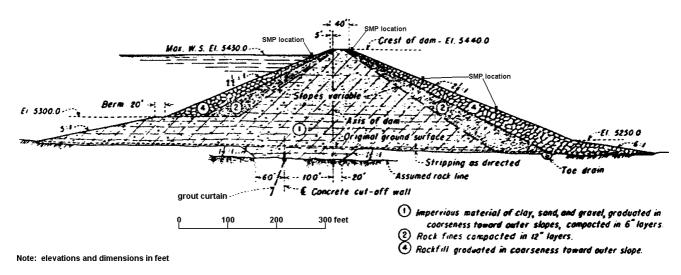
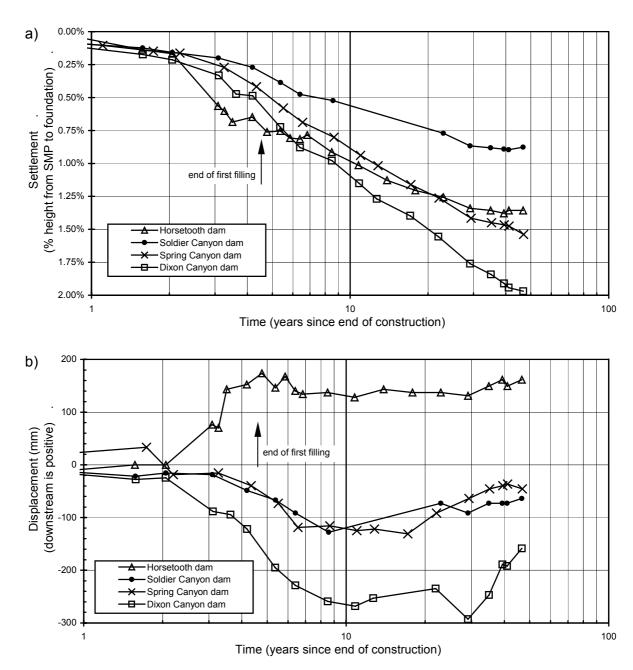


Figure 6.52: Main section at Dixon Canyon dam (courtesy of U.S. Bureau of Reclamation).



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Figure 6.53: Post construction (a) settlement and (b) displacement of SMPs on the upper upstream slope at the Horsetooth Reservoir embankments.

The influence of the foundation on the settlement and displacement of the crest at Horsetooth was significant (refer Section 3.3 of Appendix B). It is also likely to have influenced the deformation of the upper upstream slope, but to a lesser degree, as the SMP overlies the downstream edge of the cut-off trench to bedrock. The downstream displacement and large settlement on first filling of the SMP on the upstream slope at Horsetooth dam (Figure 6.53) are likely due to the foundation influence.

At Dixon Canyon and Spring Canyon dams the foundation is considered to have a limited influence on the post construction deformation behaviour and most of the measured deformation occurs within the embankment. This is based on the comparison of the trend of deformation at these embankments to that at Horsetooth dam where the influence of the foundation was very significant during first filling, but much less so post first filling. In addition, the SMPs on the upstream slope at Dixon Canyon and Spring Canyon dams directly overlay the cut-off trench and therefore the foundation influence will be very limited.

Collapse compression of the dry placed earthfill, particularly in the coarser soil types likely to be present in the upstream portion of the main earthfill zone, is considered to be a possible explanation for the observed potentially "abnormal" deformation behaviour of the upstream shoulder of Horsetooth, Spring Canyon and Dixon Canyon dams, and the crest at Spring and Dixon Canyon dams. The effect of softening of the shear strength and reduction in compressibility on wetting is likely to have some influence on the on-going very high settlement rates (rate per log time).

The similarity between the case studies considered to be susceptible to collapse compression, including Hirakud dam, is that the materials were placed well dry of Standard optimum moisture content, and were of low plasticity clayey sands to clayey gravels to sandy clays. In all cases the earthfills were reportedly placed in thin layers (150 mm) and well-compacted by rollers.

6.4.2 "High" Shear Stress or Marginal Stability Conditions Within the Embankment

Three of the case studies within the database of embankment with very broad core zones suffered failures in the upstream slope during large drawdown, these were Belle Fourche, San Luis (Von Thun 1988; Stark and Duncan 1987, 1991) and Steinaker (Cyganiewicz and Dise 1997) dams. The failures are clearly indicative of marginal stability conditions of the embankments under drawdown. Hunter and Fell (2002b) discuss the possible mechanism/s of the failures.

The following discussion is on the monitored deformation behaviour of Belle Fourche and San Luis dams, concentrating on "abnormal" or potentially "abnormal" trends that may be potential indicators of an upstream failure condition on drawdown. At both these embankments no SMPs or other monitoring instrumentation was located within the actual failed section of the embankment. At Steinaker dam the failure was in a section of the embankment that was not monitored and was different in design to the main section, so is not discussed further. The mechanism and failure itself at each embankment are not discussed in any detail here.

6.4.2.1 Belle Fourche dam.

Belle Fourche dam (Figure 6.54) is an earthfill embankment of 35 m maximum height and about 1850 m crest length that was constructed over the period 1905 to 1911. Foundations for the embankment consist of medium plasticity alluvial adobe clay overlying a thin sand and gravel layer and in-turn shale bedrock. The earthfill materials used in construction were the medium plasticity adobe clays placed in 150 mm layers, "sprinkled" with water and compacted using heavy rollers with diagonal lugs.

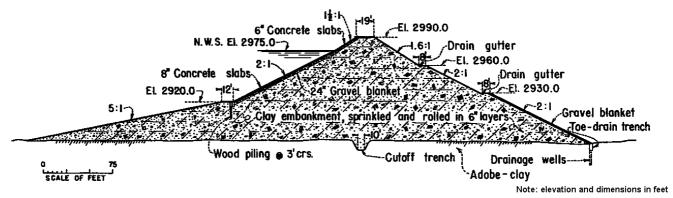


Figure 6.54: Belle Fourche dam; main section as constructed in 1911 (courtesy of U.S. Bureau of Reclamation).

The embankment design incorporated steep upstream and downstream slopes for the type of construction materials used. The steep portion of the upstream slope was faced with concrete slabs placed on a 0.6 m layer

of gravel and supported at the toe (elevation 2920 feet) on a concrete footing and driven wooden piles. The closure section at Owl Creek, the highest section of the embankment between stations 40+50 and 42+50, was constructed last and was built very rapidly.

The reservoir was first filled over the period from March 1911 to September 1915 (4.2 years after construction) and is subjected to a seasonal drawdown, usually over the late spring to early autumn period, generally in the range of 4 to 8 metres (Figure 6.56) with occasional larger event of greater than 9 m.

The embankment has performed relatively poorly during the larger drawdown events (Sherard 1953; USBR 1996). In 1928 (17 years after construction) longitudinal cracking on the downstream edge of the crest (Sherard 1953) was observed in November after the drawdown had been completed. A total of 5 cracks, each 5 to 50 m in length, 12 to 50 mm in width and 1 to 3.5 m deep, were observed along the higher embankment sections, including the closure section. In 1931 (19 years after construction) a slide in the upstream slope occurred during drawdown on 2nd August. The slide was approximately 110 m in width and 3 to 5 m in depth located within the steeper upstream slope between Stations 43 and 46+50. The head of the backscarp was about 7 m below crest level. The slide mass was excavated and the upstream slope rebuilt to its original configuration in 1931.

Remedial works of the steep upstream slope were undertaken in 1939 (following the failure of the upstream slope on rapid drawdown in 1931) and again in 1977 to address concerns over embankment stability under drawdown. In 1939 a gravel stabilising berm was added to the lower upstream slope below elevation 2950 feet, and in 1977 the upper portion of the upstream slope was flattened to a slope of 2.33H to 1V, the crest widened 1.4 m to upstream and the crest re-surfaced.

On large drawdown in 1985 (74 years after construction) longitudinal cracking was observed on the crest between Stations 39 and 46, located 1.2 m from the upstream edge of the crest. The crack, which was centred on the Owl Creek closure section, was about 200 m in length and up to 75 to 100 mm in width. A vertical displacement to upstream of 50 mm was measured across the crack. Investigation found that the crack was coincident with the upstream edge of the original structure, located directly above the buried concrete kerb. On drawdown in 1988 and 1989 (77 and 78 years after construction) the 1985 crack re-opened and the differential settlement to upstream across the crack increased.

During piezometer installations in 1982 softened and very wet zones were encountered within the earthfill and foundation (Hickox and Murray 1983).

Instrumentation for monitoring deformation behaviour at Belle Fourche dam is by surface measurement points (SMP). At end of construction a series of SMPs were installed along the crest in 1911 and were monitored for vertical deformation over a period of about 17 years to 1928 (Figure 6.55), so no monitoring records are available for the large drawdowns in 1928 and 1931. In 1985, some 74 years after construction, SMPs were installed on the crest and slopes between Stations 26 and 46 and the deformation records of the SMPs installed at Station 40 to 42 (within the Owl Creek closure section) are shown in Figure 6.56.

When compared to similar embankments, a number of aspects of the post construction deformation behaviour at Belle Fourche dam are clear outliers and considered as indicative of "abnormal" behaviour. These include:

- The large magnitude of crest settlement in the first 17 years of operation, reaching more than 2% of the embankment height (Figure 4.48 and Figure 4.60).
- The very high crest settlement rate (Figure 4.63a). At 10 to 12 years after construction the rate was about 1.8% per log cycle of time, and increased to about 4.5% per log cycle of time at 75 to 85 years after construction.

- The very high rates of settlement of SMPs on both the upstream (Figure 4.83) and downstream (Figure 4.78a) shoulders within the closure section.
- The very high rate of downstream displacement of the downstream slope (Figure 4.80) and upstream displacement of the upstream slope (Figure 4.87) within the closure section at 75 to 85 years after construction.

Of the SMPs installed on the crest and slopes of the embankment after 1985, the deformations between Stations 40 and 42 (within the Owl Creek closure section) were up to 1.5 to 2 times greater than those measured elsewhere on the embankment, and were very much larger at MP5 (located at Station 42+00) on the upstream slope. During large drawdown at 74 years (1985) and again at 77 years (1988) acceleration in settlement and non-recoverable upstream displacement is observed at MP5 on the upstream slope. This is coincident with the observation of cracking and greater settlement of the upstream edge of the crest. At 86 years further non-recoverable upstream displacement was observed for MP5.

