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Appendix

Partial list of CFRDs with basic data, current as of Nov, 2004
PREFACE

Bulletin 70, Rockfill Dams with Concrete Facing, was published in 1989. The Bulletin was authored by Jorge E. Hacelas and Alberto Marulanda on behalf of the Colombian Committee on Large Dams for the ICOLD Committee on Materials for Fill Dams. In addition to Bulletin 70, the following were the main sources for this update:

- Proceedings, Second Symposium on Concrete Face Rockfill Dams, Brazilian Committee on Dams, Florianopolis, Brazil, October, 1999.
- J. Barry Cooke Volume, Concrete Face Rockfill Dams, Beijing, September 2000.

Following the 1985 symposium in Detroit, USA, and during the decade of the 1990s, the concrete face rockfill dam has become common. A cursory review of the listing of CFRDs in the appendix indicates the widespread use and popularity of this type of dam.

The updated Bulletin contains eleven chapters devoted to design concepts, analysis, foundation treatment, instrumentation, construction, and performance. The work of the following authors is acknowledged:

Chapters 1, 2, 3, 4, 8, and 10: David E. Kleiner
Chapters 5 and 6: Jason E. Hedien
Chapters 7 and 9: Archie V. Sundaram and David E. Kleiner
Chapter 11: Carlos Jaramillo
Appendix: J. Barry Cooke

The Chairman, Alberto Marulanda, provided detailed review and many contributions to all chapters that added significantly to the bulletin.

Alberto Marulanda, Chairman
Committee on Materials for Fill Dams
Chapter 1

DEVELOPMENT OF THE CONCRETE FACE ROCKFILL DAM

The concrete face rockfill dam, CFRD, had its origin in the mining region of the Sierra Nevada in California in the 1850s. Experience up to 1960 using dumped rockfill, demonstrated the CFRD to be a safe and economical type of dam, but subject to concrete face damage and leakage caused by the high compressibility of the segregated dumped rockfill. As a result, the CFRD became unpopular, although rockfill had been demonstrated to be a high strength and economical dam building material. Partly in response to these problems, the earth core rockfill dam, with compressible dumped rockfill, was developed. The dumped rockfill was found to be compatible with the earth core and its filters. With the advent of vibratory-roller-compacted rockfill in the 1950s, the development of the CFRD resumed. Although design is largely based on precedent, there has been continuous progress in design aspects and in construction methods. Today, the CFRD is again a major dam type. Figure 1-1 illustrates the trends in the height of the CFRD up to the year 2000.

Figure 1-1  Trends in the Height of the CFRD with Time (Cooke, 1997, extended to 2000)
Several CFRDs, 140 m high or higher, completed or under construction since the year 2000 include:

- Antamina, Peru, 140 m high, completed in 2001.
- Mohale, Lesotho, 145 m high, completed in 2002.
- *Campos Novos, Brazil, 202 m high, under construction, 2003.
- *Barra Grande, Brazil, 140 m high, under construction, 2003.
- *Bakun, Sarawak, Malaysia, 205 m high, under construction, 2003.


1.1 Typical Current Section of the CFRD

Figure 1-2 is a schematic section of the CFRD consisting of sound compacted rockfill founded on a sound rock foundation. Outer slopes can be as steep as 1.3H:1V. For a weaker rockfill and foundation, upstream and downstream slopes, zoning, drainage and construction are adapted to accommodate the weak rock. For a potentially erodible foundation, additional sealing and filter provisions are constructed downstream of the plinth.

The zone designations of 1, 2, and 3 have become the standard:

- Zones 1A, 1B – concrete face protection (upstream) zones, in increasing order of maximum particle size,
- Zones 2A, 2B – concrete face supporting (downstream) zones, in increasing order of maximum particle size, these are processed granular materials, and
- Zones 3A, 3B, etc. – rockfill zones, in increasing order of maximum particle size.

Zone 1B provides support for Zone 1A and in some cases also resists uplift of the face slab prior to reservoir filling. Zone 1A, a cohesionless silt or fine sand, is placed to a higher elevation on high dams so that it can act as a joint or crack healer over the perimeter joint and the lower part of the face slab. Compaction of Zones 1A and of the random Zone 1B is by hauling and spreading equipment.

Zone 2A is a processed fine filter with specific gradation limits, minus 20 mm or minus 12 mm. It is to limit leakage in the event of waterstop failure and to self heal with underwater placement of silt or silty fine sand. Zone 2B, the face support zone, has often been specified as crusher run minus 75 mm sound rock material. Alternatively, specific gradation limits are specified. The zones 2A and 2B, their gradation, placement and protection during construction, have received considerable attention recently. A detailed discussion of these materials is contained in Chapter 8, Fill Materials.
Zone 3 is quarry run rockfill. The differences in A, B and C are principally in layer thickness and size and type of rock. Zone 3A is to provide compatibility and limit void size adjacent to Zone 2B.

---

Figure 1-2. Zones for CFRD of sound rock on sound rock foundation (adapted from Cooke, 1991, 1997)
Zone 3B resists the water loading and limits face deflection. Zone 3C receives little water loading, and settlement is essentially during construction. The thicker layer in Zone 3C accepts larger rock, is more economical to place, and its lower density (about 5% less than Zone 3B density) saves rock volume. Large rock is often placed at the downstream toe to resist scour and tailwater wave action.

The typical section is shown for rockfill. Gravel, when available in adequate quantity, can be more economical even with the necessary flatter slopes. Its higher modulus is desirable but not always necessary. The layer thicknesses for Zones 3B and 3C of gravel are thinner, on the order of 0.6 m and 1.2 m respectively.

Figure 1-3 illustrates a layer of compacted rock. The rockfill is end dumped on the edge of the layer being placed and spread by the dozer. There is inherent segregation in the dumping and intentional segregation in the spreading. The smooth surface of fines on top of the layer is desirable for compaction and for reduced tire and dozer track costs. The top half consists of smaller size rock and is well graded in comparison to the larger rocks in the bottom half. The upper half is of higher density. Energy is transmitted through the larger rocks providing strength and density by wedging and crushing of edges. A method specification is used; density tests are sometimes taken for the record. The A, B and C density designations in the figure are respectively for poorly, average and well graded quarry run rock placed in 1 m layers and compacted by four passes of the 10 static ton vibratory roller. All the densities are satisfactory depending on the specific gravity of the rock and the void ratio of the fill. Low void ratios are desirable and lead to the least settlement within the fill.

The maximum size rock in a layer may be equal to the layer thickness. Immediately adjacent rockfill will not be fully compacted and does not need to be. The larger rock particles will attract load in the area.

1.2 Summary Of 1965-2000 Progress To Current Practice

During the 1965-2000-development period, many CFRDs were adopted to replace a previously selected arch, gravity or earth-core-rockfill dam type. Reasons for the change to the CFRD included
the late discovery of adverse foundation conditions for a concrete dam, cost, or lack of appropriate core material for an earth-core-rockfill dam. Today, the CFRD is an established major dam type to be included in initial project feasibility studies. A summary of progress and current practice is:

1. Precedent maximum heights have jumped from 90 to 187 m. Maximum heights under construction or planned exceed 200 m.

2. The reinforced concrete plinth anchored to the foundation, first used at Exchequer and Cabin Creek, has since become standard practice. Up to about 1958, standard practice for the connection of the concrete face with the rock foundation was a concrete cutoff as described by Cooke, 1960, with examples of Salt Spring Dam, Lower Bear River Dam and Wishon Dam. This practice changed dramatically after Terzaghi, 1960, provided a discussion of the Cooke paper. After stating that the cutoff serves only the purpose of reducing the seepage losses to a tolerable value, he proposed that the most economical procedure for intercepting the flow of seepage would be to eliminate the concrete cutoff, and replace it by a plinth anchored to the foundation, and to grout the rock beneath the slab. His final remarks are appropriate: “it is rather difficult to understand how the brutal practice of blasting a cutoff trench out of sound rock came into existence. It may be the vestige of the days when the technique of rock grouting was still unknown”.

3. Foundation treatment below and downstream of the plinth always receives close attention.

4. Gravel is used in the dam cross section whenever economically available.

5. When rockfill is not positively free draining, liberal provision is made for internal drainage.

6. Face zones of highly pervious, semi-pervious and impervious material have all been satisfactory. Current practice of crusher-run minus 50 or 75 mm rock, obtained from a sound, competent source, is satisfactory, economical and practical.

7. Plate vibrator compaction within 3 m of the perimeter joint on the horizontal and sloped surface is now required. The vibratory roller cannot get close enough to adequately compact this zone, and poor compaction has been a cause of excessive offset and of waterstop damage. Many modern dams now use the “curb” method to provide face protection during construction and the ability to achieve good compaction of Zones 2A and 2B. Use of the curb eliminates the need for plate vibrator compaction (Resende and Materon, 2000).

8. A fine filter, minus 20 or 12 mm, is specified within 1 to 3 m of the perimeter joint, the location where leakage incidents have occurred. It limits leakage and allows sealing by silt or silty fine sand, a non-cohesive material.

9. Face slab thickness has been reduced from $0.3 + 0.0067 \ H$ to $0.3 + 0.002 \ H$ in meters, or is a constant 0.3 meters for dams of moderate height.
10. Reinforcing has been reduced from 0.5% each way to 0.3% horizontal and 0.35 or 0.4% vertical and near the abutments.

11. Experience, with partial filling due to extreme floods before the concrete face has been placed, has demonstrated the ability of rockfill to accept high leakage safely. Rockfill is an effective energy dissipater.

1.3 Features Of The CFRD

The CFRD dam has many attractive features in design, construction, and schedule.

Design Features.

1. All of the zoned rockfill is downstream from the water barrier. The sliding factor of safety often exceeds 7. The dam supports the abutments.

2. A plinth with appropriate foundation treatment below, upstream and/or downstream, connects the water barrier (concrete face slab) to the foundation. A parapet wall at the crest provides a wider surface for construction of the face slab and reduces the volume of rockfill.

3. Uplift is not an issue. The pressure on the foundation exceeds reservoir pressure over three-quarters of the base width.

4. Water load is transmitted into the foundation upstream from the dam axis, an inherently safe feature.

5. Since all of the rockfill is dry, earthquake shaking cannot cause internal pore water pressure.

6. The conditions of high shear strength, no pore pressure, and small settlement under seismic loading make the zoned rockfill inherently resistant to seismic loading.

7. The only credible mechanism of failure of a CFRD founded on rock is erosion by sustained overtopping flow. Hydrology, spillway, and freeboard design is the response to this risk. Piping of the foundation is a potential mode of failure as a result of the increasing use of CFRDs on weathered rock and alluvial foundations.

