Rockfill Modulus and Settlement of Concrete Face Rockfill Dams

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Abstract: A method is presented for estimating the modulus of compacted rockfill in dams based on the particle size, unconfined compressive strength of the rock, compaction layer thickness, compactive effort, and the applied vertical stress. Also presented are methods for predicting the crest settlement and face slab deformation of concrete face rockfill dams during first filling and in the long term. It is demonstrated that the modulus is stress dependent and guidance is provided on how to assess this, as well as effects of construction in narrow valleys where arching may affect the stresses in the dam. These methods are based on analysis of the 35 dams with good quality information on construction materials and placement methods, and good quality internal and surface settlement monitoring records.

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CE Database subject headings: Deformation; Dams, rockfill; Settlement.

Introduction

Rockfill has been used in the construction of dams for over 150 years. Up until the 1960's the rockfill was usually dumped in high lifts (as high as 20 to 50 m), with sluicing under high pressure adopted from the 1930's. From the late 1960's rockfill has been placed in 1 to 2 m layers and compacted using steel drum vibratory rollers, usually with water added before rolling (Galloway 1939; Cooke 1984, 1993).

The lower compressibility from compaction of rockfill in thin layers resulted in the resurgence in use of the concrete face rockfill dam (CFRD) since the late 1960's due to the significant reduction in leakage rate and post-construction deformation. Now there are now many CFRD higher than 150 m and up to 240 m, such as the 185 m high Aguamilpa dam in Columbia completed in 1993 and the 180 m high Tianshengqiao dam in China completed in 1999. Compacted rockfill has also been used in central core earth and rockfill dams since the 1960's and embankment heights have since reached up to 200 to 300 m, such as the 260 m high Chicoasen dam in Mexico completed in 1980, and the 240 m high Guavio dam in Columbia completed in 1989.

The large particles (up to the layer thickness in diameter) within the rockfill and the intralayer variation in grading and density make laboratory testing to determine the compressibility properties of rockfill difficult. It is often necessary to rely on the historic performance of other dams to estimate the properties. There is some published information to enable this and to estimate the deformation of CFRD (e.g., Marsal 1973; Clements 1984; Sherard and Cooke 1987; Pinto and Marques Filho 1998; Giudici et al. 2000) but these are based on limited data, and often concentrate on only one or two factors which affect either the rockfill modulus or the measured deformation.

Good quality monitoring records and construction materials data has been gathered for 35 CFRD from Australia, South America, South East Asia, Mexico, China, and the United States. This is for mainly modern dams with compacted rockfill, but some data on dumped rockfill has been gathered, including that for the earth and rockfill El Infiernillo dam. This data has been analyzed to develop methods for predicting the rockfill modulus from the rockfill particle size distribution, compaction method, and unconfined compressive strength (UCS) of the rock; the deformation of the face slab of CFRD on first filling; and predicting long term crest settlements of CFRD. While most of the data is from CFRD it will have application to rockfills in other dams types and in civil or mining construction works.

Definitions and Terms Adopted

The following definitions and terms have been used:

Concrete Face Rockfill Dam Zoning

The typical zoning of a CFRD is shown in Fig. 1. Zones 3A and 3B are the main rockfill zones, and are distinguished by the maximum layer thickness (usually 0.9 to 1.2 m for Zone 3A, 1.5 to 2.0 m for Zone 3B) and maximum particle size allowed (usually up to the layer thickness in diameter). Zones 2D and 2E support the concrete face slab, and are, in modern dams, graded to limit leakage flows in the event of a joint opening or a crack forming in the face slab (Cooke 2000). Zones 1A and 1B are earthfill zones to limit leakage around the plinth, or in the event of a leak in the face slab. This study deals with rockfill mostly in Zone 3A and also in Zone 3B.

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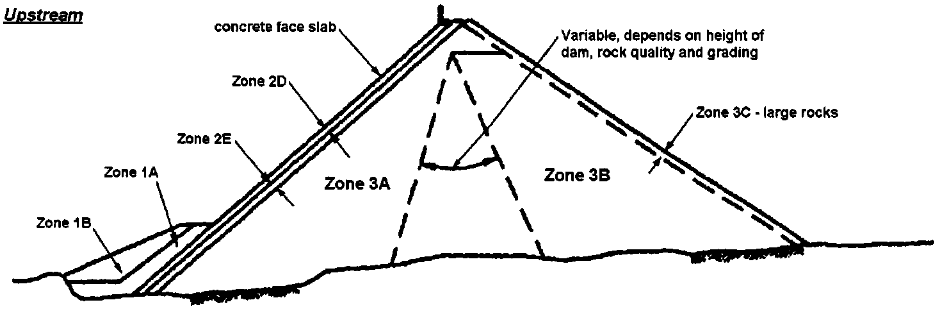

Fig. 1. Typical zoning of concrete face rockfill dams (Cooke 1997)

Rockfill Placement and Compaction

The definitions by Cooke (1984, 1993) for dumped and compacted rockfill have been used as a basis for categorization of the method of placement:

- Dumped rockfill–Rock placed in lifts ranging from several to tens of meters thickness, with or without sluicing, and without formal compaction,
- Compacted rockfill–Rock 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 four to six passes of a minimum 10 t (or possibly up to 15 t) 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 intact strength of the rock), usually placed with the addition of water and compacted with a minimum four passes of a 10 t deadweight smooth drum vibrating roller (SDVR). The exception to water addition is for rockfills sourced from rock types of very high strength where the substance strength is not greatly reduced by wetting,
- Reasonable compaction–Layer thickness of 1.5 to 2 m (depending on the intact strength of the rock), usually placed without the addition of water and compacted generally with four passes of a 10 t SDVR, and
- Reasonable to well compacted–Layer thickness typically of 1.2 to 1.6 m (depending on the intact strength of the rock), placed with the addition of water and compacted with four to six passes of a 10 t SDVR.

Rockfill Moduli

Fitzpatrick et al. (1985) defined two moduli for assessment of the deformation behavior of rockfill (Fig. 2), the rockfill modulus during construction E_{rc}, and the rockfill modulus on first filling E_{rf}, calculated from Eqs. (1) and (2).

$$E_{rc}=\gamma H d_1/\delta_s \tag{1}$$

$$E_{rf}=\gamma_w h d_2/\delta_n \tag{2}$$

where E_{rc} and E_{rf} are in MPa; γ = unit weight of the rockfill in kN/m^3; γ_w = unit weight of water in kN/m^3; δ_s = settlement of layer of thickness d_1 due to the construction of the dam to a thickness H above that layer; δ_n = face slab deflection at depth h from the reservoir surface; and d_2 is measured normal to the face slab as shown. H, h, d_1, and d_2 are all measured in meters, and δ_s and δ_n are measured in millimeters.

Eq. (1) does not allow for the effect of embankment shape on the distribution of vertical stress in the embankment. This study allows for the effect of embankment shape by using the stress intensity factors for elastic solutions by Poulos and Davis (1974). E_{rf} is not a true modulus of the rockfill rather it is an artifact of the method of calculation, and should only be used as discussed below to estimate the face slab deformation.

Unconfined Compressive Strength (UCS) of Rock

The rock used as rockfill has been classified as medium to high strength (UCS 6 to 70 MPa) and very high strength (UCS 70 to 240 MPa). The medium to high strength grouping is recognized as being very broad for compacted rockfill. But the database consists of very few cases of medium strength rockfill, and the data indicates they can be grouped with the high strength rockfills without introducing significant uncertainty into the statistical correlations.

Database of Dams Data and Parameters Considered

Data was collected for 35 CFRD and one earth and rockfill dam. Of the CFRD, seven were constructed of dumped rockfill, and

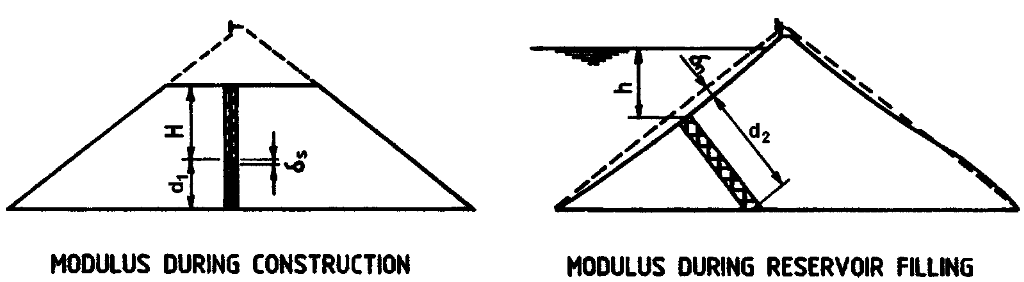

Fig. 2. Simplified methods for determination of rockfill moduli (Fitzpatrick et al. 1985).

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Table 1. Dams Used in Analysis

Placement method[a]	Dam names and country of location[b]
Dumped rockfill (7 dams)	Cogswell, Courtright, Dix River, Lower Bear No.1 and 2, Salt Springs, Wishon (all dams in the United States)
Compacted rockfill (26 dams)	Aguamilpa (Mexico), Alto Anchicaya (Columbia), Bastyan (Aust.), Cethana (Aust.), Chengbing (China), Foz do Areia (Brazil), Ita (Brazil), Kangaroo Creek (Aust.), Khao Laem (Thailand), Kotmale (Sri Lanka), Little Para (Aust.), Mackintosh (Aust.), Mangrove Creek (Aust.), Murchison (Aust.), Reece (Aust.), Salvajina (Columbia), Scotts Peak (Aust.), Segredo (Brazil), Serpentine (Aust.), Shiroro (Nigeria), Tianshengqiao 1 (China), Tullabardine (Aust.), White Spur (Aust.), Winneke (Aust.), Xibeikou (China), Xingo (Brazil)
Compacted gravel (4 dams)	Aguamilpa (Mexico), Crotty (Aust.), Golillas (Columbia), Salvajina (Columbia)
Earth and rockfill dam-dry placed and poorly compacted rockfill (1 dam)	El-Infiernillo (Mexico)

[a]Concrete face rockfill dam unless stated.