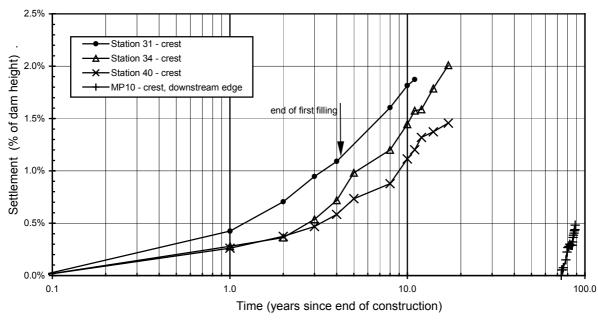
The embankment performance during large drawdown is indicative of the marginal stability of the upstream slope on drawdown, as evidenced by the failure in the upstream slope during the 1931 drawdown.

USBR (1996) attributes the observed cracking and monitored "abnormal" deformation behaviour of the upstream slope to upstream crest region during the drawdowns in 1985, 1988 and 1989 to settlement / consolidation of the original earthfill under the added weight of the granular filling in the upstream shoulder berm placed in 1977. However, they do not discount the potential for deep-seated movements. A notable aspect of the deformation behaviour of the upstream slope during large drawdown is that only at MP5 (located within the Owl Creek closure section) is the acceleration in settlement and displacement observed, no such behaviour is observed at other SMPs on the upstream slope also located on the newly placed granular filling.

The deformation records after 75 years show relatively similar behaviour for the crest and downstream slope, with average settlement rates of 6 to 10 mm/year and downstream displacement rates of 6 to 13 mm/year. An increase in displacement rate of SMPs on the crest and downstream slope is observed after 82 years, and is approximately coincident with a period of higher average reservoir level. The increase in settlement rate of the downstream edge of the crest and downstream slope after year 85 (1996) may reflect a change in effective stress conditions due to a rising phreatic surface under the high average reservoir level.

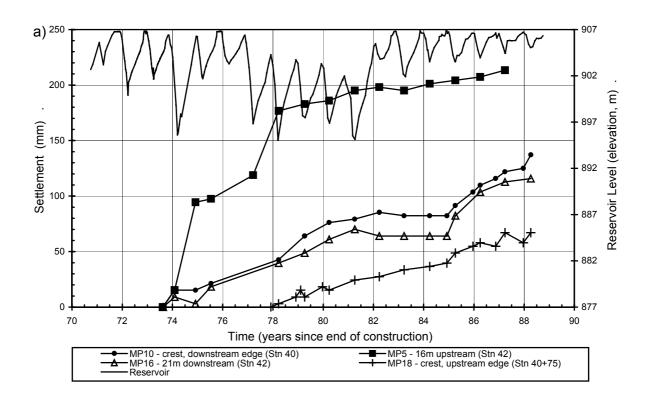
Overall, the long-term deformation behaviour at Belle Fourche dam is "abnormal" in comparison to similar embankments. The "abnormal" deformation of the crest and slopes, particularly in the vicinity of the Owl Creek closure section (Stations 40 to 42), possibly reflects the "highly stressed" conditions within the embankment due to the steepness of the embankment slopes for an earthfill embankment constructed of medium plasticity clays. An important aspect of the deformation behaviour is the much higher crest settlement rates (per log cycle of time) from recent monitoring compared to those at 10 to 12 years after construction. This possibly reflects softening of the undrained strength properties of the earthfill due to wetting of the earthfill, strain weakening under the high stress conditions imposed during large drawdown, lateral spreading in the crest region and cracking in the upper portion of the embankment.

The USBR comment that the Owl Creek closure section, which was constructed very rapidly, is a definite discontinuity along the dam embankment and that the deformation behaviour within this closure section is somewhat unique to the rest of the embankment. This is indicated by the greater magnitudes of deformation measured for the SMPs on the crest and slopes of the embankment installed more than 75 years after the end of construction.



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Figure 6.55: Belle Fourche dam; post construction crest settlement over the period 1911 to 1928 (to 17 years post construction).



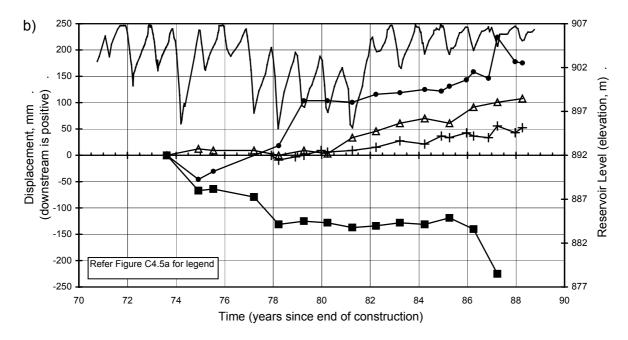


Figure 6.56: Belle Fourche dam closure section; post construction (a) settlement and (b) displacement normal to dam axis of the crest and slopes over the period 1985 to 2000 (73 to 89 years post construction).

6.4.2.2 San Luis dam

San Luis dam is a zoned earth and rockfill embankment with very broad central earthfill zone constructed in the 1960's. The embankment is of 116 m maximum height and 5650 m crest length. Foundation conditions vary along the length of the embankment, from deep alluvial deposits in the floodplain area to bedrock at shallow depth on hillslopes. Changes to the embankment design were made according to the foundation conditions (refer Section 2.2 of Appendix B). A section at the slide area is shown in Figure 6.57.

The very broad central earthfill core was constructed using mainly low to medium plasticity sandy clays, sourced from alluvial terrace and floodplain deposits, compacted by tamping rollers in 150 mm layers to high density ratio at moisture contents on average 1.2% dry of Standard optimum. The miscellaneous fill zone (Zone 3) in the outer downstream shoulder and upstream toe regions comprised materials ranging from Zone 1 type earthfills to weathered rock, and were placed and compacted in 300 mm layers.

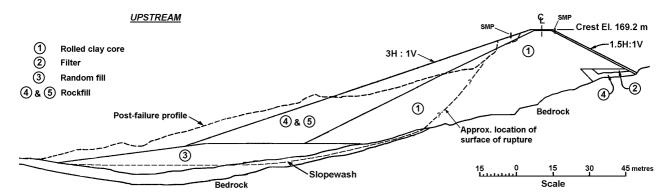


Figure 6.57: San Luis dam, section of the slide in the upstream slope in 1981 at Station 135+00 (courtesy of United States Bureau of Reclamation).

Full supply level was reached in July 1969, 2.1 years after construction, and post first filling the reservoir has been subjected to a seasonal drawdown ranging in magnitude from 5 to about 65 m (Figure 6.60a).

In September 1981, some 14 years after construction, a slide occurred in the upstream slope during large drawdown (Von Thun 1988; Stark and Duncan 1987, 1991). The deep-seated slide (Figure 6.57) was located on the left abutment in a region where the original ground surface under the embankment sloped in an upstream direction at about 10 to 15 degrees. It was approximately 460 metres in width (from Station 122 to 137), 1 million cubic metres in volume, and slid a distance of about 20 m.

Von Thun (1988) considered that the persistent, but minor, longitudinal crest cracking over the 14 years up to the slide and settlement behaviour of SMPs on the upstream at Stations 136 and 138 as indicators of "abnormal" deformation behaviour prior to the slide and precursory warning signs of potential instability. Although, he added that, actual prediction of the timing of the slide was not possible from the monitored deformation.

Von Thun (1988) suggests that in the area of the slide the crest cracking "was associated with the saturation and progressive straining of the slopewash on the hillsides". But, the longitudinal crest cracking was not confined to the region of the slide, it was more of a general occurrence along the length of the embankment. This is confirmed by comparison of the displacements along the line of SMPs upstream of the dam axis with those downstream of the dam axis, which showed that leading up to the slide lateral spreading of 50 to 150 mm occurred along most of the embankment length. In addition, most of the differential displacement in the region of the slide occurred during first filling (refer Section 2.2 of Appendix B). However, there is some query over the "original zero" and the accuracy of the lateral displacement measurements in the 1960's and 1970's. For the overall embankment it could be said that the longitudinal crest cracking is likely to be associated with differential deformations due to progressive development of the phreatic surface in the dry placed and brittle earthfill, and the stress conditions developed in the earthfill. Therefore, it is difficult to consider the crest cracking as an indicator of "abnormal" deformation behaviour and a precursor to slope instability in this case, unless persistence of the cracking was confined to the vicinity of the slide.

With respect to the settlement behaviour of SMPs on the upstream slope, Von Thun (1988) comments that the very large differential between the actual and predicted post construction settlements at Stations 136 and 138 (Figure 6.58) was a clear indicator of the "anomalous" behaviour in the region of the slide prior to the failure. The SMPs on the upstream slope at Stations 136 and 138 are located several metres above and therefore outside of the initial slide area as shown in Figure 6.57.

Figure 6.59 presents the actual post construction settlements measured to the date of the last reading prior to the slide for SMPs at 13 m upstream and those at 6.5 downstream of the dam axis. The figure shows that at most locations the measured settlements up and downstream of the crest are similar, however, a differential is notable for the SMPs from Station 130 to 140, in particular at about Station 136 where the slide initiated. In addition, the settlement versus time plot of SMPs in the region of the slide (Figure 6.60) shows that during the first large drawdown period from 8 to 10 years after construction an acceleration in settlement rate was measured for the SMPs on the upstream slope. When the settlement is normalised with respect to the height from the SMP to foundation level (Figure 6.60b) the magnitude of the settlement during drawdown was greater for the SMP at Station 136. This trend in the settlement behaviour might be considered "abnormal" in comparison to other SMPs on the upstream slope at San Luis dam both in the slide vicinity and elsewhere along the embankment.

In summary, the settlement behaviour of the SMP on the upstream slope at Station 136, located in the region of but not within the slide area, during the first large drawdown appears "abnormal" compared to similar embankments and more importantly to other SMPs on the upstream slope at San Luis dam. During the next large drawdown in 1981, at 14 years after construction, the slide in the upstream slope occurred. In hindsight, the settlement behaviour of this SMP could be considered as a precursory sign of the failure, although, it would

be almost impossible to predict the occurrence of the slide based on the deformation records as concluded by Von Thun (1988).

The longitudinal crest cracking at San Luis dam is not considered as a reliable indicator of "abnormal" deformation behaviour and precursory sign of slope instability in this case because of its widespread occurrence along the embankment.