8. Post construction movements are small, and cease after several years.

9. Surveillance by monitoring surface movement and measuring leakage is required, as for any dam, but little or no instrumentation is needed for safety monitoring.
Construction and Schedule Features.

1. Ramps are permitted within the body of the dam in any direction. This minimizes haul roads to the dam and facilitates traffic and placement on the dam. Inappropriate construction of ramps, delay in placement of rockfill within the downstream shoulder of the dam to accommodate staging, and haul roads that cross the plinth leaving holes to be filled later, can cause irregular settlement of the dam and can lead to cracking of the concrete face prior to and subsequent to reservoir filling.

2. Where site conditions permit, rockfill may be placed on abutments prior to river diversion. This allows required excavations to be placed directly into the dam. On major rivers, early placement of rockfill on abutments decreases the volume of rockfill in the closure section, thus reducing or eliminating overtopping risk during construction.

3. CFRDs allow great flexibility for the management of the river during construction. Their natural strength to overtopping, combined with special design features, such as, reinforced rockfill in the downstream face, and RCC cofferdams, allow the use of lower interval recurrence floods and still have an equivalent risk exposure during the construction period to other types of embankment dams.

4. The plinth construction and grouting are outside the dam and do not interfere with embankment placement or the construction schedule.

5. Rockfill placement is relatively unrestricted and not affected by rainfall. Scheduling is reliable.

6. The slip forming of the concrete face is a repetitive planned procedure that can be reliably scheduled.

7. The concrete face can be constructed in stages at the convenience of the Contractor. Too many stages and delay in placement of rockfill in the downstream shoulder of the dam can cause adverse settlement that can affect the performance of the concrete face slab prior to and subsequent to reservoir filling.

8. The plinth and internal slab can be constructed by slipforming simplifying construction.

9. The parapet wall located at the crest of the dam can be constructed of precast elements, thus improving schedule.

10. The use of the upstream extruded curb have reduced segregation of the transition 2B and eliminating upstream slope compaction (Resende and Materon, 2000).

1.4 Evaluation of Leakage Performance

The leakage performance of the CFRD has recently been criticized in the professional literature (Anthiniac, et al, 2002). Four case histories were studied and evaluated using numerical analysis:
Aguamilpa, Mexico, 187 m high; Xingo, Brazil, 140 m high; Tianshengqiao 1, China, 180 m high; and Ita, Brazil, 125 m high. Based on the analysis of these four dams, the authors state:

“The safety of the dams was never called into question, since the materials of which they are made enable water to flow out freely without causing any damage, but the leakage rates were deemed to be too high, given the type of dam and the functions involved. Moreover, the current trend is to accept increasingly high leakage rates, implying that leakage is not a danger.”

Particular design details and/or construction defects lead to larger absolute and differential deformations of the face slab, which can then lead to face slab cracking. Avoiding these causes by means of appropriate selection of filter and rockfill materials, upstream and downstream shell placement in thinner layers along with generous use of water during compaction, elimination of rock protrusions downstream of the perimeter joint, and avoiding inappropriate shell construction sequences, will lead to smaller deformations and a reduction in face slab cracking. One of the purposes of this Bulletin is to emphasize that careful selection of design and construction details is extremely important to avoid face cracking and high leakage rates. Nevertheless there are occasions, especially for large rivers where high leakage rates are economically acceptable as they do not impair the safety of the dam.

It is certainly not the position of this Bulletin or of ICOLD “to accept increasingly high leakage rates” in CFRDs. Many modern CFRDs, designed and constructed in recent years, have performed extremely well with respect to leakage rates (see Chapter 11, Performance). As an example, the recently completed Antamina CFRD in Peru, 140 m high, had a leakage rate of less than one liter per second upon completion of reservoir filling.

1.5 References


Chapter 2

ANALYSES FOR DESIGN

Although few analyses are required for the design of the CFRD, several analyses are suggested to provide the engineer and owner with information upon which to judge the performance of the dam throughout the life of the project from construction, first reservoir filling, and normal project operation. Estimates of performance during an extreme event, such as an earthquake or major flood are useful for comparison in the aftermath of such an event.

Most design details are developed based on precedent and on an understanding of the foundation conditions and the construction materials to be used in the dam. This chapter summarizes several simplified analyses that can be performed to evaluate static and dynamic stability, settlement and displacement, and leakage.

This chapter also summarizes the performance of several rockfill dams that have successfully survived major earthquakes. In addition, a case history of the design analyses and anticipated response of a 200-m tall CFRD to static and seismic loading is presented. Both empirical techniques and the finite element method (FEM) were used to estimate the response and performance of the dam.

2.1 Static Stability of the CFRD

Shear Strength of Compacted Rockfill

Leps (1970) reviewed the shear strength of compacted rockfill and gravel fill as measured with the use of large diameter laboratory triaxial tests. The summary plot of the data, as presented in the paper, is shown on Figure 2-1. The shear strength, measured by the angle of internal friction, is plotted against the normal stress on the failure plane. Note that shear strength includes no apparent cohesion. The data clearly indicates the variation of shear strength with normal pressure. In general, Leps found that:

- At normal stress below about 70 kPa (10 psi), the angle of internal friction varies from about 45° for low density, poorly graded weak particles to as high as 60° for high density, well graded strong particles. Leps defined weak particles as rock having an unconfined compressive strength of 3.4 to 17.2 MPa (500 to 2500 psi) and strong particles as rock having an unconfined compressive strength of 69 to 207 MPa (10,000 to 30,000 psi).
- Friction angles reduce by 6° or 7° per 10 times increase in the normal pressure on the failure plane.
- Well graded materials exhibit higher shear strength than poorly graded materials.
- Higher density materials exhibit higher shear strength than low density materials.
Angular materials exhibit higher shear strength than rounded materials, other factors being equal.

Dry materials exhibit higher shear strength than saturated materials.

Data from other rockfill and gravel fill dams generally support the findings reported by Leps, as presented in ICOLD Bulletin, 92, “Rock Materials of Rockfill Dams”.

Figure 2-1  Shear Strength of Rockfill from Large Triaxial Tests (Leps, 1970)

Various studies of the shear strength of rockfill (Marsal 1973, Barton and Kjærnsli 1981, Charles and Watts 1980, ICOLD 1993, and others) confirmed that the actual behavior of rockfill is non linear, and that a relation between the shear stress and the normal stress is of the form:

\[ \tau = A \cdot (\sigma')^b \]

where:
- \( \tau \) = shear stress,
- \( \sigma' \) = effective normal stress,
- A, b = empirical coefficients that depend on the type of rock.
Infinite Slope Stability Analysis

A simple infinite slope stability analysis using a friction angle of $50^\circ$ and a 1.3H:1V slope yields a factor of safety of 1.55, satisfactory for well compacted rockfill. Slopes as steep as 1.2H:1V have been used on the downstream slope between access road berms on some CFRDs. Because modern compaction equipment can easily and routinely create a dense, high strength fill, outer slopes of the CFRD are selected based on:

- The height of the dam. Somewhat flatter slopes are selected for dams exceeding 120 m.
- The quality of the rockfill. Flatter slopes are selected when poorer quality rock is used.
- The seismicity of the region in which the CFRD is to be constructed. Flatter slopes are selected when the project is located in a region with strong seismicity.

Simple Limiting Equilibrium Stability Analyses

Figure 2-2 presents the results of limiting equilibrium analysis performed at the crest of a 200-m tall CFRD. The parapet wall is 7 m tall, outer slopes are 1.5H:1V, and freeboard above the maximum operating pool is 15 m. The analysis was performed to provide input to an estimate of the performance of the dam during the maximum design earthquake, assumed equal to the maximum credible earthquake. The analysis conservatively used an angle of internal friction equal to $40^\circ$. Factors of safety between 2.1 and 2.2 were calculated. Computed factors of safety, using friction angles of 45 to $50^\circ$, would yield substantially higher factors of safety.

Relatively high static factors of safety can be anticipated when performing slope stability analysis of the CFRD. This is partly the result of the high frictional shear strength that is present and partly the result of the absence of saturation and internal pore water pressure.

Limited equilibrium stability analyses, with and without seismic effects, are also required of potential failure surfaces passing through both the embankment and foundations, where the foundation contains weak seams (Casinader and Stapledon, 1979, Gosschalk and Kulasinghe, 1985).

2.2 Dynamic Stability of the CFRD

Measured Performance of Rockfill Dams During Earthquake

Table 2-1 presents the measured earthquake induced deformation of rockfill dams. The data was compiled by Swaisgood and presented at the May 1995, western regional conference of the Association of State Dam Safety Officials. Relative settlement is the measured crest settlement of the dam divided by the combined height of the dam plus any underlying alluvium expressed as percent. The Earthquake Severity Index was added to the Swaisgood table based on the estimated and recorded peak ground accelerations that occurred at the dam site.
Figure 2-2, Limit Equilibrium Analysis at Crest of CFRD