[b]Aust. = Australia.

four were constructed using gravels as Zone 3A. The height of most dams is in the range 75 to 150 m, with an overall range of 25 to 185 m. Table 1 lists the dams.

Table 2 summarizes the geometrical factors of the embankment and the material properties and placement methods of the rockfill that were considered in the analysis. Also considered were the valley shape and timing of first filling from the end of main rockfill construction.

The monitored deformation data gathered for each case study, where available, was from internal vertical deformations during embankment construction, face slab deformation during first filling, and crest settlements after the end of main rockfill construction. Details are presented in Hunter and Fell (2002) including graphical plots of vertical stress versus strain, calculated secant and tangent modulus versus vertical stress for the rockfill during construction, and crest settlement versus time post construction.

Effect of Valley Shape

The influence of valley shape is identified (Pinto and Marques Filho 1998; Giudici et al. 2000) as a significant factor affecting the vertical stresses within the dam because of arching across the abutment slopes. They indicate it is particularly important to consider the valley shape for embankments constructed in narrow valleys with relatively steep abutment slopes.

Table 2. Properties of Embankment and Rockfill Considered in Database

Group	Description
Embankment dimensions	Height (H), crest length (L), ratio L/H, upstream slope
Rockfill material properties and placement methods	Rock (or gravel) geology, UCS, particle shape, uniformity coefficient ($C_u = D_{60}/D_{10}$), particle size distribution ($D_{\max}, D_{90}, D_{80}, D_{60}, D_{50}, D_{30}, D_{10}$, % finer 19 mm);
	Rockfill dry density, void ratio, layer thickness; Roller weight, number of passes, water added

To check these hypotheses a two-dimensional finite difference analysis was undertaken assuming an idealized rockfill embankment of 100 m height constructed in valleys with river widths of 20, 50, and 100 m, and abutment slopes (constant slope) of 0, 26.5°, 45°, and 70°. The rockfill was modeled as a linear elastic material with Young's modulus of 100 MPa and Poisson's ratio (ν) of 0.27, and the foundation with Young's modulus of 50 GPa. After establishing initial stresses in the foundation, the embankment construction was modeled in 5 m lifts and the stresses monitored (major and minor principal stresses, vertical stress, and horizontal stress). The analysis was then repeated by construction of the embankment in a single 100 m lift as was done in the analysis by Giudici et al. (2000). While recognizing the limitations of the analyses, including the use of two-dimensional (2D) rather than 3D analysis, and the use of elastic parameters, it is felt that the general trends of the stresses and influence of cross valley shape were reasonably modeled. The results showed that

1. Cross-valley arching is significant (greater than 20% reduction in vertical stress) for narrow valleys (river width less than 30 to 40% of the dam height) with steep abutment slopes (greater than about 50°, but also dependent on river width), and then only in the lower third to half of the embankment. Where the river width is approximately equal to half the embankment height, cross-valley arching has some effect (10 to 20% reduction in vertical stress) for abutment slopes steeper than about 45°, and then only in the lower third to half of the embankment. A negligible influence of cross valley arching is observed for river widths equal to about the embankment height regardless of the abutment slope;
2. Modeling embankments in single or very large lifts is likely to result in significant under-estimation of stresses within the embankment, particularly for river widths less than about half the embankment height; and
3. The method used to calculate the stresses in the dam from which moduli are estimated is important. Pinto and Marques Filho (1998) and Giudici et al. (2000) appear to have used the simplifying assumption of Fig. 2, which over-estimates the stresses, and therefore under-estimates the moduli in the

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Table 3. Approximate Stress Reduction Factors to Account for Valley Shape

W_r/H ratio (river width to height)	Average abutment slope angle (degrees)	Stress reduction factor (embankment location)			
		Base (0 to 20%)	Mid to low (20 to 40%)	Mid (40 to 65%)	Upper (65% to crest)
0.2	10 to 20	0.93	0.95	0.97	1.0
	20 to 30	0.88	0.92	0.96	0.98
	30 to 40	0.82	0.88	0.94	0.97
	40 to 50	0.74	0.83	0.91	0.96
	50 to 60	0.66	0.76	0.86	0.94
	60 to 70	0.57	0.69	0.82	0.92
0.5	<25	1.0	1.0	1.0	1.0
	25 to 40	0.93	0.95	0.97	1.0
	40 to 50	0.91	0.92	0.95	0.05–1.0
	50 to 60	0.87	0.88	0.93	0.05–1.0
	60 to 70	0.83	0.85	0.90	0.05–1.0
1.0	All slopes	0.95–1.0	0.95–1.0	1.0	1.0

lower part of the dam. This, along with (2) may explain why these writers assess valley arching to be more important than shown by this analysis.

The influence of valley shape on the vertical stresses within the embankment has been taken into account for the case studies as part of this study by applying a stress reduction factor (Table 3) to the two-dimensional estimate of vertical stress at the end of construction.