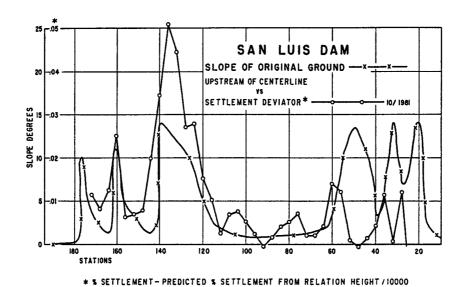


Figure 6.58: San Luis dam, difference between actual and predicted settlement (Von Thun 1988)

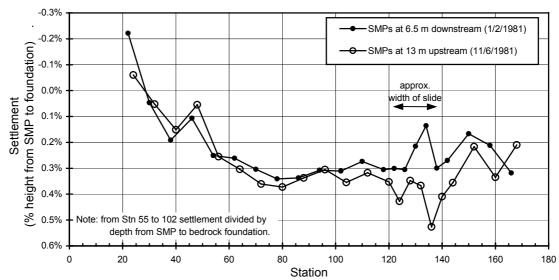


Figure 6.59: San Luis dam, comparison of settlement between SMPs at 13 m upstream of dam axis and SMPs at 6.5 m downstream.

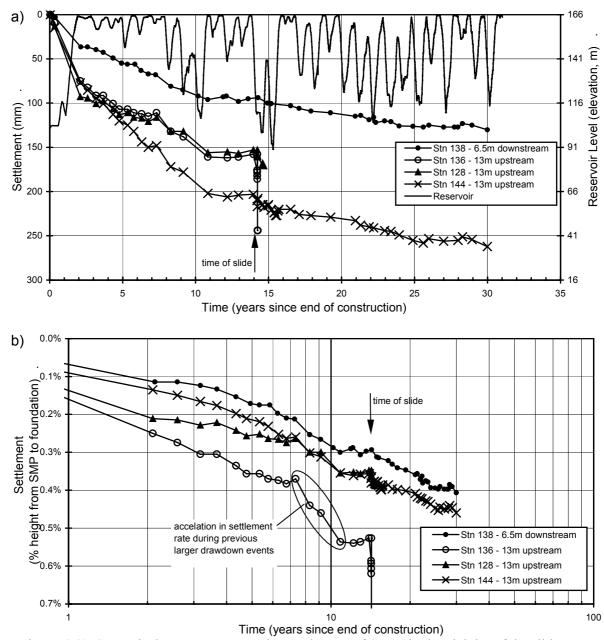


Figure 6.60: San Luis dam, post construction settlement of SMPs in the vicinity of the slide area.

6.4.3 The Effect of the Development of the Phreatic Surface on the Displacement of the Crest and Downstream Shoulder.

For a number of the earthfill embankments or zoned embankments with very broad core width the displacement of the crest and/or downstream shoulder shows an increase in the rate of displacement long-term (Figure 4.76 for the crest, and Figure 4.80 for the downstream shoulder). The affected case studies include Horsetooth, Soldier Canyon, Spring Canyon, Dixon Canyon, Pueblo (both left and right abutment embankments), San Luis and Belle Fourche.

For a number of embankments of very broad core width constructed of low permeability earthfills the phreatic surface developed gradually within the embankment over tens of years (refer Section 4.3.3 item f). Changes in the reservoir operation are also likely to influence the phreatic surface. Maintaining the reservoir at higher elevations for longer periods that previously experienced will result in a gradual rise in the phreatic surface. The effect of a slow increase in the phreatic surface will cause a gradual change in the effective stress conditions within the embankment and softening of the material strength properties as the degree of saturation increases, particularly if the core is yielded or dilated. These changes are likely to influence the deformation behaviour and are considered a factor in the unusual observation of higher rates of displacement post first filling.

At Belle Fourche dam the increase in displacement rate of SMPs on the crest and downstream slope is observed after 82 years, and is approximately coincident with a period of higher average reservoir level (Figure 6.56). In addition, the increase in settlement rate of the downstream edge of the crest and downstream slope after year 85 may also reflect a change in effective stress conditions due to a rising phreatic surface under the high average reservoir level.

At all the Horsetooth Reservoir embankments (refer Section 3.3 in Appendix B), an increase in displacement rate to downstream was measured many years after the end of first filling for SMPs on the crest and downstream slope. USBR (1997) indicate that the pore water pressures within the earthfill zones of all embankments were still rising more than 50 years after construction. The increase in pore water pressures suggests that equilibrium conditions are yet to be reached, indicating that progressive wetting up is still occurring.

6.4.4 Other Cases of Potential "Abnormal" Deformation Behaviour

6.4.4.1 Horsetooth dam

As previously discussed, the foundation had a significant influence on the post construction deformation behaviour at Horsetooth dam. The embankment design over the broad gully region (Figure 6.61) was such that the bulk of the embankment was founded on the over-burden soils and the cut-off trench to bedrock was located upstream of the dam axis. The depth of over-burden soils in the broad gully region was up to 10 to 20 m.

Settlements in the foundation under the centreline of the embankment were very large, estimated at almost 15% of the depth of the over-burden (500 to 1000 mm at the IVM locations). Post construction, the settlement of the foundation was also very large (325 mm at IVM A or up to 5% of the depth of the over-burden over nearly 50 years), most of which (about 70%) occurred on first filling. The post construction vertical strains in the foundation were very much larger than in the earthfill, where they average 1.05% at 45 years after construction.

The large differential settlement between the foundation and earthfill in the cut-off had the effect of causing a downstream rotation of the embankment downstream of the cut-off trench. As shown in Figure 6.62, the upper

upstream shoulder to downstream shoulder displaced downstream during the period of first filling from 2 to 4.5 years after construction.

Compared to similar embankments the displacement at Horsetooth dam on first filling is an outlier (Figure 4.38 to Figure 4.40), but not "abnormal" because of the influence of the compressible foundation.

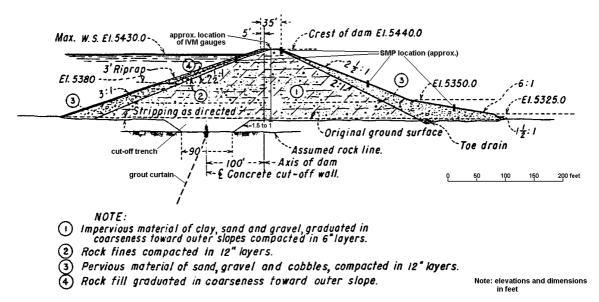


Figure 6.61: Main section at Horsetooth dam (courtesy of the United States Bureau of Reclamation).

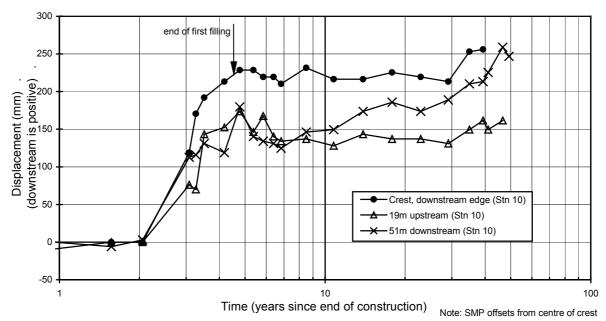


Figure 6.62: Horsetooth dam, post construction displacement of SMPs near to the main section.

6.4.4.2 Pueblo dam, left abutment embankment

The left embankment at Pueblo dam (Figure 6.63) is of 37 m maximum height and about 1100 m crest length. The main earthfill zone consisted of clay to gravel size alluvium placed in 150 mm layers and compacted by tamping rollers. The moisture specification called for the mean moisture content to be in the range 0.5% dry to 1.5% dry of Standard Proctor optimum moisture content. Coarser and more permeable soils were used in the outer up and downstream regions of the main earthfill zone.

During construction (in the early to mid 1970's) shear type deformations were measured in the downstream toe region of the left embankment when the embankment was within about 7 m of design crest level. Inclinometers showed that the deformations were concentrated along weak bentonitic clay seams in the foundation. By end of construction total shear type displacements were estimated (by the USBR) at more than 150 mm at the downstream toe of the left embankment. The shear deformations virtually ceased shortly after the end of construction. In the early 1980's a stability berm was constructed along the downstream toe of the left abutment embankment due to concerns over the potential limiting stability of the downstream slope.

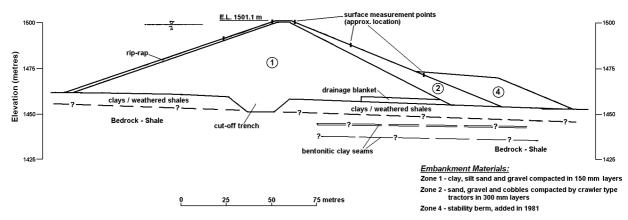


Figure 6.63: Pueblo dam; cross section of left abutment embankment at Station 75

Post construction settlements of the downstream shoulder of the left abutment embankment at Pueblo dam were large (Figure 6.64), and in comparison to similar type embankments the magnitude is "abnormally" high (Figure 4.50 and Figure A2.10a of Appendix A). The long-term settlement rate of the downstream shoulder is on the high side, but not "abnormally" so (Figure 4.78).

In the vicinity of Station 75 borehole records indicate the soils are mainly weathered shales, for which the post construction deformation is likely to be limited and therefore not greatly influence the deformation behaviour of the embankment. In addition, the shear type deformations observed in the foundation during construction had virtually ceased at the end of construction, and therefore would not influence the post construction deformation behaviour of the downstream shoulder with the stability berm added.

As Figure 6.64 shows, the first settlement reading after the base survey of the SMP on the downstream slope is very high (0.75% or almost 150 mm) and contributes significantly to the large post construction settlement (42% of the total settlement measured over 23 years). After this first reading the settlement behaviour is similar in magnitude to other SMPs on the embankment crest and upstream slope, although the settlement rate is slightly higher. Other SMPs along the downstream slope of the left embankment abutment show similar settlement behaviour to that at Station 75.

It is not clear what the cause of this observed settlement behaviour for the downstream slope is. The influence of construction of the stability berm is not the cause because construction started in 1980 some 5 to 5.5 years after end of construction, after the period of large settlement. It is possible that it could be survey error. Apart from the settlement of the downstream shoulder, the post construction deformation behaviour at the left abutment embankment of Pueblo dam is "normal". Further details on the deformation behaviour at Pueblo dam are discussed in Section 3.4 of Appendix B.

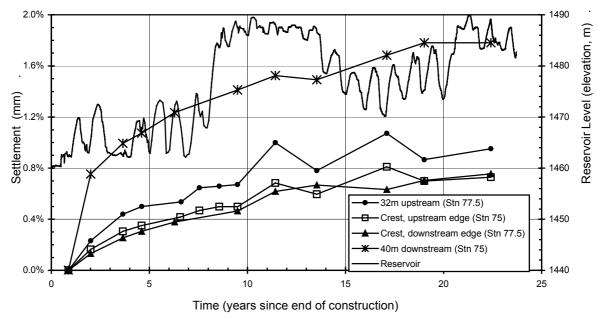


Figure 6.64: Pueblo dam, post construction settlement of the left embankment near Station 75.