### Table 2-1

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<td>0.38</td>
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<td>ECRD</td>
<td>52.1</td>
<td>0.0</td>
<td>Nihonkai-Chubu</td>
<td>1983</td>
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<td>0.08</td>
<td>5.8</td>
<td>0.11</td>
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<tr>
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<td>ECRD</td>
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<td>75.3</td>
<td>Playa Azul</td>
<td>1981</td>
<td>7.3</td>
<td>0.09</td>
<td>14.3</td>
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<td>6.1</td>
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<td>0.0</td>
<td>Playa Azul</td>
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<td>7.3</td>
<td>0.05</td>
<td>6.4</td>
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<tr>
<td>North Dike</td>
<td>California</td>
<td>ECRD</td>
<td>35.7</td>
<td>0.0</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>0.42</td>
<td>3.0</td>
<td>0.09</td>
</tr>
<tr>
<td>El Infiernillo</td>
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<td>ECRD</td>
<td>146.0</td>
<td>0.0</td>
<td>Mich.-Guerrero</td>
<td>1985</td>
<td>8.1</td>
<td>0.13</td>
<td>11.0</td>
<td>0.08</td>
</tr>
<tr>
<td>San Justo</td>
<td>California</td>
<td>ECRD</td>
<td>39.9</td>
<td>14.0</td>
<td>Loma Prieta</td>
<td>1989</td>
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<td>3.7</td>
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</tr>
<tr>
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<td>0.0</td>
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<td>1990</td>
<td>7.7</td>
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<td>4.3</td>
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</tr>
<tr>
<td>Leroy Anderson</td>
<td>California</td>
<td>ECRD</td>
<td>71.6</td>
<td>0.0</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>7.1</td>
<td>0.26</td>
<td>4.3</td>
<td>0.06</td>
</tr>
<tr>
<td>Cogswell</td>
<td>California</td>
<td>CFRD</td>
<td>81.1</td>
<td>0.0</td>
<td>Sierra Madre</td>
<td>1991</td>
<td>5.8</td>
<td>0.46</td>
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</tr>
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<td>0.0</td>
<td>Playa Azul</td>
<td>1979</td>
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</tr>
<tr>
<td>Nagara</td>
<td>Japan</td>
<td>ECRD</td>
<td>52.1</td>
<td>?</td>
<td>Chiba-Toh</td>
<td>1967</td>
<td>6.9</td>
<td>0.27</td>
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</tr>
<tr>
<td>La Villita</td>
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<td>ECRD</td>
<td>69.1</td>
<td>75.3</td>
<td>n/a</td>
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<td>0.02</td>
<td>4.6</td>
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<td>Taiwan</td>
<td>ECRD</td>
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<td></td>
</tr>
<tr>
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<td>CFRD</td>
<td>81.1</td>
<td>0.0</td>
<td>Northridge</td>
<td>1994</td>
<td>6.7</td>
<td>0.10</td>
<td>2.1</td>
<td>0.03</td>
</tr>
<tr>
<td>El Infiernillo</td>
<td>Mexico</td>
<td>ECRD</td>
<td>146.0</td>
<td>0.0</td>
<td>n/a</td>
<td>1975</td>
<td>5.9</td>
<td>0.08</td>
<td>3.7</td>
<td>0.03</td>
</tr>
<tr>
<td>Leroy Anderson</td>
<td>California</td>
<td>ECRD</td>
<td>71.6</td>
<td>0.0</td>
<td>Morgan Hill</td>
<td>1984</td>
<td>6.2</td>
<td>0.41</td>
<td>1.5</td>
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<tr>
<td>Miboro</td>
<td>Japan</td>
<td>ECRD</td>
<td>129.9</td>
<td>0.0</td>
<td>Kitamino</td>
<td>1961</td>
<td>7.0</td>
<td>0.15</td>
<td>2.7</td>
<td>0.02</td>
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<td>75.3</td>
<td>n/a</td>
<td>1975</td>
<td>7.2</td>
<td>0.04</td>
<td>2.4</td>
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</tr>
<tr>
<td>El Infiernillo</td>
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<td>ECRD</td>
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<td>0.0</td>
<td>n/a</td>
<td>1975</td>
<td>7.2</td>
<td>0.03</td>
<td>2.4</td>
<td>0.02</td>
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<tr>
<td>Magat</td>
<td>Philippines</td>
<td>ECRD</td>
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<td>0.0</td>
<td>Philippines</td>
<td>1990</td>
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<td>0.05</td>
<td>0.6</td>
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<td>Oroville</td>
<td>California</td>
<td>ECRD</td>
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<td>1975</td>
<td>5.9</td>
<td>0.10</td>
<td>0.9</td>
<td>0.00</td>
</tr>
</tbody>
</table>

**Legend:**
- DH: Height of dam in m
- AT: Thickness of alluvium below the dam in m
- ECRD: Earth core rockfill dam
- CFRD: Concrete face rockfill dam
- PGA: Peak ground acceleration
- Relative Settlement: Crest settlement divided by the combined dam height and thickness of alluvium, in %
- Earthquake Severity Index: $\text{PGA} \times (\text{Earthquake Magnitude} - 4.5)^3$
The Earthquake Severity Index, introduced by Bureau, 1985, is defined as follows:

\[
ESI = PGA \times (M - 4.5)^3
\]

where:

- \(ESI\) = Earthquake Severity Index
- \(PGA\) = Peak horizontal ground acceleration at the site
- \(M\) = Earthquake Magnitude

As shown on Figure 2-3, a rough relationship exists between the Relative Settlement and the Earthquake Severity Index.

In general, both types of rockfill dams, those with earth cores (ECRD) and those with a concrete face (CFRD), have performed well during large earthquakes. A major difference between these dam types is that the upstream shell of the ECRD is saturated by the reservoir, whereas in the CFRD, no portion of the embankment is saturated. Except for the potential of cracks in the concrete face or in the parapet wall, the performance of the CFRD during earthquake is anticipated to be as good as the ECRD.

There are only a few records of the performance of CFRDs during and subsequent to an earthquake. Table 2-1 presents data for three CFRDs: Minase in Japan, Cogoti in Chile and Cogswell in California.
Minase Dam. In June, 1964, Minase Dam was shaken by the Niigata Earthquake (M 7.5, 147 km epicentral distance from the dam, 750 mm/s² estimated peak ground acceleration at the dam). As a result of this earthquake, the crest settled about 150 mm and displaced horizontally about 100 mm. The earthquake temporarily increased leakage from about 100 l/s to somewhat over 200 l/s. Within a few days, leakage returned to pre-earthquake levels. Further discussion of the performance of Minase Dam is presented in Chapter 11, Performance of CFRDs.

Cogoti Dam. Arrau, et al, 1985, reported on the performance of the Cogoti CFRD during the 1943, magnitude 7.9, earthquake. The dam, located approximately 90 km from the epicenter, settled nearly 400 mm but little other damage occurred. Increased leakage as a result of the earthquake or cracking of the concrete slab was not reported. The only repair to the dam subsequent to the earthquake was the replacement of rockfill at the crest where settlement of the rockfill away from the concrete face had occurred. The rockfill forming the body of the dam was dumped in lifts of “greatest height practicable”. At the time of construction, 1938, the rockfill for the embankment of the typical CFRD was end dumped in lifts from 30 to 50 m high. Crest settlement subsequent to completion of construction in the five years prior to the earthquake was about 400 mm, approximately equal to the instantaneous settlement that occurred during the earthquake. The crest settled an additional 300 mm in the 42 years between 1943 and 1985. In spite of the lack of compaction of the rockfill, the dam suffered remarkably little damage.

Cogswell CFRD. Cooke (1995), reports on the performance of the Cogswell CFRD, constructed in 1933, during the 1991 Sierra Madre Earthquake, Magnitude 5.8. The crest of the dam settled about 40 mm and displaced horizontally about 20 mm. Vertical cracks occurred in the concrete face adjacent to each abutment. Cracks on the right side extended 11 m down from the crest; cracks on the left side extended 5 m down. During construction, the rockfill was dumped in 7 meter lifts without compaction and without sluicing with water. Upon completion of embankment construction, heavy rains caused the 80-m high fill to settle more than six meters. This unanticipated settlement was caused by the loss of strength of the fill upon saturation. Prior to the placement of the concrete face, the rockfill was thoroughly wetted to achieve further settlement. Further settlement of 400 mm occurred. In 1994, the dam was shaken again by the magnitude 6.7, Northridge Earthquake. An additional 20 mm of crest settlement was measured. Again, in spite of the lack of compaction of the rockfill, the dam suffered remarkably little damage.

Sugesawa CFRD. Masumoto et al, 2001, reported on the performance of dams as a result of the October 2000 earthquake on the main island in Japan. The magnitude Mw was 6.6 with epicenter at a depth of 11 km. The Sugesawa dam consists of a concrete gravity dam, 73.5 m tall, and a CFRD saddle dam in the right abutment, 17 m tall. Peak ground acceleration in the right abutment was 0.36g. The performance of both dams was satisfactory.

Torata CFRD. EERI 2003, reported on the performance of the Torata CFRD, during 2001, Magnitude Mw 8.4 earthquake. The level of ground shaking at the site was estimated to range between 0.12 to 0.33g, and the mean PGA of 0.20g. The dam was built to divert the Torata River to the Cuajone pit project in Perú. The dam is a 130-meter-high CFRD. The upstream concrete face consists of 300 and 500 mm thick adjacent slabs, separated by vertical construction joints spaced every 15 m, fitted with water stops. Before the earthquake, maximum settlements at the
crest and mid-length of the concrete face slab were 460 and 190 mm, respectively. The maximum horizontal displacements at the same locations were 510 and 140 mm. Settlements and horizontal displacements of the concrete face at the crest as a result of the June 2001 earthquake, showed a maximum settlement and horizontal displacement of 62 and 36 mm, respectively. The earthquake caused minor cracking and joint separation in the concrete face near the left abutment. This was the result of seismic compression that occurred at that location.

In summary, ECRDs and CFRDs have performed well during large earthquakes. In spite of the poorly compacted rockfill in the older concrete face rockfill dams, remarkably little damage has occurred.

Anticipated Performance of a 200-m tall CFRD

For purposes of the analysis presented herein, the following definitions are used:

**Maximum Credible Earthquake (MCE).** The MCE is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a geographically defined tectonic province under the presently known or presumed tectonic framework. The MCE is defined as an upper bound of expected magnitude.

**Maximum Design Earthquake (MDE).** The MDE will produce the maximum level of ground motion for which the dam should be designed or analyzed. Typically, for dams whose failure would present a hazard to life, the MDE is characterized by a level of motion equal to that expected at the site from occurrence of the controlling MCE. It is required that the impounding capacity of the dam be maintained when subjected to that seismic load.

**Operating Basis Earthquake (OBE).** The OBE represents the level of ground motions at the dam site that would result in only minor and an acceptable level of damage. USCOLD defines the OBE as the level of ground motion with a 50% probability of not being exceeded in 100 years. The dams, appurtenant structures, and equipment should remain functional and damage easily repairable from occurrence of earthquake motion not exceeding the OBE.

Where fault geometry and activity are well known, the deterministic method should be used to estimate the MCE. In locations where the nature of active faulting is not well known, an annual probability on the order of 1/3,000 to 1/10,000 is recommended to define input motion representing the MCE, depending on the risk rating of the structures (U.S. Committee on Large Dams, 1999). For projects classified with a high risk rating, the 10,000 year period of return is recommended for design. This corresponds to an approximate 1 percent probability of exceedance in a period of 100 years.

**Selection of Seismic Design Parameters.** In this example, it is assumed that an active regional fault is located within about 26 km of the dam site and that the fault can generate an MCE equal to magnitude 7.8. An earthquake of this magnitude could produce peak ground acceleration at the dam site on the order of 0.6g. Because the dam is a high hazard structure, the MDE is selected equal to the MCE. The following parameters are selected:
Earthquake magnitude: 7.8
Peak bedrock acceleration at the dam site: 0.6 g.

The anticipated response of the dam to the MDE can be evaluated using empirical methods of analysis based on actual performance of dams during earthquakes and on simplified analytical methods using procedures suggested Makdisi and Seed (1977) and Bureau (1997).

**Earthquake Severity Index.** For this analysis, it is assumed that the rockfill materials for the dam will be obtained from required excavations and from quarries and that these sources, when properly compacted, will produce a well-graded high density rockfill.