Prediction of the Modulus of Compacted Rockfill During Construction

The estimation of the secant moduli of the rockfill during construction from the case study data has been determined from the internal vertical deformation records in close proximity to the dam centerline and from the lower half of the embankment. The proposed method for prediction of the secant modulus during construction of compacted rockfill is to

1. Determine the representative secant modulus at the end of construction E_{rc} from the D_{80} size (size for which 80% is finer) and unconfined compressive strength of the rock in the rockfill using Fig. 3.
   The representative secant modulus at the end of construction E_{rc} is for Zone 3A rockfill placed in layers 0.9 to 1.2 m thick, water added and compacted with four to six passes of a 10 t smooth drum vibratory roller (i.e., well compacted rockfill), and is applicable to average vertical stresses of
   • 1,400 kPa for the very high strength, well-compacted rockfills,
   • 800 kPa for the medium to high strength, well-compacted rockfills, and
   • 1,500 kPa for the well-compacted gravels.
   The D_{80} size should be obtained from construction records, rolling trials or particle size estimated from samples from test pitting into the existing rockfill;
2. For Zone 3B or other rockfills placed in layer thicknesses up to 2 m, apply a correction factor of up to 0.5 to obtain a representative E_{rc}. This correction factor is based on the ratio of E_{rc} (of Zone 3A to Zone 3B) from six field cases. There is not sufficient data to be prescriptive regarding this correction factor, but as a guide, a correction factor of 0.5 would apply to rockfill placed in 2 m layers without the addition of water and compacted with four to six passes of a 10 t vibratory roller (i.e., reasonably compacted rockfill). A correction factor of 0.75 would apply for rockfill placed in 1.5 to 1.6 m layers with the addition of water and compacted with four to six passes of a 10 t vibratory roller (i.e., reasonably to well compacted rockfill);
3. To account for the nonlinearity of the stress-strain relationship for rockfill, estimates of modulus for stress levels less than or greater than the representative E_{rc} are done by
   • For very high strength rockfills apply a correction of $\pm 7.5\%$ per 200 kPa to the E_{rc} estimated from Fig. 3 for a

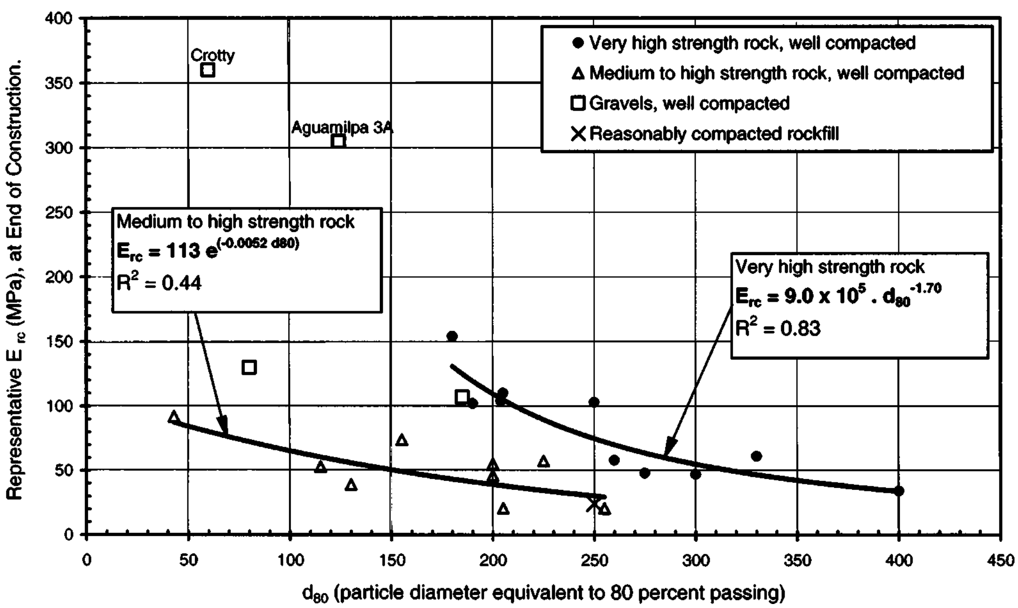

Fig. 3. Representative secant modulus of compacted rockfill at end of construction E_{rc} versus rockfill particle size and unconfined compressive strength

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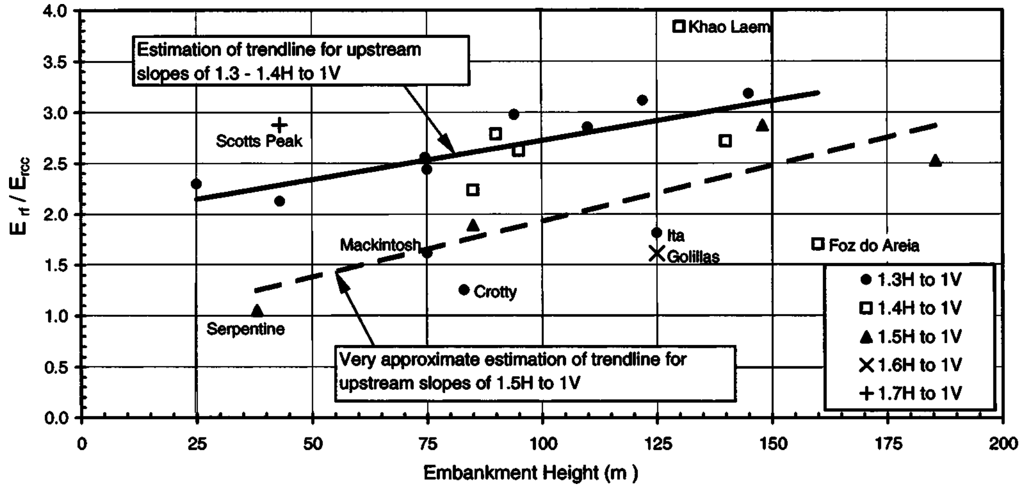