6.5 "ABNORMAL" DEFORMATION BEHAVIOUR OF PUDDLE CORE EARTHFILL DAMS

For puddle core earthfill dams, trends of "abnormal" deformation behaviour are specific to the long-term deformation behaviour given the period of construction of these embankment types. Identification of "abnormal" deformation behaviour is from comparison with the deformation behaviour of other puddle dams, mainly in terms of rates of deformation, as well as the deformation trend in relation to the creep model under constant stress conditions. The limited number of records from the end of construction does not allow for comparison based on the magnitude of deformation.

As previously discussed, what may initially be termed "abnormal" may later be proven by investigation and additional monitoring to in-fact be "normal". In the case of puddle core earthfill embankments, the large deformations observed during historically large drawdown events (Section 5.3.5) could well be considered as "abnormal" when compared to the deformation trends under normal reservoir operating conditions.

6.5.1 Comparison with the Deformation Behaviour of Other Puddle Dams

Most of the available data to assess the long-term deformation behaviour of a puddle core earthfill embankment relates to crest settlement (Table 5.1, Table D1.1 in Appendix D). There is some, but limited, data on settlement of the shoulders and at the downstream toe, and on horizontal displacement.

In terms of crest settlement, comparisons can be made as follows:

- Total settlement from the end of construction (Figure 5.5). Although, only limited data is available for comparison. The age of the embankment, embankment height, compaction methods and moisture control affect the total settlement. From Figure 5.5:
 - For dams constructed after about 1900, and particularly after about 1930, the total settlement at about 30 to 50 years is less than 4% of the total height of the embankment. In comparison, the total crest settlement of Hollowell Dam (completed in 1937) stands out as being "abnormal" and is discussed later.
 - For older puddle dams, there is insufficient information to make a judgement on what could be expected as "normal" total settlement. Yan Yean and Hope Valley dams show total vertical settlements of 8 to 13% of the embankment height at more than 100 years.

- The long-term crest settlement rate, S_{LT} , under normal reservoir operating conditions. As discussed in Section 5.3 long-term rates of crest settlement depend on a number of factors, in particular the level of fluctuation of the reservoir and the pore pressure response in the earthfill zone upstream of the puddle core.
 - For "steady state" conditions S_{LT} is typically less than 1% per log cycle of time (settlement as a percentage of the embankment height). "Steady state" conditions are defined as virtually steady reservoir operating levels or, in the case of fluctuating reservoir level, negligible to minor changes in pore water pressure in the earthfill zone upstream of the puddle core to fluctuations in the reservoir level.
 - For embankments subject to reservoir fluctuation and in which the upstream earthfill is permeable (i.e. the changes in pore water pressure in the earthfill zone upstream of the puddle core approximate the fluctuations in reservoir level), S_{LT} rates of 2.6% to 7.4% per log cycle of time have been measured, and are considered to be "normal".
 - Figure 5.11 presents data on crest settlement versus drawdown height for dams subject to reservoir fluctuations greater than 20% of the dam height and in which the earthfill zone upstream of the puddle core is permeable. The data has been adapted from Tedd et al (1997b). This figure shows some correlation between height of drawdown and permanent crest settlement, and could be used as preliminary tool for assessment of "abnormal" deformation. However, the use of Figure 5.11 is limited given that the data set comprises older UK dams (pre 1910) of similar height and dimensions, and constructed using similar materials.

Other than for crest settlement, the deformation data on puddle core earthfill dams is limited due to the limited data available. Comparisons based on settlement of the embankment shoulders are not possible. However, assessment of the horizontal displacement of the crest and downstream slope is possible.

The post construction deformation behaviour of earthfill and earth-rockfill dams indicates that the general displacement trend of the crest and downstream shoulder is for downstream displacement at a decreasing rate (on log time scale) with time, eventually reaching a point where the net displacement is negligible with some fluctuation about the general trend due to fluctuations in reservoir level. A similar analogy is possible for puddle dams. The limited amount of available data tentatively suggests:

- Negligible long-term rates of downstream displacement of the crest and downstream slope for case studies at "steady state" conditions.
- Long-term displacement rates of the downstream crest and slope of up to 3 to 5 mm/year downstream for embankments where the upstream shoulder is relatively permeable and the reservoir is subject to relatively large drawdowns (more than about 20% of the embankment height) and the fluctuation is within the range of normal reservoir operation.

The deformation behaviour of the following case studies is considered to be "abnormal" based on comparison with other puddle dams:

• The crest settlement of Hollowell dam in the period shortly after embankment construction. The magnitude of total crest settlement of Hollowell dam in comparison to puddle dams of similar age is significantly greater (Figure 5.5). At 10 years after the end of construction crest settlement at Hollowell dam was more than 6% compared to less than 2% for Selset and Burnhope dams. Kennard (1955) indicates the large deformations at Hollowell dam were due to the marginal factor of safety of the downstream slope at the end of construction as a result of the high pore water pressures in the foundation. Localised shear type deformations occurred during construction and measures were undertaken to improve the factor of safety. The long-term settlement rate for Hollowell dam more than 10 years after the end of construction is about 0.7%, which is comparable to dams of similar characteristics.

- The high long-term rate of settlement of the downstream edge of the crest at Yan Yean dam measured more than 130 years after the end of construction (refer Section 2.2 of Appendix D for more detailed comments on the deformation behaviour at Yan Yean dam). Yan Yean dam has been categorised in the class of "steady state" embankments based on the minor pore water pressure response in the earthfill zone upstream of the puddle core compared to the change in reservoir level. S_{LT} rates for the crest would be expected to be less than about 1% per log cycle of time, which is the case for the upstream edge of the crest. However, for the downstream edge of the crest S_{LT} is a maximum of 11% over a period of 14 years from 128 to 142 years after construction, much greater than would be expected of "normal" type behaviour. High rates of settlement and displacement of the downstream crest and slope were also observed over this period of monitoring (up to 8 to 10 mm/year settlement, and up to 4.5 - 5.5 mm/year downstream crest displacement). These high rates are also considered indicative of "abnormal" deformation behaviour. The increasing rate of deformation with time of several SMPs (refer Section 2.2 of Appendix D) is also a strong indicator of "abnormal" deformation behaviour. The deformation behaviour of the downstream slope is considered to be independent of the reservoir level operation and possibly indicative of a tertiary creep (or creep to failure) phase of movement (refer Section 6.5.2). Investigations showed fissured clays in the foundation were contributing to a marginal stability condition, and a berm has now been constructed to improve stability under normal operating conditions and earthquake, and internal erosion and piping control.
- For Hope Valley dam the rate of downstream displacement of the upper downstream slope of 5 mm/year over the monitoring period from 118 to 128 years after construction (refer Section 2.3 of Appendix D) maybe on the high side, but would not be classified as "abnormal" from the available data. The deformation behaviour of Hope Valley dam could be indicative of potential instability of the downstream slope. Hope Valley dam has also been remediated with a downstream berm (Gosden et al 2002).

6.5.2 General Movement Trends Indicative of Deformation to Failure

From the basic creep model of time dependent deformation under constant deviatoric stress conditions (Singh and Mitchell 1968; Mitchell 1993), an increasing rate of deformation under constant stress is indicative of a deformation to a failure condition (i.e. a tertiary creep phase of deformation). A localised trend of increasing rate of deformation at one SMP may indicate shallow surficial deformations or a faulty SMP, and not a marginal stability condition and potential failure condition. It is therefore important that a similar deformation trend is evidenced by other SMPs on the embankment in the close proximity to each other and that a vector plot of the deformation at a section is indicative of a potential deep-seated movement.

The deformation trends of several SMPs on the downstream crest and slope of Yan Yean dam (Figure 6.65) show an increasing rate of deformation, indicating a potential tertiary creep mode of deformation, that is considered to be independent of reservoir level fluctuation. The possible mechanisms involved in the deformation behaviour of the downstream slope are considered to be a combination of stress changes at the top of the slope due to moisture infiltration into cracks when the reservoir is in a drawdown condition and to progressive failure of the foundation or saturated base of the embankment. It would appear that stick-slip type deformation behaviour of the downstream toe at several locations indicates that the movements are being driven from the top of the slope resulting in build up of stresses in the foundation (or saturated base of the embankment). Continued movements of this type within the over-consolidated foundation could result in strain weakening and a progressive failure. As previously mentioned a downstream berm has now been constructed to improve stability.

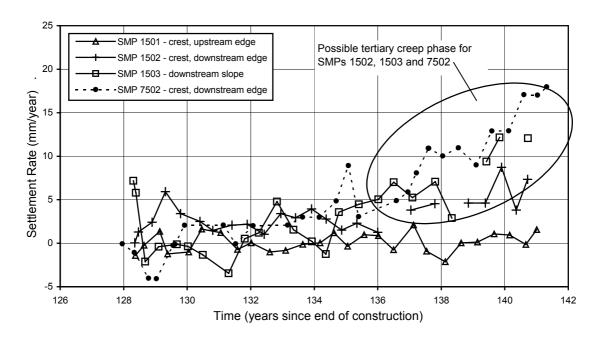


Figure 6.65: Yan Yean dam; settlement rate versus time of SMPs at Chainage 150 and 750 m.

6.5.3 Other Indicators of "Abnormal" Deformation Behaviour

Several other examples from the long-term monitoring trends of puddle core earthfill embankment case studies are considered to be indicative of possible "abnormal" deformation behaviour. These were observed at Ramsden and Hope Valley dams, both of which are discussed in detail in Appendix D.

For Ramsden Dam, the abnormal deformation behaviour is related to the internal deformation of the puddle core on the abnormally large drawdown in 1988 to 1989. It is considered possible that shearing occurred in the puddle core between 10 m and 11 m depth below the crest as indicated by the localised high vertical strain and possible upstream shear displacement at this depth. This is associated with a permanent settlement of 51 mm at the crest, smaller permanent settlements of the upstream slope (2 to 10 mm) and 7 mm permanent upstream displacement of the crest.

For Hope Valley dam (Appendix D, Section 2.3) the displacement rate of the upper downstream slope (5mm/year) is considered to be relatively high in comparison to other puddle dams. The rate is similar to that of Yan Yean dam, although the rate at Hope Valley dam was virtually constant and not increasing. In comparison to the crest of Walshaw Dean and Ramsden, which also shows a steady downstream displacement rate, the rate is higher at Hope Valley dam, however this may be due to type of material used as shoulder filling and possibly a lower factor of safety of the downstream slope.

Based on the available deformation records for Hope Valley dam and in comparison with other puddle dams, it is considered that a significant proportion of the movement of the downstream slope can be attributed to long-term creep and cyclic stress changes associated with reservoir level fluctuations. However, it is not possible to conclude that these are solely the explanation for the movement without considering the possibility of slope instability. The discerning factor is the high and constant rate of downstream displacement.