The anticipated response of the dam to earthquake motions can be estimated by direct comparison with the actual performance of rockfill dams to large earthquakes. This approach was followed by Bureau (1985) and is further expanded herein. Table 2-1 presents the deformation of rockfill dams in terms of the actual settlement at the crest of the dam and Bureau’s Earthquake Severity Index (ESI), as defined earlier. Figure 2-3 presents the relationship between Relative Settlement of the dam as measured at the crest and the ESI.

The ESI for the MDE (PGA = 0.6 g, M = 7.8) is equal to 21.6. The estimated crest settlement taken directly from Figure 2-3 is as follows:

<table>
<thead>
<tr>
<th>Estimated crest settlement in response to the MDE, m</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Bound of Data in Figure 2-3</td>
</tr>
<tr>
<td>Mean of Data in Figure 2-3</td>
</tr>
</tbody>
</table>

The anticipated response to a large earthquake compares favorably to the planned 15 m of freeboard above the normal maximum operating pool. During the 1990 earthquake in northern Luzon in the Philippines, the 120-m high Ambuklao Dam experienced crest settlement and deformation on the order of one meter (USCOLD, 1992).

**Makdisi and Seed Method.** In the 1965 Rankine Lecture, Newmark introduced a method to estimate earthquake-induced displacements in embankment dams based on the concept that slope movements are initiated when inertia forces on a potentially sliding mass exceed the available yield resistance along the bounding surface of failure. Newmark treated the sliding mass as a rigid body. Makdisi and Seed (1977) modified Newmark’s approach by recognizing that an embankment dam responds as a flexible structure and introduced a technique to estimate the amplification of the ground motions to the crest of the dam. The analysis, then, is based on estimating the maximum peak crest acceleration \( \ddot{u}_{\text{max}} \) for a given ground motion then determining the maximum acceleration of the potentially sliding mass, \( k_{\text{max}} \). The yield acceleration, \( k_y \), of the sliding mass is estimated by finding the average horizontal acceleration coefficient in a conventional slope stability analysis which will obtain a factor of safety equal to 1.0. This coefficient is defined as the ratio of a horizontal destabilizing force (as might be caused by an earthquake) to the weight of the sliding mass. The ratio of \( k_y \) to \( k_{\text{max}} \) can then be used to estimate...
displacement at the crest of the dam. This estimated displacement has both a horizontal and vertical component.

For this analysis, the amplification of peak ground acceleration from base to crest of dam was estimated by using Jansen's unpublished plot, titled "Measured Ratios (Amplification) of Crest and Base Accelerations at Embankment Dams in Response to Earthquakes", Figure 2-4. The value of $k_{\text{max}}$ was assumed to be equal to $\ddot{u}_{\text{max}}$ which is equal to the peak ground acceleration times the amplification factor. Based on previous analyses of high dams and judgment, the fundamental period of the dam was selected as 1.5 sec. The value of $k_y$ was determined based on conventional stability analysis as summarized below:

<table>
<thead>
<tr>
<th>Factor of Safety without Earthquake</th>
<th>Yield Acceleration, $k_y$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upstream slope</td>
<td>2.2</td>
</tr>
<tr>
<td>Downstream slope</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Notes:

1. The graph represents measured accelerations at embankment dams ranging widely in size, geometry, materials, and foundation conditions.
2. The two plotted values for La Villita Dam for each indicated year are based on the positive and negative amplitudes from asymmetric accelerograms of crest motion.
3. The envelope is drawn as an upper limit of amplifications, reflecting the average of La Villita peak crest accelerations in the 1985 earthquake.

Figure 2-4 (unpublished Jansen, 1994)
The Makdisi and Seed charts, taken from Bureau (1997), are shown on Figure 2-5. The procedure indicates a displacement of about 1.3 m during the MDE. Use of a friction angle of 45 to 50° would yield factors of safety approaching 3.0. A larger yield acceleration and an estimated displacement less than one meter would result. Again, the estimated displacement compares favorably with the 15 m of freeboard.

**Bureau’s Method.** As an extension of the 1985 analysis, Bureau (1997) presented a chart, Figure 2-6, that relates the Relative Crest Settlement to the Earthquake Severity Index for several values of the friction angle of the fill material. The chart is based on finite element analyses of typical rockfill dams. Note that the settlement must be obtained by multiplying the height of the dam by the value read from the chart, then dividing by 100. Use of this method indicates a displacement of about two meters during the MDE, when using a friction angle of 40°. Use of 50° friction would indicate a crest displacement on the order of 1 meter. Again, the estimated displacement compares favorably with the 15 m of freeboard above the normal maximum pool elevation.

**Anticipated Response.** Based on the above empirical analyses, the estimated settlement or displacement at the crest of the 200-m tall CFRD is on the order of one meter. These movements that might occur during the MDE could lead to cracking and settlement of the fill at the crest and to cracking and joint separation within the parapet wall and the concrete face at the crest. An increase in leakage as a result of cracks and settlement could be expected but this would not result in a question concerning the fundamental safety of the dam.
2.3 Defensive Design Concepts

Materials, Concrete Face and Drainage

The concrete face will be supported by processed crushed rock, high strength and high modulus materials. Because the water barrier is located at the upstream face of the dam, the embankment materials will not be saturated and, therefore, no deformations will take place during or subsequent to an earthquake as a result of increased pore water pressure within the CFRD. Rockfill zoning is such that permeability increases progressively from upstream to downstream.

If the embankment consists of semi-pervious sands and gravels, an inclined drainage zone consisting of processed alluvium or crushed rock should be provided to separate the upstream zones from the downstream sand and gravel and rockfill zones. This drainage zone should be continuous from abutment to abutment and from the base to the crest of the dam. The drain should be connected to a high capacity underdrain located at the base of the dam. Provisions to monitor the flow from the underdrain should be incorporated into the design so that flow rates can be monitored during first filling, during project operation and immediately after earthquakes and floods. Measurements of flow rates, deformations, and joint movements should be taken to evaluate the overall performance of the dam subsequent to earthquake.
Design Features

The design should incorporate defensive features against the effects of earthquake. Ample freeboard above the normal maximum pool elevation should be provided to mitigate against the effects of a major earthquake. The freeboard above the maximum operating pool elevation should be not less than three to four times the maximum estimated deformations that might occur during the maximum design earthquake. Often, the maximum operating pool elevation is located below the horizontal joint between the parapet wall and the concrete face. This design requirement automatically provides a freeboard, for high CFRDS, in excess of four or more meters.

2.4 Settlement and Compression

Deformation Modulus

The deformation modulus varies widely depending on the void ratio of the rockfill and the parent rock material. Uniformly graded rockfill, such as that used at Foz do Areia and Segredo in Brazil have low deformation moduli. Compacted gravel fill dams have considerably higher moduli. The moduli are derived from measurements of vertical settlement during construction and the calculated vertical fill load above the settlement gage, as follows:

\[ E_v = \frac{H \cdot \gamma_r \cdot h}{1000 \cdot s} \]

Where:
- \( E_v \) = Vertical deformation modulus, MPa
- \( H \) = Vertical depth of rockfill above the settlement gage, m
- \( \gamma_r \) = Unit weight of rockfill, kN/m³
- \( h \) = Column of rockfill below the settlement gage, m
- \( s \) = Settlement of the gage, m

Several projects that illustrate the range of calculated moduli, based on field measurements, are listed below:

<table>
<thead>
<tr>
<th>Project</th>
<th>Rock Type</th>
<th>Deformation Modulus, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foz do Areia</td>
<td>Basalt</td>
<td>32</td>
</tr>
<tr>
<td>Segredo</td>
<td>Basalt</td>
<td>45</td>
</tr>
<tr>
<td>Aguamilpa</td>
<td>Gravel</td>
<td>190</td>
</tr>
<tr>
<td>Salvajina</td>
<td>Clean Gravel</td>
<td>390</td>
</tr>
<tr>
<td>Alto Anchicaya</td>
<td>Hornfels-diorite</td>
<td>145</td>
</tr>
<tr>
<td>Golillas</td>
<td>Dirty gravels</td>
<td>210</td>
</tr>
</tbody>
</table>

Pinto and Marques (1998) evaluated the moduli of deformation of various rockfill materials with respect to the void ratio and the shape of the canyon or valley in which the several dams were constructed. Their data are plotted on Figure 2-7 and are shown on Table 2-2. Two curves are plotted in the figure. The shape factor is defined as the area, \( A \), of the concrete face in m² divided by the maximum height of the dam, \( H \), squared. For narrow canyons with shape factor, \( A/H^2 \), equal to three or less, the indicated moduli of deformation are larger as shown by the upper curve.
in the figure. This appears to be the result of arching across the canyon and stress transfer of load into the abutments. Thus, the measured settlements are less than those that might be expected by evaluating the void ratio of the material and the calculated vertical load above the settlement gage.

**Figure 2-7 Correlation Deformation Modulus vs Void Ratio (Pinto and Marques, 1998)**

**Estimating Construction Settlement**

Settlement during construction at any location within the embankment varies with the deformation modulus, the thickness of compressible material beneath the location of interest, and the load on the compressible material. The simple relationship, modulus of deformation, $E_v$, is equal to stress (the load placed on the compressible material) divided by strain (settlement of the top of the layer divided by the thickness of the layer) can be used to estimate the settlement, and, during construction, the relationship can be used to calculate the modulus of deformation based on measurements of settlement. If, for example, a column within an embankment dam is divided into 10 horizontal layers, the settlement at the top of the bottom layer, $0.1H$ in thickness, caused by the load of one layer, $0.1H$ in thickness, placed on top of the bottom layer is equal to:

$$S = \left( \frac{\gamma r \cdot H^2}{100 \cdot E_v} \right)$$
Where:

\[ \begin{align*}
S &= \text{settlement in meters} \\
\gamma_r &= \text{unit weight of rockfill, MN/m}^3 \\
H &= \text{height of the column within the dam in meters} \\
E_v &= \text{vertical deformation modulus, MPa}
\end{align*} \]

For example, the settlement of the top of the 20-m thick base layer at the bottom of a 200-m tall CFRD with a deformation modulus equal to 100 MPa, under a load of one 20-m layer of rockfill with a unit weight of 22 kN/m\(^3\), is 0.09 m. Under the nine layers of rockfill, 180m, the top of the 20-m thick layer at the base of the dam would settle 0.09 times 9 equals 0.8 m. The maximum settlement within the dam would occur at about mid-height. At this location, five layers of compressible material are located below mid-height and five layers are located above. The settlement at mid-height in the example 200-m tall dam would be 0.09 times 5 times 5 equals 2.2 m. Note that the base of the basal layer does not settle because the analysis assumes an incompressible foundation. Also, at the instant of completion of the dam, no settlement of the crest occurs because the added load is zero.