Fig. 4. E_{rf}/E_{rcc} ratio versus embankment height for compacted rockfill

vertical stress of 1,400 kPa. Apply positive corrections for decreasing stresses and negative corrections for increasing stresses. The applicable range is 400 to 1,600 kPa, and

• For medium to high strength rockfills apply a correction of $\pm 6\%$ per 200 kPa to the E_{rc} estimated from Fig. 3 for a vertical stress of 800 kPa. The applicable range is 200 to 1,200 kPa.

4. For medium strength rockfills (UCS of 6 to 20 MPa) apply a multiplication factor of 0.7 to the E_{rc} value determined from the equation for medium to high strength rockfills; and
5. Tangent moduli can be estimated from the secant moduli versus vertical stress relationship estimated in (3).

The following points should be noted:

- C_u, the uniformity coefficient for the particle size distribution curve, is implicitly allowed for in the D_{80} value. Generally, decreasing C_u is observed for increasing D_{80},
- For materials placed with larger rollers (e.g., 13 to 15 t deadweight vibrating rollers) the data does not indicate an increase in moduli for the greater compactive effort. This may be due to a greater material breakdown under the heavier rollers, and a resultant reduction of D_{80},
- For weathered rockfills the intact strength will be lower than for fresh rock, which will result in a decrease in secant moduli. This will be countered by a greater breakdown in the particle size distribution, which will give an increase in moduli associated with a reduction in D_{80},
- If testing on the proposed rockfill material indicates a significant reduction in UCS on wetting is likely and only limited water has been, or is proposed to be, used in construction, the rockfill is likely to be more susceptible to settlement on wetting due to rainfall and flooding. There is insufficient data to advise on how to quantify this,
- Only a limited number of cases of well-compacted gravels were available, insufficient to undertake analysis. These generally have significantly greater secant moduli at any given stress level in comparison to quarried, very high strength rockfill. Where the gravel is of finer size (e.g., Crotty Zone 3A) the use of large scale laboratory test that virtually encompasses the field particle size distribution would provide suitable estimates of moduli provided the layering and density in the field are reflected in the laboratory testing, and
- Monitoring should be undertaken during the embankment construction as a check on the rockfill modulus and preconstruction estimation of deformation.

Data to allow calculation of the moduli of dumped rockfill was not available. It could be anticipated that the modulus would be much lower than for compacted rockfills. Back-analysis of the deformation of a narrow central core earth and rockfill dam built of dry dumped, somewhat weathered basalt rockfill implied an equivalent secant moduli of 5 MPa for the rockfill on the upstream side, but probably higher on the downstream side. At El Infiernillo dam the outer rockfill zone was dry placed and spread without formal compaction in 2 m lifts, and the secant modulus was estimated at 17 to 27 MPa (average 22 MPa). At Ita dam, the estimated secant modulus for the rockfill dumped into 10m of water was in the range 15 to 19 MPa.

Estimation of Settlements during Construction

Settlement during construction can be estimated using the moduli calculated as detailed above, and estimating the vertical stress profile on the dam centerline allowing for embankment shape, using simplified methods (e.g., Poulos and Davis 1974) or numerical analysis. Valley shape can be allowed for using Table 3, or more rigorously using 3D numerical analyses.

In all cases the nonlinearity of the stress-strain relationship should be modeled. In numerical modeling it is essential the model be ‘‘constructed’’ in a number of finite layers.

Method for Estimating Concrete Face Rockfill Dam Face Slab Deformation on First Filling

Approximate Empirical Method

This method uses the empirical approach of Fitzpatrick et al. (1985) shown on Fig. 2 and using equation 2. E_{rf} is estimated from the E_{rf}/E_{rcc} ratio using Fig. 4, where E_{rcc} is the E_{rc} value estimated from Fig. 3 adjusted for vertical stress such that it is representative of the average vertical stress in the lower 50% of the rockfill in the central region of the embankment.

Reasons for a number of the outliers (Scotts Peak, Khao Laem, Mackintosh, Foz do Areia, Crotty, and Ita) in Fig. 4 are detailed in Hunter and Fell (2002). They mostly relate to complications arising from complex zoning (Ita, Crotty, Scotts Peak dams), and face slab deformations measured in a localized narrow section of the valley causing arching (Khao Laem dam).