6.6 SUMMARY OF "ABNORMAL" DEFORMATION BEHAVIOUR

The purpose of identification of "abnormal" deformation behaviour of embankment dams is for the early detection of potential problems, mainly with respect to slope instability. But, it may also have implications with

respect to internal erosion and piping because the deformations may lead to cracking and softening. Following identification of "abnormal" deformation behaviour it is necessary to understand the mechanism/s causing the observed behaviour, which may incorporate investigation and analysis, and may eventually lead to some form remedial works. In most cases "abnormal" deformation behaviour does not equate with stability issues for the embankment, but in a number of cases it has.

The methods of identification of "abnormal" deformation behaviour from the case study analysis are mainly as outliers to the "normal" deformation behaviour of similar embankment types in terms of magnitude, rate and trends. In summary, the methods are:

- During construction (for earthfill and zoned earth and earth-rockfill embankments), outliers in terms of:
 - Total core settlement during construction (Figure 4.23, Table 4.6 and Table 4.7).
 - Magnitude of vertical strain within the core at a given depth or effective vertical stress level (Figure 4.18, Figure 4.20 and Figure 4.21).
- Post construction, outliers in terms of:
 - Magnitude of settlement or displacement.
 - Settlement rate (rate in terms of log time) including:
 - Magnitude of the settlement rate
 - Acceleration in rate over short periods of time, often, but not always, post first filling on drawdown.
 - Displacement:
 - Direction of displacement
 - Magnitude of the long-term displacement rate (rate in terms of log time)
 - Change in direction of the underlying general trend, i.e. the trend outside of that due to reservoir fluctuation.
 - Short periods of non-recoverable displacement post first filling, such as a permanent upstream displacement on large drawdown.
- Development of localised regions of high strain (during or post construction) indicative of the formation of a shear surface, and the ongoing deformation within these regions. The case study analysis concentrated on vertical strains in the core region of the embankment, but this would be equally applicable to concentration of lateral strain in the core or foundation measured in inclinometers.
- The tertiary creep analogy from the model of creep under constant stress conditions. Tertiary creep (or creep to failure) is creep at an increasing rate (rate in terms of normal time) with time and is an indication of the onset of failure. Primary creep, or creep at a decreasing rate (rate in terms of normal time) with time is indicative of "normal" type behaviour.

Several important aspects on the deformation behaviour should be noted:

- For zoned earth and rockfill dams, "abnormal" or potentially "abnormal" deformation behaviour was much more likely in embankments where the rockfill was susceptible to large settlements due to collapse compression on wetting or was of high compressibility. These include rockfills that are poorly compacted, dry placed and poorly to reasonably compacted, dumped and sluiced rockfills, weathered rockfills, and rockfills of rock type susceptible to large loss in unconfined compressive strength on wetting. Conversely, sound rockfills that are wetted and well and reasonably to well compacted are, in most cases, not susceptible to large collapse compression on first filling and the overall post construction deformation behaviour of the embankment is "normal" and generally of limited magnitude.
- The development of a shear zone in the earthfill core does not equate with a marginal factor of safety, although it may. The shear zone may be a result of differential settlement between the core and shoulder or lack of support from the shoulders. The core types within which shear surfaces developed (and the timing of the shear development) included:

- Compacted silty sands to silty gravels. Shear surfaces in the core generally developed during first filling or with further movements shortly thereafter (often drawdown related). High shear stresses at the interface between the core and upstream rockfill shoulder that developed due to the greater settlement of the upstream shoulder as it collapse compressed on wetting were considered the mechanism in most cases for the shear development.
- Dry placed clayey sand earthfills. Shear surfaces in the core generally developed during first filling with further deformations associated with reservoir operation.
- Dry placed sandy clay and clay earthfills. Shear surface developed on first filling (e.g. Ataturk dam) or many years post first filling, usually on large drawdown (see below).
- Wet placed clayey earthfills. Examples of shear surface development during construction and on first filling, as well as post first filling.
- Longitudinal cracking does not equate with a marginal factor of safety. In most cases it may simply be due to differential deformation between the zones in the embankment and may only occur during the period of and shortly after first filling. However, persistent longitudinal cracking may be indicative of a marginal stability condition (refer below).
- Acceleration of the deformation of the crest and upstream slope on large drawdown and resultant permanent
 deformations do not equate with marginal stability, but they may. These type of deformations are not
 uncommon on historically large drawdowns where yielding may occur under effective stress levels not
 previously experienced (such as observed in several puddle dams). Persistent observations of acceleration
 of the deformation on large drawdown may be indicative of a marginal stability condition (refer below).
- For several central core earth and rockfill embankments with dry placed clay cores and dry placed and/or poorly compacted rockfills, shear surfaces developed (or were thought to have potentially developed) in the central region of the core many years after the end of construction. In the case of Eppalock dam it was considered that significant cracking in the core preceded the shear development as indicated by observed cracking and/or the change in displacement trend of SMPs on the crest. The cracking prior to shear development was considered a significant factor in the development of the shear surface, which was first identified during a large drawdown (not necessarily the first large drawdown). This type of shear surface development was considered to have possibly developed at several other similar type embankments (i.e. dry placed clayey core with rockfill shoulders susceptible to collapse compression on wetting).

The issue of potential instability or marginal stability of the embankment is the foremost aspect of any deformation monitoring that is undertaken given the potentially catastrophic consequences of a slope failure condition. Some guidelines from the analysis of those case studies where failure occurred or where the embankment was considered to be in a marginal stability condition are:

- Instability during construction:
 - Deformation behaviour during shutdown periods. Large and ongoing deformations during shutdown or acceleration in the rate of deformation (rate in terms of normal time) may indicate marginal stability or an impending failure condition.
 - The incremental magnitude of deformation with increasing dam height or stress level (refer Penman 1986). High or increasing (e.g. Carsington dam) incremental magnitudes of deformation may be indicative of a marginal stability condition or the onset to a failure condition.
 - Localised regions of "abnormally" high strain as measured in internal settlement gauges or inclinometers may be an indicator of internal shear and potentially a progressive failure mechanism.
 Increases in the incremental magnitude of localised strain may be indicative of the onset to a failure condition.
- Post construction on drawdown:
 - Persistent development of cracking on large drawdown

- Persistent acceleration in settlement and/or displacement of SMPs on the crest and/or upstream shoulder on large drawdown.
- A similar or increasing magnitude of non-recoverable deformation on consecutive large drawdowns of similar magnitude (consecutive meaning 1, 5, 10 or more years between drawdowns of similar magnitude interspersed with smaller magnitude drawdowns).
- Acceleration in settlement or displacement that is confined to one region of the embankment, such as observed at San Luis dam in the region of the slide in the large drawdown preceding the one that triggered the slide (refer Section 6.4.2.2).
- Localised regions of high magnitude deformation compared to similar locations elsewhere on the embankment or compared to predicted deformations (Von Thun (1988), for San Luis dam).
- Post construction, downstream shoulder:
 - Tertiary creep phase of deformation (e.g. Yan Yean dam).

Hunter and Fell (2002b), from a database of 53 failures in embankment dams (excludes failures in hydraulic fill dams), discuss the mechanics associated with failures in embankment dams. At failure, the conversion of a significant portion of the potential energy into kinetic energy is required for acceleration of the slide mass and large post failure deformation. Factors contributing to this conversion in energy for the failure case studies included:

- Potential for material strain weakening on shearing at or after failure in drained or undrained loading conditions (including static liquefaction of structured soils in undrained loading).
- Internal brittleness in the slide mechanics, such as due to internal brittleness or toe buttressing, and/or brittleness on the lateral margins.
- The slope failure geometry and orientation of the surface of rupture.

The deformation behaviour has important implications on the embankment performance related to the potential for internal erosion and piping, as well as the long-term stability of the embankment. With respect to internal erosion and piping a number of aspects are considered to increase the potential for formation of a seepage path through the core of a zoned embankment, including:

- Cracking. The influence of cracks across the core is readily apparent. The possible causes of cracking associated with deformation behaviour are discussed by others (Sherard 1973; amongst others). In addition to these mechanisms of crack formation, localised large deformations are likely to give 3D cracking that may persist through the core. Also, further differential deformations post construction, such as between the core and shoulders or core and foundation, can open up or widen existing cracks.
- Further differential deformations post construction, such as between the core and shoulders or core and foundation, can result in further arching and reduction in the stress conditions within the core, thereby increasing the potential for hydraulic fracture.
- Shear or softened zones that persists across the width of the core are also potential seepage paths.

It is evident from the deformation behaviour at a number of embankments that the softening process in ongoing and can lead to deterioration of the embankment over time, and potentially a gradual reduction in the factor of safety of the embankment's stability. The clearest example is evidenced by the timing of upstream slope instability on drawdown in rolled earthfill embankments (e.g. Belle Fourche and San Luis dams) where the embankment was subjected to several large drawdown events before the failure occurred. Part of the mechanism associated with these and other upstream slope failures is considered to be progressive strain weakening in undrained loading under cyclic reservoir operation within the well-compacted over-consolidated rolled earthfill shoulder. This mechanism was recognised by Stark and Duncan (1987, 1991) as significant in the failure at San Luis dam and is reflected in the composite shear strength concept recommended by Duncan et

al (1990) for limit equilibrium analysis under drawdown. Other softening processes associated with the deformation behaviour of embankment dams are:

- Lateral spreading and/or cracking of the core. Lateral spreading (and therefore cracking) in central core earth and rockfill dams is more significant where the rockfill is susceptible to large settlements from collapse compression on wetting. Part of the reason for this is considered to be due to the transfer in stress to the core associated with the greater settlement of the shoulders, and due to the reduction in support that is provided by shoulders that have suffered large deformations.
- In earthfills susceptible to collapse compression on wetting, the reduction in shear strength and compressibility properties is significant. These earthfills are virtually normally consolidated on softening due to wetting or post collapse. In the case of Hume dam (Cooper et al 1997) a significant factor in the marginal stability of the downstream shoulder of the concrete core-wall earthfill embankment at near full supply level was the low undrained shear strength of the near normally consolidated saturated earthfill in the lower portion of the downstream slope.