The above example demonstrates that the settlement of the surface of each layer will be proportional to the product of the number of layers below that elevation and the number of layers of fill above. When the embankment is divided into 10 layers as in the example, the settlement of each layer will be approximately proportional to the Distribution Factor shown in the following chart. It may be seen that the distribution of vertical settlement within the dam is roughly parabolic, with the maximum settlement occurring at about mid-height. On the day the dam is completed, the settlement of the crest and the base is zero.

<table>
<thead>
<tr>
<th>Layer Number</th>
<th>Layers Above the Top of the Layer Number</th>
<th>Distribution Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<td>0</td>
</tr>
<tr>
<td>1</td>
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<td>9</td>
</tr>
<tr>
<td>2</td>
<td>8</td>
<td>16</td>
</tr>
<tr>
<td>3</td>
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<td>6</td>
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<tr>
<td>7</td>
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<td>9</td>
</tr>
<tr>
<td>10</td>
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<td>0</td>
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</tbody>
</table>

These simple techniques can be used to back-calculate the deformation modulus during construction when the dam is partially complete.
## Construction and Behavior Parameters of some CFRDs

<table>
<thead>
<tr>
<th>Dam</th>
<th>Country</th>
<th>Year and Height, m</th>
<th>Rock Type</th>
<th>L m</th>
<th>A 1000 m²</th>
<th>$\gamma_r$ kN/m³</th>
<th>$\varepsilon$</th>
<th>$E_v$ MPa</th>
<th>$A/H^2$</th>
<th>D m</th>
<th>Joint movement, mm</th>
<th>Leak V</th>
<th>Leakage l/s</th>
<th>ET MPa</th>
<th>ET/E T</th>
<th>$E_T$** MPa</th>
<th>$E_T/E_T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cethana</td>
<td>Australia</td>
<td>1971 110</td>
<td>Quartzite</td>
<td>213</td>
<td>24</td>
<td>26.5</td>
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<td>135</td>
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<td>11.5</td>
<td>-</td>
<td>7</td>
<td>300</td>
<td>2.2</td>
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<tr>
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<td>Hornfels-Diorite</td>
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<td>106</td>
<td>15</td>
<td>1800/180*</td>
<td>440</td>
<td>3.0</td>
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<td>Basalt</td>
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<td>1993 140</td>
<td>Basalt</td>
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<td>-</td>
<td>-</td>
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<td>1994 140</td>
<td>Granite</td>
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<td>37</td>
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<td>0.30</td>
<td>30</td>
<td>34</td>
<td>-</td>
<td>180</td>
<td>190</td>
<td>5.1</td>
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<tr>
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<td>1993 187</td>
<td>Gravel</td>
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<td>137</td>
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<td>190</td>
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<td>&lt;0.15</td>
<td>19</td>
<td>16</td>
<td>5.5</td>
<td>260/100</td>
<td>680</td>
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<td>Salvajina</td>
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<td>Gravel</td>
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<td>28.0</td>
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<td>&lt;0.10</td>
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<td>Gravel</td>
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<td>210</td>
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<td>0.16</td>
<td>-</td>
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<td>-</td>
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<td>310</td>
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<tr>
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<td>Nigeria</td>
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<td>76</td>
<td>4.2</td>
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<td>30 &gt;50</td>
<td>21</td>
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<td>Lower Pieman</td>
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<td>1986 122</td>
<td>Dolerite</td>
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<td>0.24</td>
<td>160</td>
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<td>-</td>
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<td>Mackintosh</td>
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<td>1981 75</td>
<td>Graywacke</td>
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<td>40</td>
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<tr>
<td>Murchison</td>
<td>Australia</td>
<td>1982 89</td>
<td>Rhiolite</td>
<td>-</td>
<td>16</td>
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<td>0.17</td>
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<td>Bastyan</td>
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<td>20</td>
<td>5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

*Initial leakage/value after repair works. **Computed by formula $E_T = 0.003 H^2/D$ (MPa)

Where:

- $L$ = Crest length, m
- $A$ = Face area in 1000s of square meters
- $\gamma_r$ = In-situ unit weight of rockfill, kN/m³
- $\varepsilon$ = Void ratio
- $E_v$ = Vertical deformation modulus, MPa
- $D$ = Deformation of the face slab, measured at mid-height, with reservoir full, m
- $O$ = Joint opening perpendicular to the perimeter joint, mm
- $S$ = Settlement measured perpendicular to the face slab, mm
- $T$ = Shear movement measured parallel to the plinth, mm
- $E_T$ = Deformation modulus, measured perpendicular to the face slab, as a result of reservoir filling, MPa

Table 2–2
From Pinto and Marques, 1998
Estimating Construction Compression

During construction, settlement takes place causing the rockfill to compress. An estimate of the total compression to be expected can be estimated using the settlement expression derived above. Compression in terms of percent of the height, H, of a column within the dam is equal to the summation of the settlement within each of the 10 layers of the dam. The following expression is derived:

\[ C = S \times \frac{100}{H} \times (9+8+7+6+5+4+3+2+1+0) = 45 \times \gamma_r \times \frac{H}{E} \]

Thus, the compression, C, of a 200-m tall column within the example rockfill dam, with E equal to 100 MPa and \( \gamma_r \) equal to 0.022 MN/m³, is 2.0% of the column height. The compression, C, of a 100-m high column within the 200-m high rockfill dam would be about 1.0%. Using these simple procedures, the overall compression of the rockfill dam during construction can be estimated.

2.5 Estimating Face Slab Deformation

Pinto and Marques, 1998, present an empirical approach to estimating maximum face slab deformation under the load of the reservoir. Maximum face deformation is measured normal to the face slab and occurs at about 0.4 to 0.5 of the dam height. Face movements, as with settlement during construction, are proportional to \( H^2/E_t \). \( E_t \) is the transverse modulus of deformation measured in the direction of movement under the load of the reservoir and is larger than the vertical modulus of deformation, \( E_v \), measured during construction. The compression of the rockfill that occurred during construction creates a denser fill with a higher transverse modulus of deformation. The database developed by Pinto and Marques is presented in Table 2-2.

As previously discussed, the valley shape factor, \( A/H^2 \), affects the construction deformation modulus, \( E_v \). In narrow valleys, with lower values of the shape factor, the arching effects across the valley reduce the load within a vertical column of rockfill at the maximum height section, thus reducing the measured settlement. The smaller settlement results in a larger estimate of the vertical modulus of deformation, \( E_v \). In this case, the indicated transverse modulus would only be slightly larger than the calculated \( E_v \). The ratio between the estimated \( E_v \), using the formula provided in Table 2-2, and \( E_v \), is based on measurements of vertical settlement and maximum measured face displacement in the dam. The resulting calculated ratio was then plotted against the valley shape factor as shown in Figure 2-8. As can be seen from the figure, larger ratios of \( E_v/E_v \) result from larger values of the shape factor, \( A/H^2 \). The results of the data analysis by Pinto and Marques are shown in Figure 2-9. For example, the estimated maximum face deformation in a 200-m tall dam under full reservoir load would be on the order of 0.4 m, if the vertical modulus of deformation during construction, \( E_v \), were equal to 100 MPa and if the dam were located in a valley with shape factor equal to 4.
These simple relationships between valley shape, modulus of deformation during construction, and the maximum height of the dam can be used to estimate the performance of the dam during the first filling of the reservoir.

Figure 2-8  Ratio Transverse to Vertical Modulus as a Function of A/H² (Pinto and Marques, 1998)
2.6 Estimating Seepage Through the Foundation and the Slab

Leakage is a key parameter that relates to the overall performance of the CFRD. Large leakage rates are an indication that damage has occurred to the perimeter joint and/or that the concrete face has cracked to some extent. Seepage through the foundation may also be a contributing factor to large leakage rates.

Seepage through the foundation can be estimated following the usual concepts of flow in porous media, or more complex methods that include the effect of discontinuities in the rock mass, and the effect of the grout curtain. (Giesecke et al. 1992).

The fundamental design concept of the CFRD is that the several embankment zones of the dam including the face support material, filters, transitions, underdrainage and the body of the dam must remain stable even if extremely large leakage rates were to occur. The ability of rockfill to accept and pass large flows is well known in the literature. Thus, if the embankment zones and the foundation treatment have been designed and constructed appropriately, the large leakage rates are not an indication that safety is a problem, but rather that remedial treatment may be required to reduce the leakage.
Flow through Cracks

The importance of designing and constructing appropriate treatment at joints is easily demonstrated by developing estimates of leakage through potential openings at the perimeter joint or through cracks in the face slab. The rate of flow through a crack is commonly expressed as being proportional to the crack width cubed. C. Louis, 1969, using the cubic equation and based on experimental studies, developed a model for flow through a crack as follows:

\[
q = \frac{gw^3i}{12v\left[1 + 8.8\left(\frac{m}{2w}\right)^3\right]}
\]

Where:

- \(q\) = unit flow rate, \(m^3/s/meter\) of crack length
- \(g\) = acceleration of gravity = 9.81 m/s\(^2\)
- \(w\) = crack width, meters
- \(i\) = hydraulic gradient, where \(i = h/d\)
  - \(h\) = frictional head loss associated with flow through the crack, meters
  - \(d\) = depth of crack through which head loss occurs, meters
- \(v\) = kinematic viscosity of water, \(1 \times 10^{-6}\) m\(^2/s\) at 20\(^\circ\) C
- \(m\) = roughness parameter, approximately equal to the dimension of protusions into the crack, meters

If the roughness parameter, \(m\), is defined as some fraction or multiple, \(a\), of the crack width, \(w\), then the above equation can be written:

\[
q = \frac{817500w^3i}{1 + 8.8\left(\frac{a}{2}\right)^3}
\]

For a crack with smooth side walls, such as a smooth joint surface in rock or a pre-formed joint in concrete, the value of “\(a\)” might be on the order of 0.1, (\(m = 0.1w\)). For a crack with rough side walls such as a hairline crack in concrete, the value of “\(a\)” might be on the order of 1.0 or 2.0 (\(m = 1.0\) to 2.0\(w\)). The following table provides insight concerning the magnitude of the rate of flow that can be experienced through one-meter long cracks of several different widths with varying values of roughness. Note that the above equation cannot be used for turbulent flow conditions. Thus, the above equation is not applicable for flow rates that exceed about 0.2 l/s/meter of crack. A discussion of the use of the C. Louis equation as applied at several dams in Australia can be found in the paper by Casinader and Rome, 1988.
Estimates of Rates of Flow through a Crack

<table>
<thead>
<tr>
<th>Crack width, mm</th>
<th>Roughness a</th>
<th>Head loss, h, m</th>
<th>Depth of crack, d, m</th>
<th>Gradient h/d</th>
<th>Flow rate, q, m³/s/m</th>
<th>Flow rate, q, l/s/m</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>0.1</td>
<td>100</td>
<td>0.6</td>
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<tr>
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<td>3.75E-04</td>
<td>0.38</td>
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</table>

For the examples presented in the table, a constant value of head loss through a constant depth of crack was used to illustrate the range of estimated flow rates that can occur through hairline cracks and through a damaged or poorly constructed joint. As can be seen from the table, hairline cracks, on the order of 0.1 mm wide, will not allow a large rate of flow, even under a high gradient and with rough side walls. Flow rates increase dramatically when the crack is 0.3 mm wide or wider and the wall of the crack is smooth. It is clear that great care is necessary in developing the joint details and in constructing the support of the face slab especially at the perimeter joint.