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The apparently higher modulus implied by E_{rf}/E_{rcc} ratios in the order of 1.5 to 4 is due to the stress paths within the rockfill in the upstream shoulder on first filling. As shown by Saboya and Byrne (1993) the bulk stress within the upstream rockfill increases with an increasing reservoir level during first filling, however, the deviator stress initially decreases during the early stages of filling and then increases in the later stages. Overall, the net effect is for a relatively small increase in deviator stress and a large increase in mean normal stress in the upstream rockfill.

Numerical analyses were carried out to assess the hypothesis put forward by Cooke (1984) that the layer of rockfill would have a higher horizontal moduli than vertical due to the high degree of compaction in the upper part of each layer. These showed this had little effect.

It is reiterated that E_{rf} is not a true rockfill modulus and is only to be used in Eq. (2) to estimate face slab deflection on first filling. The steps to estimate E_{rf} are

1. Estimate the ratio E_{rf}/E_{rcc} based on the embankment height and upstream slope angle using Fig. 4. Trend lines are given for upstream slopes of 1.3–1.4 H to 1 V, and 1.5 H to 1 V, which cover most CFRD designs;
2. Estimate E_{rc}, the representative secant moduli at end of construction, of the Zone 3A rockfill from the methods outlined above. To estimate E_{rcc} adjust the E_{rc} value for vertical stress such that it is representative of the average vertical stress in the lower 50% of the rockfill in the central region of the embankment as outlined above;
3. The E_{rcc} values used in the derivation of Fig. 4 were not corrected for arching effects due to valley shape because valley shape is potentially likely to affect both E_{rcc} and E_{rf}, and would therefore be taken into consideration in the E_{rf}/E_{rcc} ratio. Hence, E_{rc} estimates derived from Fig. 3, which have been corrected for valley shape, must therefore be uncorrected for valley shape by dividing by the stress correction factors in Table 3 to give E_{rcc}; and
4. E_{rf} can then be estimated by multiplication of the E_{rcc} value with the E_{rf}/E_{rcc} ratio.

As is apparent from the scatter of data in Fig. 4, this method is approximate. The trend line for upstream slopes of 1.3–1.4 H to 1 V is considered to provide a reasonable basis for estimation of E_{rf} given it is based on a data set of 12 cases. However, the trend line for the upstream slope of 1.5 H to I V is particularly uncertain given the limited number of cases on which it is based. The method is only applicable for CFRD with relatively simple zoning geometries comprising a significant Zone 3A component (greater than 50 to 60%). The method should not be used where the embankment design is such that the zoning geometry is relatively complex in the upstream shoulder and/or there are large variations in moduli between rockfill zones.

Numerical Analyses

For more reliable estimates of face slab deformation on first filling, finite element or finite difference analyses using the stress-strain relationship determined from Fig. 3 as described above could be used. Where valley shape is likely to be significant, it would be preferable to use 3D modeling.

Postconstruction Crest Settlement of Concrete Face Rockfill Dam

The postconstruction crest settlement behavior for CFRD comprises two main components; an on-going time dependent deformation component, and deformation on first filling due to increases in effective stress conditions above those previously experienced by the rockfill. Collapse compression of the rockfill on wetting can also result in relatively large deformations. However, observations of large collapse settlements postconstruction are limited in CFRD as it requires wetting up of a significant portion of the rockfill, usually as a result of high leakage rates through rockfill of relatively low permeability, and the rockfill to be susceptible to collapse compression.

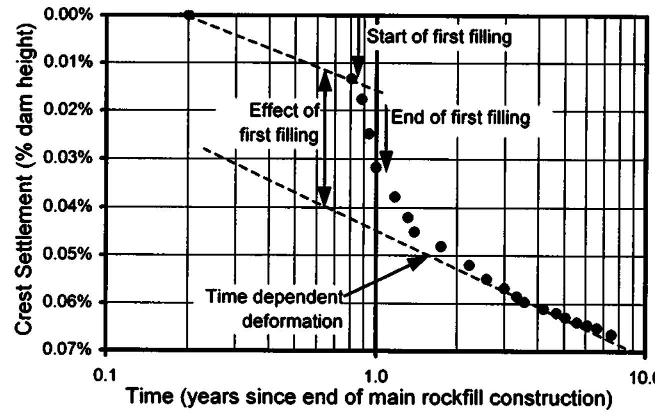

Fig. 5. Postconstruction crest settlement, Bastyan Dam

Where first filling commences more than about 0.3 to 0.5 years after the end of main rockfill construction the effect of first filling is often observed as an acceleration in the rate of crest settlement during the latter stages of and shortly after first filling. Fig. 5 presents a typical example of this effect, showing the "S" shaped curve of the crest settlement versus log time plot. The base point of time (or zero time) for post construction crest settlement has been established at the end of main rockfill construction; i.e., at completion of the Zones 3A and 3B rockfill. In current construction procedures this will be before completion of the concrete face and crest detail.