In limit equilibrium analysis therefore, consideration should be given to the potential for softening and the influence of the embankment deformation behaviour on the shear strength properties of the embankment materials. Some guidelines are:

- Use the method by Duncan et al (1990) for analysis of the upstream shoulder under drawdown. It takes into consideration the effects of progressive strain weakening of over-consolidated earthfills under cyclic reservoir operation.
- For zoned earth and rockfill embankments where the shoulders are susceptible to large settlements associated with collapse compression on wetting, the use of fully softened c' and ϕ' parameters is wise for over-consolidated earthfill cores because of the potential for softening from lateral spreading and cracking (in addition to the softening on wetting from reservoir seepage).
- For earthfills susceptible to collapse compression (refer below) the collapsed earthfill is likely to be near normally consolidated. The potential for development of shear induced positive pore water pressures should be considered in the stability analysis and this may require the use of undrained strength rather than a conventional *c'*, φ'analysis.
- Where a concentrated shear is developed residual strength parameters are appropriate. Concentrated shears have developed within the core zone in a variety of core types from silty sands/silty gravels to high plasticity clays, and in both over-consolidated dry placed clays and wet placed clay cores of low undrained strength. Consideration should also be given to the potential for strain localisation near to the interface between zones (e.g. in the core near to interface with the shoulder, such as observed at Ataturk dam (Cetin et al 2000)) and the use of residual strength parameters for these shear zones.

It is widely recognised that poorly compacted and dry placed earthfills are susceptible to collapse compression on wetting leading to large deformations (Gould 1954; Bernell 1958; Penman 1986; Charles 1997; amongst others). The susceptibility of an earthfill to collapse compression is dependent on its material type, density and moisture content at placement, and the level of stress within the embankment. The case study evidence of earthfills suspected of collapse compression indicates:

- Dry placed and poorly compacted earthfills are susceptible to collapse compression on wetting (e.g. outer earthfill zone in old puddle embankments (pre 1900/1920)). There is no information from the database to suggest how dry poorly compacted earthfills need to be placed for them to be susceptible to collapse compression.
- Formally compacted earthfills susceptible to collapse compression include:
 - Earthfills placed on the dry side of optimum. Clayey earthfills placed drier than about 2% dry of Standard optimum are susceptible. But, this will vary depending on the material type, fines content, fines plasticity and compacted density ratio. Silty sands and gravels and clayey earthfills with low fines

- content or low plasticity fines may be susceptible when placed only 1% dry of Standard optimum. Charles (1998) suggests the air voids be reduced to less than 5% in clay earthfills to ensure the earthfill is not susceptible to collapse compression.
- Material types including silty sands and gravels, clayey sands and gravels and sandy clays generally of low plasticity. Medium to high and high plasticity clays do not appear to be susceptible to collapse compression.
- Layer thickness and the variation in density within the layer are also important considerations. The
 lower portion of thick placed layers, where the density is often lower, is more susceptible to collapse
 compression than the upper, more heavily compacted part of the layer.

7.0 SUMMARY AND METHODS FOR PREDICTION OF DEFORMATION OF EMBANKMENT DAMS

This section presents a summary of the outcomes from analysis of the deformation behaviour of earthfill, zoned earth and earth-rockfill, and puddle core earthfill embankments from Sections 4.0 and 5.0 for use in prediction of, or comparison of the deformation behaviour of an embankment. The methods for evaluation of the "normal" deformation behaviour were developed to identify potentially "abnormal" behaviour, and the figures and tabulated data provide a means for prediction or comparison to similar embankment types.

In part, the summary is a pointer to figures and tables in the report relating to specific aspects of embankment deformation behaviour. It also briefly summarises the factors affecting the deformation behaviour as determined from the analysis as well as bringing together some of the data in the report for ease of use.

7.1 EARTHFILL, AND ZONED EARTH AND EARTH-ROCKFILL DAMS

The methods for prediction of deformation behaviour of earthfill and zoned earth and earth-rockfill dams are divided into two subsections, deformation during construction and deformation post construction.

7.1.1 Prediction of Deformation Behaviour During Construction for Earthfill and Zoned Earth and Earth-Rockfill Embankments

Prediction of the deformations during construction of earthfill and zoned earth and earth-rockfill dams are best undertaken by finite element methods. The difficulty with these methods is in selection of properties and the constitutive model to represent the various material zones, as well as consideration of the use of coupled models and dealing with partial saturation of earthfills and the change in matric suction with stress level.

The total and effective stress conditions established in an embankment during construction are dependent on the embankment geometry, the embankment zoning geometry, and the strength and compressibility properties of the embankment materials. The simplest case to analyse is the elastic analysis of a homogeneous embankment on a rigid foundation where non-linearity of material properties is taken into account in the model used to represent the compressibility properties of the earthfill. Results show that under the embankment centreline lateral strains are negligible and deformation is only in the vertical direction. The homogeneous model is shown to be a reasonable assumption for the stress conditions under the embankment axis for:

- Zoned embankments with broad central earthfill zones for rolled earthfill placed dry of Standard optimum (more than about 0.5% to 1% dry), where only limited to negligible positive pore water pressures are developed during construction. Analysis and the case study data suggests it is a reasonable assumption for embankments with central core widths having a combined core slope (i.e. upstream and downstream core slope combined) greater than about 1.5 to 2H to 1V, but it may also be a reasonable simplification for combined central core widths down to 1H to 1V.
- Zoned embankments where the compressibility properties of the central earthfill zone and gravelly or rockfill shoulders are similar; i.e., compacted sandy and gravelly soils with non-plastic fines or low (less than about 20% finer than 75 micron) plastic fines contents, with shoulders of compacted gravels or rockfill. This is on the proviso that pore pressures generated in the core during construction are small and potential plastic type core deformations due to lateral spreading of the core are negligible.

Case study analysis shows that for dry placed earthfills the confined secant modulus shows a gradual increase with increasing effective vertical stress (ignoring matric suction). Therefore, reasonable numerical deformation

solutions can be obtained without consideration of pore water pressures provided the model used reasonably approximates the compressibility of the earthfill. Values of confined secant modulus of dry placed earthfills estimated from field data is presented in Table 4.5 and Figure 4.19 sorted based on material type. For dominantly sandy and gravelly earthfills the data is further sorted based on fines plasticity and fines content. The data provides useful bounds for comparison with laboratory test data on proposed core materials and assistance in selection of the confined moduli for use in analysis.

At the other end of the spectrum is wet placed earthfill cores of low undrained shear strength used in zoned earth and earth-rockfill embankments. Numerical analysis modelling the core in undrained conditions is considered to provide a reasonable approximation of the deformation behaviour for earthfills of low permeability, where the deformation of the core occurs largely as undrained plastic type deformations. The amount of lateral spreading of the core, which is influenced by the lateral stresses developed in the core and the compressibility of the supporting shoulder zones, has a significant influence the deformation of the core.

Modelling becomes more complex where:

- Pore water pressure dissipation in the core occurs during construction and the use of a coupled model is desirable.
- Over the period of construction the initial deformation of the core is largely due to compression of air voids, but in the latter stages largely occurs as plastic type deformations in undrained conditions.

For rockfill and gravel earthfill zones, Hunter and Fell (2002a) provide information for use in selection of the confined modulus properties based on intact rock strength, placement method and particle size distribution.

Several methods have been developed for evaluation of the deformation behaviour during construction that are useful for comparative purposes. They are:

- Total core settlement for the period of construction. Estimation of the total core settlement is based on embankment height, core material type and core width (Figure 4.23, Table 4.6 and Table 4.7). This method was effectively used to identify "abnormally" large deformations during construction for several embankments.
- Vertical strain profile in the core at end of construction; Figure 4.18 for dry placed earthfill cores, Figure 4.21 for wet placed dominantly sandy and gravelly cores (SC/GC/SM/GM cores), and Figure 4.22 for wet placed clay cores. The figures provide approximate bounds for "normal" type deformation behaviour. They can also be used for evaluation of core deformation as construction proceeds. The figures are useful for identification of regions of the core in embankments where vertical strains were excessively large. Plots of vertical strain versus effective vertical stress or height above the cross-arm interval (e.g. Figure 4.15 or Figure 4.20) for regions of high strain can then be used to assess the incremental vertical strain and evaluate the possibility of shear type deformation.
- Lateral deformation of the core for central core earth and rockfill dams of thin to medium core width (Section 4.1.2). The estimated lateral displacement ratio (LDR) of the core at selected depths in a limited number of case studies is presented in Figure 4.12 and Figure 4.13. The data indicates:
 - LDR is influenced by the lateral stress developed in the core and so will be greater for wet placed than
 dry placed cores, and earthfills of low permeability (i.e. limited dissipation of pore water pressures
 during construction).
 - The compressibility of the supporting shoulder zones has a significant influence on LDR. LDR increases with increasing compressibility of the shoulders.
 - The location of measurement affects LDR. Finite element analysis of a dam on a rigid foundation showed LDR at end of construction to be a maximum at about 50 to 70% of the depth below crest level.

7.1.2 Prediction of Deformation Behaviour Post Construction for Earthfill and Zoned Earth and Earth-Rockfill Embankments

A number of figures and tables have been developed for evaluation and/or prediction of the deformation behaviour post construction of SMPs on the crest and slopes of the embankment. In summary they include:

- Settlement (based on zero time at the end of embankment construction):
 - Magnitude of settlement for the crest and shoulders at 3, 10 and 20-25 years after construction
 - Settlement versus time plots for the crest and shoulders
 - Settlement rate (rate in terms of log time)
- Lateral displacement:
 - Of the crest, upstream shoulder and downstream shoulder on first filling (Section 4.2.3)
 - Of the crest post first filling
 - Lateral displacement versus time plots for the crest and shoulders (based on zero time at the end of embankment construction)

(a) Deformation versus time plots

A large number of figures have been produced of deformation (i.e. settlement or displacement) versus time, with zero time established at the end of embankment construction. Table 7.1 and Table 7.2 are a pointer to the plots of settlement and displacement versus time for the crest and shoulders respectively. Note that for zoned earth and earth-rockfill dams with thin to thick core widths the upstream edge of the crest is included in the upstream shoulder region, and the crest region is from the central to downstream edge of the crest (refer Figure 4.37).

The case studies for crest deformations were sorted based on core width, core material type and moisture content at placement. For embankments with very broad earthfill cores the influence of the foundation was taken into consideration. For the deformation versus time for the shoulders, the case studies have been sorted based on the material type in the shoulder, and for rockfills its compaction rating.

Core Width *1	Core Classification *2	Moisture Content *3	Figure Reference		
	Core Classification .	Moisture Content.	Settlement	Displacement	
Thin to Medium	CI /CII	Dry placed	Figure 4.53	Figure 4.68	
	CL/CH	Wet placed	Figure 4.54	Figure 4.69	
	90/00	Dry placed	Figure 4.55	Figure 4.70	
	SC/GC	Wet placed	Figure 4.56	Figure 4.71	
	SM/GM	Dry and wet placed	Figure 4.57	Figure 4.72	
Thick	CL/CH	Mostly dry placed	Figure 4.58	Figure 4.73	
	SC/GC/SM/GM	Mostly dry placed	Figure 4.59	Figure 4.74	
Very Broad	Limited/negligible founda	Limited/negligible foundation influence		Figure 4.75	
	Potentially significant four	ndation influence	Figure 4.61	Figure 4.76	

Table 7.1: Figure references for post construction crest deformation

Note:

*1 The terms used for core width classification are defined in Section 1.2.1

*2 The core classification is to Australian Standard AS 1726 – 1993: Geotechnical Site Investigation. It differs slightly to the Unified Soil Classification System in the particle size limits between sand and gravel size.