2.7 References


CHAPTER 3
FOUNDATION EXCAVATION AND TREATMENT

3.1 Foundation Treatment Objectives

Foundation treatment for the CFRD consists of

- Excavation,
- Foundation surface preparation at the plinth and beneath the body of the dam. This includes removal of unstable or unsuitable foundation material from beneath the plinth and the body of the dam. If this becomes impractical, defensive measures that preclude erosion and piping of the material must be utilized.
- Drilling and grouting and/or positive cutoffs beneath the plinth,
- Specific treatment of seams or defects both upstream and downstream of the plinth,
- Foundation and abutment drainage, and
- Combinations of the above techniques.

The dam must accommodate variable foundation conditions and the selected foundation treatment must be compatible with the dam and with the foundation characteristics.

Often, foundation conditions are erratic and difficult to define. Success depends on careful subsurface investigations to disclose strata or lenses critical to stability or seepage, design of appropriate foundation treatment, and careful execution of foundation excavation and treatment during construction. Construction often reveals conditions that were not observed during the site investigations. When this occurs, foundation treatment concepts must be reviewed and appropriate changes made.

In summary, foundation treatment must achieve the following fundamental objectives:

- Positive control of seepage beneath or around the plinth,
- Removal of unstable or unsuitable foundation material from beneath the plinth and the body of the dam,
- Preparation of foundation surfaces to receive concrete, filters and rockfill, and
- Limiting differential settlements of the plinth, the face slab and the perimeter joint.
3.2 Plinth Foundation Treatment

A continuous reinforced concrete plinth, cast on a competent foundation at an acceptable depth along the upstream toe of the dam, forms the ideal watertight connection between the concrete face slab and the rock foundation. The plinth is anchored into rock with steel bars and serves as the grout cap for foundation consolidation and curtain grouting.

The plinth is normally founded on hard, non-erodible fresh rock that can be treated by grouting. This is necessary because hydraulic gradients on the order of 15 to 20 develop along the short seepage path under the plinth. With appropriate foundation treatment, however, weathered and jointed rock, fault zones, soil-filled joints and materials susceptible to possible erosion and piping, including firm saprolites, are acceptable. The plinth must be designed to accommodate the foundation conditions and hydraulic gradients within the foundation below the plinth will be reduced as necessary to fit the specific characteristics of the foundation. Criteria are presented in Chapter 4, Plinth, that relate plinth width to foundation conditions. When changes in foundation stiffness are abrupt, the possibility of significant differential movements must be carefully studied and the plinth design adapted to the findings.

In general, for hard, competent, non-erodible rock surfaces, cleaning and preparation of these surfaces subsequent to general excavation for the plinth includes:

- Excavate soft material from cracks, crevices, joints, fractures, and cavities,
- Develop a comprehensive geologic map of the plinth foundation surface for the construction record,
- Clean surfaces with air and high pressure water, unless the rock surface can be damaged using water, in which case, use air only,
- Backfill cracks, crevices, joints, fractures, and cavities with concrete. This can be achieved concurrently with the placement of the foundation concrete below the plinth or with the placement of the plinth.

Plinths have been constructed on alluvial soils at several projects including two in Chile, Santa Juana and Puclaro. Castro, et al, lists 10 CFRDs with plinths founded on alluvium that are currently in operation, five of which are located in China. These dams vary in height from 28 m to Santa Juana, 106 m. Vertical cutoff walls were constructed within the alluvium and tied to the plinth to control seepage through the foundation. Where the CFRD is constructed on an alluvial foundation, it is important that the materials left in place will be stable under all static and dynamic (seismic) loading conditions and that deformations within the foundation are small such that the resultant movements at joints in the plinth or cracking of the concrete face will not lead to excessive leakage.

Static and dynamic analyses of the proposed 131-m high Caracoles CFRD in Argentina indicate that the characteristics of the alluvium will allow the dam and plinth to be founded on the alluvium, Castro, et al. The alluvial gravels, cobbles and boulders that are typical of materials eroded from the southern part of the Andes Mountains in South America provide suitable...
foundations. The materials are highly pervious, thus requiring cutoff, and, at the same time, exhibit low compressibility and are stable under large seismic loadings.

Chapter 4, Plinth, presents further discussion concerning of the width of the plinth, internal and external extensions, and the analyses and means to ensure stability of the plinth and backfill concrete.

The various defensive design measures that can be adopted to appropriately treat difficult foundations are illustrated with the following case histories.

**Salvajina Dam, Colombia, 1983**

The plinth of the Salvajina dam was founded on a widely varying foundation. The lower 65 m of the dam abutments consist of hard quartz sandstone and siliceous siltstone with thinly interbedded hard shales. The upper 90 m of the plinth foundation consists mainly of a weathered to intensely fractured, interbedded siltstone and friable sandstone. In the upper part of the right abutment, a large igneous intrusion is present that is deeply weathered to a saprolite. This saprolite (residual soil: MH-ML according to the Unified Soil Classification System) is a reddish silty material sensitive to piping. Seams of hydrothermal alteration, prone to piping, of varying thickness (10-150 mm) are present in the siltstone. Plinth details adopted at this site are shown in Figures 3-1, 3-2, and 3-3 and can be summarized as follows:

- The width of the plinth was increased from 1/20 to 1/25 of the water head H to 1/16 H. For intensely fractured rock and intensely weathered rock, the width was increased to 1/9 H and 1/6 H, respectively to obtain hydraulic gradients of 9 and 6. Selection of construction details resulted in smaller actual gradients.

- In zones of altered sandstone and siltstone, a hand-excavated concrete back-filled cut off, 1 m wide and 3 m deep, was tied to the plinth, see Figure 3-1, to prevent a seepage blow-out at the plinth contact. Where the saprolite was encountered, the trench depth was limited to 0.6 m and the cut off was made deformable by filling the trench with asphalt-impregnated sand (10-12% asphalt) to prevent cracking as a result of differential settlements, see Figures 3-2 and 3-3. A waterstop was placed at the junction of the cut off with the plinth.

- For the plinth resting on residual soil, differential settlements with respect to the adjacent plinth founded on rock were expected. Cracking was avoided by constructing three transverse cold joints with PVC waterstops in the plinth to add flexibility, see Figure 3-2.

- The excavated residual soil and deeply weathered foundation surfaces downstream of the face slab and upstream of the dam axis were covered with a concrete sand filter with an additional layer of the face supporting gravels as transition to prevent any possibility of migration of eroded fines into the embankment, see Figures 3-1 and 3-2. This treatment provides confidence that suitable filters placed downstream of the face slab can be used at almost all sites with rock foundations, even if they present extensive weathering zones.
• Deep, low pressure (100-200 kPa) consolidation grouting was carried out throughout the entire plinth foundation, except in the area where the low permeability saprolite was present. Consolidation grouting was carried out in a single stage through holes at 4 m spacing perpendicular to the slab and depths ranging between 5 and 10 m, depending on rock quality. Additional holes were drilled and grouted in different directions to intersect particular features, such as a steep set of open relief joints running parallel to the valley.

• Seams of weathered, crushed and friable rock were excavated to a depth of 3 to 4 times the width of the seam and backfilled with either mortar or concrete. Particular attention was given to those features that cross the plinth foundation transversally.

• All upstream exposed rock slopes resulting from excavation for the plinth were covered with a layer of steel-mesh-reinforced shotcrete to extend the seepage path.

Sugarloaf (formerly Winneke Dam), Australia, 1979

At Sugarloaf (formerly Winneke) Dam, the depth of weathering of the siltstone foundation rock was such that the plinth could not be economically founded on groutable rock. As a result of the presence of dispersive clays in seams within both the highly weathered and fresh rock, specific measures were taken to minimize erosion of these seams under seepage forces. The presence of systematic weak seams in the foundation also gave rise to concern about the plinth sliding downstream due to water load. The following measures, as illustrated in Figure 3-4, were adopted:

• To deal with the infill seams, present mainly in the upper 6 m of the highly weathered zone, the upstream toe was excavated to at least 6 m into the highly weathered rock, in the form of a wide toe trench.

• The width of the plinth was set at 0.1 H or 6 m minimum. The rock foundation downstream of the plinth was blanketed with a concrete slab 150 mm thick to such a distance that the hydraulic gradient across the plinth plus the downstream concrete extension was not greater than 2.

• Grout holes were flushed with air and water under pressure, prior to grouting, to flush out as much of the dispersive clay as possible.

• The foundation surface for a distance downstream of the concrete blanket, equal to half the reservoir head, was covered with filter material.

• To ensure the stability of the plinth against sliding, inclined anchors were placed to connect the plinth to the rock at depth. Where anchors were not used, a buttress was constructed at the downstream side of the plinth extending it far enough into the embankment so that pressure from the fill could be more assuredly mobilized.
Figure 3-1 Salvajina plinth founded on less competent rock
Figure 3-2 Salvajina plinth founded on residual soil Section A-A
Figure 3-3 Salvajina plinth founded on residual soil. Section B-B
Mohale Dam, Lesotho, 2001

At Mohale, two erodible seams cross the right abutment, one at about mid-height, the other toward the top of the abutment. It is conceivable that the seams could open as a result of reservoir loading. Treatment of the mid-height seam consists of multiple defenses:

- an upstream impervious blanket placed in a trench excavated along the seam,
• over-excavation of the seam at the plinth and construction of a “socle” below the plinth, 50 m long and 18 m wide,

• “stitch” grouting across the seam from the upstream edge to the downstream edge of the socle in seven rows, 16 holes per row, and

• fine filter and transition protection over the seam beneath the entire body of the dam.

In addition, an adit was driven from the downstream right abutment parallel to the mid-height seam to provide locations for observation and for access should remedial treatment become necessary.