Crest Settlement during First Filling

Table 4 summarizes the crest settlement during first filling. The settlements (as a percentage of embankment height) are representative of the settlement due to first filling alone, as shown in Fig. 5, by neglecting the effect of on going time dependent deformations. In general, larger crest settlements are observed for dumped than for compacted rockfill, increasing embankment height and for compacted rockfills of lower UCS. However, the data shows a

Table 4. Crest Settlements Attributable to First Filling

Rockfill placement method and intact strength	Embankment height, H (m)	Crest settlement (% of H)
Dumped rockfill	43 to 100	0.08 to 0.55
Compacted gravels	<50 to 60	<0.02
and compacted	50 to 100	0.01 to 0.04
very high	100 to 150	0.02 to 0.09
strength rockfill		(most<0.05)
150 to 180	0.05 to 0.10	
Compacted medium	<40 to 50	0.00 to 0.05
to high	50 to 100	0.05 to 0.15
strength rockfill	100 to 150	0.10 to 0.25

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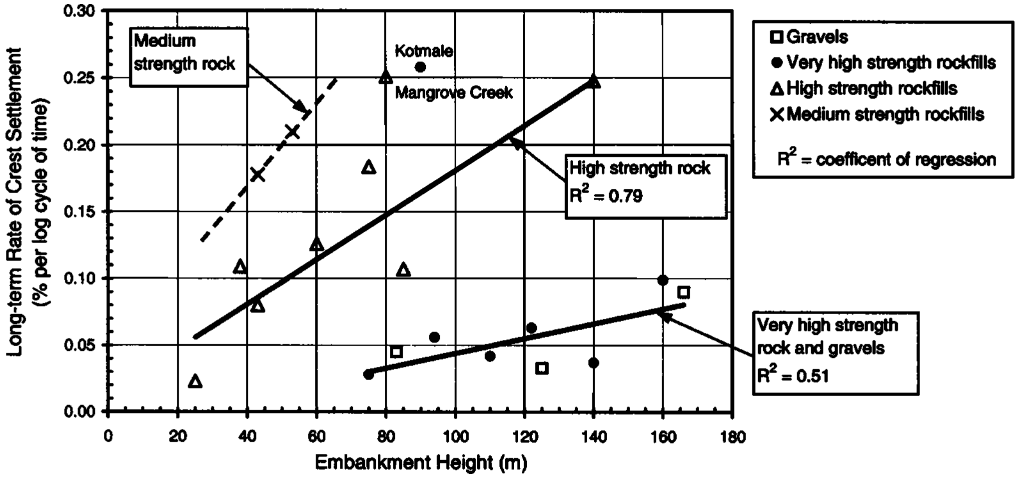

Fig. 6. Long-term crest settlement rates versus embankment height for compacted rockfills

high degree of variability, which is reflected in the broad range of crest settlement within several of the placement method and height groupings. Causes of this variability are likely to include differences in embankment zoning geometry, upstream slope angle, and the timing of and time period for first filling from the end of main rockfill construction. Attempts to incorporate these factors resulted in dilution of the database.

Estimation of the crest settlement due to first filling should be based on the strength and placement method of the dominant rockfill zone under the embankment crest. Within any range, lower values are in general more applicable to CFRD with broad Zone 3A geometries and higher values for narrow Zone 3A (or broad Zone 3B) geometries.

Time Dependent Settlements for Compacted Rockfills and Gravels in Concrete Face Rockfill Dam

Fig. 6 shows the long-term rate of crest settlement (as a percentage of the embankment height per log cycle of time) versus embankment height, for compacted medium, high, and very high strength rockfills. Data points are also shown for CFRD where compacted gravels form the bulk of the main fill. Zero time is taken at the end of construction of the main rockfill and time is in years. Each case study is represented as a point in Fig. 6 for which the crest settlement rate has been estimated from a line of best fit to the portion of the settlement time curve after first filling, as shown in Fig. 5. In addition, division into the various rock strength categories is based on the dominant rockfill zone under the embankment crest, which is not necessarily Zone 3A.

The line of best-fit method assumes a logarithmic relationship exists between crest settlement and time. It is evident from the case study data that this assumption provides a good approximation of the crest settlement for periods of at least 10 to 30 years postconstruction for CFRD constructed of compacted rockfills and gravels when zero time is taken at the end of main rockfill construction. In reality, the actual settlement versus time curve may indicate a reduction or increase in crest settlement with the log of time.

It is evident from Fig. 6 that the significant influences on the long-term rate of crest settlement are the embankment height and intact strength of rock. Other factors that affect the time dependent crest settlement rate include the following:

- Dams with a greater proportion of Zone 3B in the embankment design (see Fig. 1) give higher rates.
- Dams with reservoirs that have large fluctuations in level could be expected to experience higher rates.
- Dams in high rainfall areas, and weathered rockfill, or rockfill subject to weakening on wetting, can be expected to give greater rates. Dams with high leakage rates also tend to have higher long-term crest settlement rates.
- Particle size (as measured by D_{80} and C_u) does not appear to be significant.

Kotmale and Mangrove Creek stand out as outliers to the general trend. For Mangrove Creek this is thought to be due to the fact that first filling to full supply level has yet to be reached after 15 years (the long-term settlement rate is based on the settlement data more than three years after the end of construction). For Kotmale the reason is not clear, although the available crest settlement data is limited to about one year after first filling, and it is possible that the rate (per log cycle of time) may have since reduced.