^{*3} Dry placed defines cores where limited to negligible pore water pressures were developed during construction and is generally applicable to placement more than 0.5% to 1% dry of Standard optimum. Wet placed refers to cores where significant pore water pressures were developed during construction. Refer to Section 4.1.1.4 for further discussion and definition of the qualitative terms used.

Shoulder Material	Compaction Rating *1	Downstream	n Shoulder *2	Upstream Shoulder *2	
Type		Settlement	Displacement	Settlement	Displacement
Rockfill	Well compacted	Figure A2.4	Figure A2.12	Figure A2.23	Figure A2.30
	Reasonably to well compacted	Figure A2.5	Figure A2.13	Figure A2.24	Figure A2.31
	Reasonably compacted	Figure A2.6	Figure A2.14	Figure A2.25	Figure A2.32
	Poorly compacted	Figure A2.7	Figure A2.15	Figure A2.26	Figure A2.33
Gravels	-	Figure A2.8	Figure A2.16	Figure A2.27 Figure A2.3	
Earthfills	-	Figure A2.9	Figure A2.17	Figure A2.27	riguic A2.54
Very broad core width	No foundation influence	Figure A2.10	Figure A2.18	Figure A2.28	Figure A2.35
,	Foundation influence	Figure A2.11	Figure A2.19	Figure A2.29	Figure A2.36

Table 7.2: Figure references for post construction deformation of the embankment shoulders

Note: *1 Refer to Section 1.2.2 for definitions of the compaction rating terms used.

The general trend of the deformation versus log time plots is:

- For settlement, near linear to slightly increasing rate versus log time is the general trend. But, it is not unusual to have a slightly decreasing settlement rate long-term for the crest. In some cases magnitudes of settlement are large on first filling for the crest and upstream slope, particularly where the upstream shoulder is susceptible to collapse settlement.
- For horizontal displacement a general trend is more difficult to define because of the broader range in behaviour, but:
 - A large percentage of the displacement generally occurs on first filling. For the crest and downstream slope, this displacement is generally in a downstream direction.
 - Long-term, the trend of the displacement rate (rate per log time) approaches low to near zero values, with fluctuations about the trend due to reservoir fluctuation.

(b) Magnitude of post construction settlement

The magnitude of post construction settlements for the crest and shoulder regions are presented at 3, 10 and 20 to 25 years after the end of embankment construction. Table 7.3 is a pointer to tables and figures within the report, and Table 7.4 and Table 7.5 summarise the typical range of "normal" settlement magnitude for the crest and shoulders respectively.

For the crest region, the data has been sorted based on core width, core material type and placement moisture content, and indicates that:

- The post construction crest settlements are generally much smaller than the core settlement during construction.
- Nearly all dams experience less than 1% crest settlement post construction for periods up to 20 to 25 years and longer after construction.
- Most experience less than 0.5% in the first 3 years and less than 0.75% after 20 to 25 years.
- Smaller magnitude settlements are observed for dry placed clayey sands to clayey gravels and dry to wet placed silty sands to silty gravels.
- A broader range of settlement magnitude is shown for clay cores, wet placed clayey sand to clayey gravel cores, and embankments with very broad core widths.
- For zoned earth and rockfill dams, poor compaction of the rockfill is over-represented for case studies at the larger end of the range of crest settlement.

^{*2} Figures are in Appendix A.

The data for the shoulder regions indicates:

- Settlements in the order of 1 to 2% are observed for poorly compacted rockfills. Greater settlements are observed for the dry placed, poorly compacted rockfills.
- For reasonably compacted rockfills the range of settlement is quite broad, from 0.1% up to 1.0%, but the number of cases is limited. Settlements toward the upper range are observed for dry placed and/or weathered rockfills, where settlements due to collapse compression are likely to be significant.
- Much lower settlements, generally less than 0.5 to 0.7% at ten years after construction, are observed for well and reasonably to well compacted rockfills, and compacted earthfills.
- Very low settlements (less than 0.25% at 10 years) are observed for embankments with gravel shoulders.

Table 7.3: References to post construction settlement magnitude tables and plots

Embankment Region	Table Reference	Time After End of Construction (years) *1				
Embankment Region	Table Reference	3 years	10 years	20 to 25 years		
Crest	Table 4.11	Figure 4.46	Figure 4.46 Figure 4.47			
Downstream shoulder	Table 4.12	Figure 4.49	Figure 4.50	Figure A2.3		
		(Figure A2.1)	(Figure A2.2)			
Upstream shoulder	Table 4.12	Figure 4.51 (Figure A2.20)	Figure 4.52 (Figure A2.21)	Figure A2.22		

Note: *1 Figures numbers with an 'A' prefix are in Appendix A.

Table 7.4: Embankment crest region, typical range of post construction settlement and long-term settlement rate

Core Properties		Crest Settlement (%) *1, *2			Long-term Settlement Rate *1, *3		
Classification	Core Width	Moisture content	3 years	10 years	20 to 25 years	Steady/Slow Reservoir	Fluctuating Reservoir
	Thin to	dry	0.05 to 0.55	0.10 to 0.65	0.20 to 0.95	0.04+.0.50	
CL/CH	medium	wet	0.04 to 0.75	0.08 to 0.95	0.20 to 1.10	0.04 to 0.50 (most < 0.26)	0.09 to 0.57
	Thick	all (most dry)	0.02 to 0.75	0.10 to 1.0	0.5 to 1.0	(most \ 0.20)	
	Thin to medium	dry	0.10 to 0.25	0.10 to 0.40	< 0.5		
SC/GC		wet	0.15 to 0.80	0.20 to 1.10	< 1.1	0 to 0.26	0.06 to 0.37
	Thick	all (most dry)	0.05 to 0.20	0.10 to 0.35	0.10 to 0.45		
SM/GM	Thin to thick	all	0.06 to 0.30	0.10 to 0.65	< 0.5 to 0.7	< 0.10	0.03 to 0.21
Very Broad Earthfill Cores - most CL and dry placed		0.0 to 0.60	0.0 to 0.80	0.05 to 0.76	0.08 & 0.44	0.07 to 0.70 (most < 0.35)	

Note

^{*1} excludes possible outliers.

^{*2} crest settlement as a percentage of the embankment height

^{*3} long-term settlement rate in units of % settlement per log cycle of time (settlement as a percentage of dam height).

Table 7.5: Embankment shoulder regions, typical range of post construction settlement and long-term settlement
rate

Material Type	Composition	Dow	nstream Show	ulder *1	Upstream Shoulder *1		
	Compaction Rating	Settlement (%) * ²		Settlement	Settlement *2		Settlement
		3 years	10 years	Rate * ³	3 years	10 years	Rate * ³
	well	0.05 to 0.35	0.05 to 0.55	0.0 to 0.33	0.10 to 0.60	0.10 to 0.70	0.05 to 0.70
Rockfill							(most < 0.50)
	reasonably to	< 0.30	< 0.50	0.04 to 0.31	0 to 0.55	0.10 to 0.60	0.10 to 0.56
	well			(most > 0.15)			(most < 0.50)
	reasonable	0.20 to 1.0	0.10 to 1.0	0.10 to 1.0	< 0.70	- * ⁵	< 0.55
	poor	0.10 to ? * ⁵	0.15 to ? * ⁵	0.10 to 0.25	0.10 to 1.05	0.15 to 1.20	0.10 to 0.82
	poor – dry *4	0.15 to 1.60	0.30 to 2.00	0.20 to 0.75	0.15 to 1.35	0.20 to 1.6	(most < 0.60)
Gravels	-	< 0.15	< 0.25	0.02 to 0.065	< 0.15	< 0.25	< 0.21
Earthfills	-	0.0 to 0.40	0.0 to 0.70	0.0 to 0.40	0.05 to 0.60	0.10 to 0.70	0.10 to 0.60

Note:

- *2 Settlements quoted are a percentage of the height from the SMP to foundation level.
- *3 The long-term settlement rates are in units of % settlement per log cycle of time (settlement as a percentage of the height from the SMP to foundation level).
- *4 For the dry placed and poorly compacted rockfills, a large range in settlements is observed. For rockfills placed in dry climatic regions settlements are likely to be toward the upper end of the range.

(c) Long-term settlement rate

As previously discussed, post first filling the settlement rate (rate in terms of log time) of SMPs on the crest and slopes is generally close to linear. For a number of case studies the rate may increase slightly with time and for some case studies it may decrease slightly with time. The long-term settlement rate for the case studies was estimated assuming a linear relationship between settlement and log time, and over periods of time representing "normal" reservoir operating conditions. For case studies where the rate increased (or decreased) with time after first filling the estimate was generally based on the later period of measurement. Table 7.6 is a pointer to tables and figures related to long-term settlement rate within the report, and Table 7.4 and Table 7.5 provide the typical range of long-term settlement rate for the crest and shoulders respectively excluding possible outliers. The units of settlement rate are the same as used for the puddle core earthfill embankments.

Table 7.6: References to tables and figures of long-term settlement rate

Embankment Region	Table Reference	Figure Reference	Comment
Crest	Table 4.13	Figure 4.63	Figure 4.62 is for zoned embankments with thin to thick core widths, and Figure 4.63 is for embankments with very broad core widths.
Downstream shoulder	Table 4.14	Figure 4.78	
Upstream shoulder	Table 4.15	Figure 4.83	

Reservoir fluctuation was found to influence the long-term settlement rate for the crest, particularly for the zoned embankments of thin to thick core width with permeable upstream shoulder fill (i.e. rockfill or gravels). Generally, greater long-term settlement rates were measured for case studies with fluctuating reservoir levels. The influence of reservoir fluctuation on the upstream shoulder could not be clearly identified from the available data and it had limited to negligible influence on the downstream shoulder. Two qualitative categories were used for reservoir operation, "fluctuating" for reservoirs subject to a seasonal (usually annual) or regular (more than once per year) drawdown typically greater than about 10 to 15% of the maximum height of the embankment, and "steady" or "slow".

^{*} Excludes possible outliers.

^{*5} insufficient data.