Puclaro Dam, 2000, and Santa Juana Dam, 1995, Chile

The 80-m high Puclaro Dam in northern Chile is founded on alluvium, 113 m maximum depth. The design is based on several considerations:

• Site investigations indicated that the alluvium is of low compressibility and stable under seismic loadings, so that no substantial deformation will occur when the reservoir is filled,

• A diaphragm wall, penetrating the alluvium to a maximum depth of 60 m, will restrict seepage to a maximum of about 250 l/s, and

• A flexible structure can be designed and constructed to connect the diaphragm wall to the plinth so that any deformations experienced within the alluvium can be absorbed without distress. A detail of the conceptual design of this connection is shown on Figure 3-5.

The perimeter joint contains a bottom copper waterstop; a mastic filler with a Hypalon membrane covers the top of the joint. In addition, a non-cohesive soil covers the perimeter joint, the diaphragm wall and the concrete connector slabs.

The 113-m high Santa Juana Dam is founded on alluvium up to 30 m deep. The design of the valley plinth is similar to the Puclaro design except that only one slab connects the diaphragm wall with the plinth instead of two as was selected for Puclaro. The reservoir was filled in 1997 and its performance to date has been excellent, with total seepage lower than 50 l/s.

Both projects were constructed in areas of high seismicity. The maximum possible earthquake is estimated to have a return period on the order of 500 years and to produce a peak bedrock acceleration of 0.54g for Puclaro Dam and 0.56g for Santa Juana Dam.
The above case histories present a variety of design measures that are available to treat widely differing foundations to meet the fundamental objectives listed above. These measures:

- Increase the seepage path length to reduce the hydraulic gradient and to eliminate the possibility of erosion or piping in the foundation,
- Control, cut off and/or reduce the rate of seepage through the foundation, and
- Provide filtered exits for seepage beneath the body of the dam to prevent any conceivable migration of fines into the rockfill.

**Pichi-Picun-Leufu, 1999, Argentina**

Pichi-Picun-Leufu dam is located in the region of Patagonia, in southern Argentina. This 40 m high dam has performed extremely well since completion, with measured leakage of about 13.5 l/s. Because of the highly dispersive local clays, the original design, compacted gravel with a clay core, was changed to compacted gravel with concrete face.

The dam is founded on thick alluvial deposits, mainly consisting of sandy gravel with sporadic seams of sand and silt. In almost all of the plinth extensions, the gravel was removed 9 m. downstream of the plinth and substituted with well-compactcd material. This allowed construction of the dam in advance of the plinth and foundation treatment and backfill over the exposed rock downstream of the plinth. Loose alluvial sands and silts were removed from below the body of the dam to avoid the potential for strength loss during or subsequent to an earthquake. The deposits, left in place, are dense and exhibit low compressibility. Measured deformations were small, because of the low compressible gravel in the dam and foundation.

In general, excavation for the plinth was extended to a rock foundation using a concrete block of varying thickness. In the paleo-channel where the weathering of the rock was more intensive, an anchored concrete block 6 m. high was constructed under the plinth to be supported on sound rock.
El Pescador

Consolidation grouting under low pressure was performed throughout the entire plinth foundation, considering the micro-fractured rock conditions. The typical 4-meter plinth extension, was increased to 8 m in the right abutment because of the presence of completely weathered and fractured rock. The dam has performed well, the reservoir is almost full and the leakage is below 3 l/s.

Khao Laem, Thailand, 1985

This multipurpose project in central west Thailand faced extremely difficult foundation conditions at the alignment of the plinth and under the body of the dam (Watakeekul and Coles, 1985). The left abutment and valley section of the dam is founded on interbedded shale, sandstone, siltstone both calcareous and non-calcereous, locally interbedded with limestone. The strata have undergone severe faulting. Partially infilled cavities up to several meters across were encountered, many found along specific geologic features, such as faults.

The right abutment is founded on a karstic limestone that extended hundreds of meters into the abutment. Extensive solution cavities and caverns were present up to several meters across that were partly or completely plugged with clay and sand infill. Other cavities were empty with the cavity walls coated with crystalline calcite. Solution features were found to depths of 200 m below the base of the dam.

Along the plinth alignment, excavation to depths of 15 to 60 m was performed to a foundation surface from which cutoff treatment could be carried out. Foundation treatment included the following:

- Open excavated and backfilled trenches on the left abutment,
- Concrete diaphragm wall, total area, 15,500 m²,
- Deep curtain grouting to a maximum depth of 180 m, and
- High pressure air and water flushing to remove infill material before grouting.

A permanent inspection gallery was added on top of the plinth for monitoring the effectiveness of the cut off and to provide access for remedial treatment.

Right abutment treatment consisted of a combination of:

- A mined concrete diaphragm wall providing positive cut off,
- Grouting galleries, at a spacing of 14 m vertically, driven into the abutment 2.0 to 3.5 km,
- A pile diaphragm wall in zones of major karst below the lowest gallery where open shafts and trenches were not practical because of difficulty in dewatering,
- Holes, 165 mm diameter, drilled into minor karst on 330 mm centers and backfilled with tremie concrete, and
- Deep curtain grouting, locally to a depth of 250 m below the lowest gallery.

For the shell foundation, overburden excavation in the upstream one quarter of the base width was approximately to rock (Cooke, 2001). Under the remainder of the shell foundation, the
weathered decalcified sandstone and the limestone blocks floating in a clay matrix were left in place.

In spite of the extensive foundation treatment, several leakage incidents occurred since reservoir filling in 1985. These incidents are described in Chapter 11, Performance.

### 3.3 Embankment Foundation Treatment

The abutments downstream of the plinth and upstream of the dam axis should be stripped of all surface deposits to expose the high points of *in situ* rock. Any surface material remaining between rock points will not adversely affect embankment settlement after rockfill placement. An exception to this is the case of the hard firm, essentially incompressible till, found in Canada and other far north countries.

In the riverbed, deposits may be allowed to remain except within a distance equal to 0.3-0.5 the head from the plinth. A more rigorous analysis is required, if the deposits are subject to loss of strength during the maximum design earthquake. In this case, an evaluation of the strength loss and an estimate of the subsequent decrease in the factor of safety will provide insight concerning the amount of alluvial deposits that will require removal.

Erodible foundation material left in place may need to be protected with filter material in order to prevent the fines being washed into the rockfill, especially within 0.3-0.5 of the head from the plinth. The material left in place should not be so weak as to require local flattening of the embankment slopes to assure stability. For the material left in place upstream of the dam axis, which will be heavily loaded, it is required that its modulus be similar to the expected modulus of the rockfill, to avoid excessive face movements or uneven support. For a distance of 0.5 H with 10 m minimum downstream of the plinth, it is prudent to trim overhangs and vertical faces. The ANCOLD Guideline on CFRD, 1991, suggests trimming of overhangs higher than 2 m to a batter of about 0.5H:1V. In general, the requirements of embankment foundation treatment for the CFRD are less rigorous than for the ECRD.

### 3.4 Consolidation and Curtain Grouting

**Traditional Methods**

Grouting is carried out through the dowelled plinth acting as a grout cap, and outside the embankment body. Rigorous grouting standards must be met because of the very high hydraulic gradients (18 or more) that develop across the plinth.

The design of the grout curtain is outside the scope of the Bulletin, but, in general, its extent should be decided only after consideration of the hydraulic head, the details of the site geology, the potential for leakage and piping and their consequence. Normally, three rows of short consolidation holes are used, and the central row is extended to form a grout curtain. For the CFRD the consolidation grouting is of special importance because of the relatively short seepage path through the rock directly under the plinth. The holes are oriented to intersect the major joint systems as revealed by the geologic mapping and, where necessary, additional holes are drilled.
and grouted to intersect particular features observed as excavation for the plinth foundation progresses. For highly fractured rock foundations, it may be necessary to construct three rows of deep curtain grouting, plus the consolidation grouting.

Final depth of the grout curtain is defined by the specific geological conditions encountered, but it is often specified to be within the range of 1/2 to 2/3 of the reservoir head at the location of the curtain. Grout pressure is normally 25 to 40 kPa per meter of depth, measured at the packer. These pressures are equal to 1.0 to 1.5 times the overburden pressure, calculated as the unit weight of rock times the vertical depth to packer setting.

The maximum spacing of consolidation and grout holes is normally set at about 3 m. This allows detail exploration and treatment of the foundation and provides assurance that all open joints and cracks that require treatment are discovered and adequately treated. The split-spacing method is normally used to locate additional holes. The criteria to provide additional split-spaced holes varies according to the site conditions but for the CFRD is commonly a grout take larger than 50 kg of cement per linear meter (1/3 to 1/2 of a 94-pound sack of cement per linear foot) of grout hole. At some projects, a lower grout take, 35 kg/m, is used at the critical shallow depths below the plinth, and a higher grout take, 70 kg/m, at the deeper, less critical foundation depths.

Stable mixes, that is, a water-cement mix with less than 5% sedimentation, are presently used. To achieve this requirement, mixes with a w:c ratio by volume larger than 2:1 (1.3:1 by weight) should not be used. Pre-hydrated bentonite rather than dry bentonite, in quantities between 1% to 2% of the cement weight, should be used to reduce sedimentation of the cement grains. Alternatively, super-plasticizers that limit sedimentation may be used.

Some specifications have recently required on several dams that no embankment shall be in place within several meters of any grouting operation. This limitation is believed to be unnecessary by many designers since any grout leak into the embankment will do no harm. Unseen surface leaks may cause some waste of grout, but a satisfactory grout curtain is obtained inasmuch as the grouting criteria are not changed as a result of surface grout leaks. On the contrary, this limitation adversely affects the construction schedule and the final result is the placing of Zone 2 against a more segregated Zone 3 material, at the interface of the zones.

**GIN Method of Grouting**

Deere and Lombardi introduced a grouting procedure defined by the “Grouting Intensity Number” (GIN). The main features of this method include:

- A single, stable grout mix for the entire grouting process (water:cement ratio by weight of 0.67 to 0.8:1) with super-plasticizer to increase penetrability;
- A steady low-to-medium rate of grout pumping which, over time, leads to a gradually increasing pressure as the grout penetrates further into the rock fractures;
- The monitoring of pressure, flow rate, volume injected, and penetrability versus time in real time using computers and graphic presentations; and
The termination of grouting when the grouting path on the displayed diagram of pressure versus total volume injected per meter of grouted interval intersects one of the following curves:

- Limiting volume per meter of grouted interval,
- Limiting pressure,
- Or limiting grouting intensity, as given by the selected GIN hyperbolic curve, a curve of constant pressure times volume per meter.