Postconstruction internal settlements under the embankment crest were analyzed for 13 of the compacted CFRD case studies, most of which are Australian CFRD. The settlements were determined from the records of crest monitoring points and internal settlement gauges and are divided into roughly four zones; the upper 25%, the central upper 25%, the central lower 25%, and the bottom 25%. The settlements were analyzed from both the start of monitoring (or first filling) and also from the end of first filling.

The findings are summarized in Table 5 and indicate the following:

- For very high strength, compacted quarried rockfills and compacted gravels a very high percentage (mostly 60 to 90%) of the total settlement from initial monitoring through first filling occurs in the bottom 25% of the embankment and a small amount (less than 10%) within the upper 25%.
- For the medium to high strength quarried rockfills this trend is evident for some case studies, while for others the distribution is more evenly spread.
- After the end of first filling the percentage settlement in the bottom 25% of the embankment decreases compared to the percentage inclusive of first filling. The amount of the reduction is in the order of 10 to 20%.

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Table 5. Location of Internal Postconstruction Vertical Settlements within CFRD

Name of concrete face rockfill dam	Intact rockfill strength[a]	Dam height (m)	Start of time period[b]	Data records (years)	Percent of total crest settlement below crest: Top 25%	25 to 50%	50 to 75%	Bottom 25%
Crotty	Gravel	83	Start M	2.2	4	6	6	84
Bastyan	VH	75	Start FF	6.75	4		18	78
Cethana	VH	110	Start FF	14.5	0		51	49
			End FF	13.5	(heave)	64	36	
Kotmale	VH	90	Start M	1.5	(heave)	25	12	63
Murchison	VH	94	Start M	11.5	(heave)	27	37	36
			End FF	9.25	14	40	30	15
Reece	VH	122	Start M	8	4		40	79
			End FF	6.25	(heave)		21	49
White Spur	H to VH	43	Start M	5.75	40	8	18	33
			End FF	4.5	42	4	29	24
Tullabardine	H	25	Start M	5	8	52	40	
Winneke	H	85	Start M	15.5		11		89
			End FF	10.5		20		80
Mangrove Creek[c]	H	80	Start M	8.3	5	43	26	26
Mackintosh	M to H	75	Start M	7.5	(heave)	23	31	46
			End FF	6	(heave)	23	32	45
Serpentine	M to H	38	Start M	8	25	42	16	17
			Start FF	4	15	51	13	21
Scotts Peak	M	43	Start M	17	15	7	29	49
			End FF	14	37	9	15	39

[a]M = monitoring, FF = first filling.

[b]VH, H, and M refer to very high, high, and medium intact strength, respectively.

[c]For Mangrove Creek the central region (25 to 75%) is representative of the random fill zone, Zone 3B.

These observations can be explained by the fact that on first filling the increase in stress on the dam centerline is greatest at the base of the embankment, the nonlinear stress strain relationship of the rockfill and the modest dependency of long-term settlement rates on the stress level. Factors such as loss of strength on wetting of the upper parts of the dam by rainfall infiltration can affect the post construction settlement distribution, as can details of the zoning.

Time Dependent Settlements for Dumped Rockfill Concrete Face Rockfll Dam

Sherard and Cooke (1987) observed that the crest settlement of dumped rockfill CFRD are five to eight times and the time dependent settlement rate is 10 to 20 times that for similar compacted rockfill dams. Analysis of the data for the seven dumped CFRD indicates an increasing rate of crest settlement as a percentage of dam height (per log cycle of time) with time. Estimates of the long-term crest settlement rate (per log cycle of time) can be derived from Table 6 based on the time period after the end of main rockfill construction.

Table 6. Long-Term Crest Settlement Rates for Dumped Rockfill (CFRD)

Time period (From end of main rockfill construction)	Postfirst filling crest settlement rate (% per log cycle of time): Range	Mean
0.5 to 5 years	0.10 to 0.58	0.27
5 to 20 years	0.25 to 1.14	0.66
20 years plus	0.33 to 1.44	0.85

Conclusions

The methods presented here to estimate rockfill modulus, crest settlements of CFRD, and face slab deformations of CFRD, are based on a broad database of well instrumented dams with detailed information on material properties and placement methods. The methods are likely to give more reliable estimates than other published data because of the more extensive data, and the consideration of a greater number of factors that influence the compressibility of the rockfill and the postconstruction deformation behavior. Laboratory testing of rockfill for determination of its compressibility properties is inherently difficult because of the limitations of particle size in testing equipment and the difficulty in modeling the segregation and differential compaction within layers, so methods such as those presented here are likely to be more realistic. There are however large uncertainties, and it is wise to monitor deformation during construction to confirm properties.

Where existing dams are performing well outside the parameters shown here, it may indicate some deterioration in rock properties, or effects of leakage, or that construction was not as recorded in the drawings and specification, with more weathered, or lower strength rock used, or poor compaction practices used. More details on the study are available in Hunter and Fell (2002).

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