Apart from the influence of reservoir operation, the data for the crest indicated:

- For zoned embankments with thin to thick core widths:
 - Core material type has an influence on the long-term crest settlement rate. In general, earthfill cores of silty sands to silty gravels tend to have low long-term settlement rates and clay earthfill cores, on average, tend to have high long-term crest settlement rates. Clayey sand to clayey gravel cores show intermediate rates.
 - The core width and compaction moisture content appear to have little to no recognisable influence on the long-term crest settlement rate. Their influence is probably over-shadowed by other factors.
 - Embankments with rockfills susceptible to large deformations due to collapse compression have high long-term settlement rates, generally greater than 0.4% per log cycle of time, indicating these embankments are more susceptible to softening and long-term degradation.
- For embankments with very broad width earthfill zones the long-term settlement rate shows a broad variation in range. Reservoir fluctuation does influence the long-term settlement rate for those embankments with more permeable earthfills.

The data for the embankment shoulders indicates:

- The long-term settlement rate of both the up and downstream shoulder is generally less than about 0.4% per log cycle of time for most cases.
- Higher rates are observed for zoned earth and rockfill dams with rockfills susceptible to large deformations due to collapse compression, particularly in the downstream shoulder.
- Very low rates for zoned embankments with gravel shoulders.

(d) Post construction horizontal displacement

As previously indicated, a large portion of the horizontal displacement generally occurs on first filling, particularly for zoned earthfill and earth-rockfill dams with permeable fills in the upstream shoulder. Additional data (other than the displacement versus time plots) is presented for horizontal displacement, including:

- Lateral displacement on first filling:
 - Table 4.9 and Figure 4.38 relating to the crest displacement.
 - Figure 4.39 for the downstream shoulder region
 - Figure 4.40 for the upstream shoulder region
- Lateral displacement of the crest post first filling (Figure 4.77).

The case study records on crest displacement during first filling show that the typical range for most case studies is from 50 mm upstream to 300 mm downstream. A number of trends were evident sorting the data based on core width, core material type and the material type and compaction rating of the downstream shoulder. They were:

- For most groups, the displacement on first filling was downstream and less than 0.1 to 0.2% of the embankment height (less than 100 to 200 mm).
- Greater displacements (from 0.2% to 0.6% and up to almost 1% of the embankment height) were observed for central core earth and rockfill dams of thin to medium core width with:
 - Rockfills susceptible to large deformations due to collapse compression in the downstream shoulder.
 - Silty sand to silty gravel earthfill cores.

These findings suggest that large deformations in the downstream shoulder, possible due to collapse compression on wetting from rainfall, significantly influence the crest displacement for embankments of thin to medium core width. This was confirmed by the observation that those embankments with large crest displacements on first filling also experienced large downstream displacements of the downstream shoulder.

The correlation to silty sand and silty gravel cores is possibly indicative of greater magnitude increases of lateral stress in the downstream shoulder on first filling for these embankment types.

The displacement of the downstream shoulder on first filling had many similarities to the crest behaviour on first filling:

- Displacements of the downstream shoulder were less than 0.1 to 0.2% of the embankment height for embankments with very broad earthfill cores, zoned embankments with compacted gravels or earthfills in the downstream shoulder, and zoned embankments with wetted (and generally compacted) rockfills in the downstream shoulder.
- Greater magnitude displacements were generally observed for zoned earth and rockfill dams with dry placed rockfills in the downstream shoulder:
 - Up to 0.25 to 0.30% of the dam height for dry placed and well and reasonably to well compacted rockfills
 - More than 0.2 to 0.25% and up to 0.85% of the dam height for dry placed and poor and reasonably compacted rockfills.

For the upstream shoulder region, displacements on first filling ranged from 200 to 300 mm upstream to 300 mm downstream.

Crest displacements for the period from post first filling to 10's of years after construction are generally of smaller magnitude than the displacement on first filling, particularly for zoned earth and earth-rockfill embankments with thin to medium core widths. Regardless of the embankment type though, the general range of displacement post first filling (for at least 5 years post first filling and up to 40 to 50 years) is quite small, ranging from 35 mm upstream to 100 to 150 mm downstream.

The displacement of the downstream slope post first filling is more erratic than for the crest, even though for a large number of the case studies the displacement post first filling displacement is of small magnitude. In terms of the total magnitude of displacement in a downstream direction since end of construction:

- For embankments with very broad core widths, total displacements generally range from 0.05 to 0.30% of the embankment height at 25 to 45 years after end of construction.
- For zoned earthfill embankments, total displacements are up to 0.15 to 0.20% of the embankment height at 20 to 30 years after end of construction.
- For central core earth and rockfill dams:
 - For well and reasonably to well compacted rockfills, displacements of up to 0.20% are measured at 10 to 20 years after end of construction. Where the rockfill has been dry placed or is of relatively high compressibility displacements can be greater, up to 0.25 to 0.40 % of the embankment height.
 - For reasonably and poor compacted rockfills, displacements up to 1.0 to 1.6% of the embankment height can occur long-term, particularly for dry placed rockfills susceptible to large deformations due to collapse compression.

7.2 PUDDLE CORE EARTHFILL DAMS

The predictive methods for puddle core earthfill dams are appropriate to the long-term deformation behaviour of the embankment, many tens of years after construction. The following presents a summary of the analysis and discussion from Section 5.3, of which Section 5.3.6 provides a useful summary of the factors affecting the longterm deformation behaviour of puddle core earthfill dams.

The records show that the post construction crest settlement of puddle core earthfill embankments is significant, ranging from 1% up to 8% to 14% of the dam height after more than 100 years. The older (pre 1900) embankments generally show the greater magnitude post construction crest settlement and the more recent embankments (constructed in the 1930's to 1950's) generally show crest settlements of lesser magnitude. A large proportion of the settlement in the older embankments occurred during and shortly after the period of first filling due mainly to collapse compression on wetting of the poorly compacted earthfill supporting the puddle core and yielding on drawdown.

The long-term settlement rate (S_{LT}) of the embankment crest (under normal reservoir operating conditions) was significantly affected by the magnitude of reservoir fluctuation and the permeability of the earthfill zone upstream of the puddle core. Table 7.7 provides guidelines for estimation of the long-term crest settlement under normal reservoir operating conditions. The influence of reservoir fluctuation and earthfill permeability over-shadowed other factors that probably influence the long-term crest settlement including dam height, age and earthfill material type. For several dams the long-term crest settlement rate was intermediate to the ranges given. For Hope Valley dam, part of the reason was thought to be the variable pore water pressure response (ranging from 25 to 100%) in piezometers in the earthfill zone upstream of the puddle core to fluctuations in reservoir level.

Table 7.7: Predictive methods of long-term crest settlement and displacement under normal reservoir operating conditions for puddle core earthfill embankments

Reservoir	Response to Drawdown of the	No.	Crest Settlement Prediction	Crest
Operation *1	Earthfill Zone Upstream of the	Cases		Displacement *3
	Puddle Core *2			
Steady	("steady state")			
	negligible to minor	7	$S_{LT} = 0.4 \text{ to } 1.0 \% *^4$	negligible
	(or "steady state")			
Fluctuating	near full response (i.e. permeable earthfill)	4	S_{LT} = 4.5 to 7.4 % Figure 5.11 for estimation of permanent settlement based on the magnitude of the drawdown	3 to 5 mm/year (downstream)

- Notes: *1 Fluctuating defined as reservoir subject to annual drawdowns generally greater than 10 to 20% of the embankment height
 - *2 Refer Figure 5.6 for definitions of "steady state" conditions.
 - *3 crest displacement records only available for very few case studies.
 - $*^4$ S_{LT} is the long-term crest settlement rate in units of settlement as percentage of embankment height per log cycle of time. Individual values for the case studies are given in Table 5.1.

The methods based on historical performance of similar dams are very approximate because of the limited number of case studies from which they have been derived and the assumptions made regarding the reservoir operation and permeability of earthfill zones upstream of the core. The methods should only be as a general guide in consideration of these factors.

For several embankments, large permanent crest settlements (0.20 to 0.52% of the embankment height) were measured during abnormally or historically large drawdown events (Tedd et al 1997b). In terms of prediction of deformation during these abnormal events, the only available method would seem to be fully coupled finite element modelling (FEM) due to the complexity of the processes and interaction between the different elements within the embankment. But, this is not without difficulties due to the lack of known information on material properties as well as the need to consider stress history.

Available records on the long-term deformation of the shoulders and displacement of the crest are limited. Some preliminary guidance on possible magnitudes of long-term deformation under normal reservoir operating conditions are:

- For displacement of the crest and downstream slope:
 - Under "steady state" conditions long-term rates of displacement are negligible.
 - For fluctuating reservoir conditions and permeable upstream earthfill zones, long-term rates of displacement of up to 3 to 5 mm/years in a downstream direction have been recorded.
- The long-term settlement rate for the shoulders is:
 - Of similar magnitude to the crest for "steady state" conditions (refer Table 7.7).
 - Of lesser magnitude than the crest for fluctuating reservoir conditions and permeable upstream earthfill zones.

8.0 CONCLUSIONS

The main objective of the report was to develop methods for identification of potentially "abnormal" deformation behaviour of embankment dams from field monitoring records. The methods were predominantly developed from initially defining what is "normal" deformation behaviour for a particular embankment type in consideration of the imposed stress conditions, and the strength properties and stress-strain relationship of the materials. Potentially "abnormal" deformation behaviour was then broadly identified where the rate, magnitude or trend of the deformation behaviour differed from that of the "norm". Implicit in the methods are concepts such as consolidation, creep under stress conditions and collapse compression on wetting. Once the deformation behaviour for a case study was identified as potentially "abnormal", further analysis was undertaken (where records were available) for evaluation of the possible mechanism/s causing the observed deformation behaviour. Section 6.0 of the report deals with the identification and evaluation of "abnormal" deformation behaviour from the case study data, Section 6.6 providing a summary.

The database comprises some 134 embankments and included the following embankment types:

- Zoned earth and rockfill embankments, most of which were central core earth and rockfill embankments.
- Zoned earthfill embankments
- Rolled earthfill embankments, including homogeneous earthfill, earthfill with filters and earthfill with rock toe.
- Puddle core earthfill embankments.

The deformation monitoring records analysed included:

- During construction mainly internal vertical deformation of the core, but also the lateral core deformation of zoned earth and rockfill dams with thin to medium core widths.
- Post construction SMPs on the crest and slopes of the embankment, and the internal vertical deformation of the earthfill core.

From such a broad database of embankment dams it has been possible to develop methods for prediction and evaluation of the deformation behaviour during and post construction from the main types of deformation monitoring records analysed. Section 7.0 provides a summary of these methods with reference to pertinent tables and figures within the report.

The methods are considered to be an improvement on currently available methods for the embankment types considered. The influence of material type and placement methods, embankment zoning geometry, embankment height, reservoir operation amongst other factors have been considered.

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