In practice, the grouting process using the GIN technique involves steady pumping of the grout at a low to medium rate, with a slow building up of pressure as the grout penetrates further into the rock mass. Grouting of any stage is stopped when:

- The volume injected attains a specified limiting value for a grouting interval,
- The grouting pressure arrives at a previously selected limiting value, or
- A given intensity of grouting has been attained as judged by the selected limiting value of the product of the pressure times the grout volume.

Specific limiting envelopes of pressure, volume and the product of pressure and volume are developed depending on the type of rock being grouted and on the depth at which grout is being injected. The criteria used at Mohale Dam, Lesotho, in competent basalt with tight contacts between flows are summarized in Table 3-1:

Tests were performed to investigate whether an increase to 2.5 MPa pressure could be utilized below a depth of 20 meters. This pressure exceeds overburden pressure by a factor of about four times (overburden pressure at 20 m is about 0.6 MPa). The tests were conducted in eight curtain grout holes located in the valley floor in 5-meter stages from 20 m depth to 63 m depth. A total of 72 stages were tested with pressures at the packer up to 2.5 MPa. Plots of the individual GIN curves indicated the following:

- Forty five stages, a substantial majority, showed no increase in grout take when pressures were increased to 2.5 MPa. The data indicated that, in many instances, grout refusal in tight rock occurred at pressures ranging from 0.5 to 1.5 MPa.
- Hydraulic opening of joints, possibly on the contacts between flows, as observed by a sudden increase in grout take, occurred in four stages at pressures exceeding 2.0 MPa. Maximum reservoir pressure at a depth of 20 meters below the valley floor is about 1.6 MPa.
- Large grout takes at a pressure of 0.8 MPa were observed in two stages that crossed a shear zone. Features within the foundation such as shears that require attention were successfully treated at a pressure substantially less than 2.0 or 2.5 MPa. Subsequent re-drilling and grouting of these stages indicated low grout takes at 2.0 MPa pressure and no increase in grout take when the pressure was increased to 2.5 MPa.
Table 3-1

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Base Grout Mix by weight w:c *</th>
<th>Max. Pressure in bars (1.0 bar = 100 kPa)</th>
<th>Grout Intensity Number Pressure in Bars times grout volume in l/m*</th>
<th>Grout take Limit l/m*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Valley Floor</td>
<td>Abutment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-5</td>
<td>1:1.5</td>
<td>5</td>
<td>5</td>
<td>500</td>
</tr>
<tr>
<td>5-10</td>
<td>1:1.5</td>
<td>10</td>
<td>7.5</td>
<td>1000</td>
</tr>
<tr>
<td>10-15</td>
<td>1:1.5</td>
<td>20</td>
<td>10</td>
<td>1500</td>
</tr>
<tr>
<td>15-20</td>
<td>1:1.5</td>
<td>20</td>
<td>20</td>
<td>1500</td>
</tr>
<tr>
<td>20-30</td>
<td>1:1.5</td>
<td>20</td>
<td>20</td>
<td>1500</td>
</tr>
<tr>
<td>30-40</td>
<td>1:1.5</td>
<td>20</td>
<td>20</td>
<td>1500</td>
</tr>
<tr>
<td>&gt;40</td>
<td>1:1.5</td>
<td>20</td>
<td>20</td>
<td>1500</td>
</tr>
</tbody>
</table>

* w = water, c = cement plus fly ash. Take means liters of the base grout mix per meter of hole.

- Except for shears and lineaments, the basalt bedrock is extremely tight. Average grout take in the test stages (neglecting the two large takes in the shear zone) averaged 8 kg cement per meter at 2.0 MPa pressure, and 11 kg/m at 25 MPa. These low grout takes are indicative of the low permeability of the rock mass prior to grouting. In these conditions, the grout curtain is exploratory; features that require treatment are discovered by drilling and attempting to grout closely spaced holes.

The test results indicated that there was no benefit in increasing the maximum grout pressure below 20 m from 2.0 to 2.5 MPa. Pressures that greatly exceed the overburden stress can cause hydraulic jacking of pre-existing joints, which, in some instances, can result in foundation damage. High pressure is not required to discover and treat geologic features that require attention. It is noted that the foundations of many successful high head projects, world wide, have been grouted at pressures that approximate the overburden pressure or exceed the overburden pressure by a factor of no more than 1.5.

Grouting the shear zone in the left abutment was completed at considerably lower pressure as summarized in Table 3-2:
Table 3-2

Water Pressure And Grouting Pressure In Bladed Basalts And Lineaments

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Pressures (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Water Pressure Tests</td>
</tr>
<tr>
<td>0 - 5</td>
<td>200</td>
</tr>
<tr>
<td>5 - 10</td>
<td>325</td>
</tr>
<tr>
<td>10 - 15</td>
<td>450</td>
</tr>
<tr>
<td>15 - 20</td>
<td>575</td>
</tr>
<tr>
<td>20 - 30</td>
<td>700</td>
</tr>
<tr>
<td>&gt; 30</td>
<td>1,200</td>
</tr>
</tbody>
</table>

* Thicker mix applies only for the lineament.

A deep grout cut-off was constructed as part of the 140 m high CFRD in the high Andes of Peru, in sedimentary rocks at Antamina Project. GIN method was used and observations indicate that it has some limitations (Carter et al, 2003).

In this project it became clear that GIN procedures were not achieving the desired grouting control in sedimentary rock with karst zones included. Four detrimental factors were identified:

- Hydrojacking resulting from excessively high injection pressures.
- Dilation following hydrojacking shown by non-reducing curtain permeability and the lack of curtain closure.
- High take grout stages terminated at the specified volume limit while still accepting grout at high flow rates, raising concerns that important features were not being treated.
- No relationship between the GIN criterion and the total volume injected, with GIN providing no control for adequate treatment of the fracture system.

Following detailed analysis of the ground response to grout, grouting procedures were revised to blend the best of the GIN approach with the key components of the Australian Method. (Ritchie et al, 2003).

3-16
3.5 References


Castro, J., Li Liu X., Macedo G., Caracoles dam – Analysis of the behavior of the combined plinth – cutoff wall”, Proceedings, Second Symposium on Concrete Face Rockfill Dams, Brazilian Committee on Dams, Florianopolis, Brazil, October, 1999.


Chapter 4

PLINTH

The plinth or toe slab connects the foundation with the face slab. The plinth design, dimensions, stability, construction and foundation treatment are most important. Dimensions of the plinth have been selected based on precedent and generally vary with reservoir head and with foundation conditions. Excavation below the plinth as might be caused by poor foundation conditions or the requirements for access to the dam during construction often result in substantial quantities of backfill concrete below the plinth. The resulting plinth and backfill concrete must be evaluated as an integrated structure for overturning and sliding stability.

4.1 Dimensions of the Plinth

For competent, groutable, non-erodible rock, plinth widths have varied from 3 to 10 m or more, such that the hydraulic gradient through the foundation below the slab is on the order of 15 to 20. Occasionally, a hydraulic gradient through the foundation as high as 40 was adopted. The width is changed in several steps and is not tapered, mainly for construction convenience. For moderately to slightly weathered rock, the width of plinth has been increased, such that a maximum hydraulic gradient of 10 is achieved. The minimum width has been usually set at 3 m, although in Macagua dam (Prusza, et al) it was decided to limit the width of the plinth to 2 m only, because of the low height (22 m) and the massive and strong nature of the foundation granite.

The minimum design thickness of the plinth is usually on the order of 0.3 to 0.4 m with thickness varying with reservoir head, H, in accordance with the following:

\[
\text{Slab Thickness, } T, \text{ in } m = 0.3 + 0.003 H
\]

For construction convenience, a constant thickness is often specified.

Dimensions for Poor Rock Conditions (see also Chapter 3, Foundation Treatment)

The foundation rock conditions and specific erodible features within the foundation must be given special attention. Additional sealing upstream of the plinth and placement of a filter cover over the erodible feature downstream of the plinth are common treatment methods. Shotcrete or concrete with a filter cover has been used at several projects to treat specific features or to provide general treatment of an erodible foundation.

Sealing a prominent potentially-erodible shear zone upstream and providing a filter cover downstream below the rockfill of the dam were adopted at Mohale Dam in Lesotho in addition to increasing the concrete treatment below the plinth. The concrete “socle” over the shear zone increased the effective width of the plinth to about 50 m, thus reducing the hydraulic gradient to about 1.5.
The 110-m high Reece Dam on the Pieman River in Tasmania is founded on a complexly folded and faulted Precambrian sequence of schist, phyllite and laminated quartzite (Li, 1991). These rocks are weathered to depths of 30 to 50 m. The left abutment presented pronounced relaxation with the foliation. Joint and fault planes were open up to 150 mm and infilled with potentially erodible silty clay and clayey silt. The solution adopted at Reece Dam included

- Increasing the plinth width to reduce the hydraulic gradient to about 10 within the foundation beneath the plinth, and
- Constructing a shotcrete blanket, 150 mm thick, reinforced with 8 mm wires on a 100 mm grid, for a distance downstream from the plinth equal to about one-half the reservoir head. A “Z” profile copper waterstop connects the shotcrete blanket with the plinth. This treatment forces a long seepage path through the foundation and increases the effective width of the plinth.

The criteria for maximum acceptable hydraulic gradient verses foundation conditions for Salvajina dam (ICOLD, Hacelas), where the foundation was quite variable, are summarized in Table 4-1. Also presented are the actual constructed widths of plinth and the as-constructed hydraulic gradients.

**Table 4-1**

*Foundation Conditions and Width of Plinth-Salvajina Dam*

<table>
<thead>
<tr>
<th>Foundation Conditions</th>
<th>Maximum Acceptable Hydraulic Gradient</th>
<th>As Constructed Hydraulic Gradient</th>
<th>Constructed Width of Plinth, meters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hard groutable rock</td>
<td>18</td>
<td>--</td>
<td>4 to 8</td>
</tr>
<tr>
<td>Competent rock</td>
<td>18</td>
<td>17.5</td>
<td>6 to 8</td>
</tr>
<tr>
<td>Intensely fractured rock</td>
<td>9</td>
<td>6.2</td>
<td>15 to 23</td>
</tr>
<tr>
<td>Intensely weathered rock-sedimentary</td>
<td>6</td>
<td>3.1</td>
<td>15 to 18</td>
</tr>
<tr>
<td>Intensely weathered rock-residual soil</td>
<td>6</td>
<td>1.3</td>
<td>13 to 14</td>
</tr>
</tbody>
</table>

Plinth width criteria presented by Sierra, 1989, and repeated by Machado, et al, 1993 are presented in Table 4-2. Foundation conditions are divided into four categories from highly competent rock to completely decomposed residual soil. Plinth widths vary from 1/18 times the reservoir head over the plinth for the best foundation conditions to 1/3 for the worst foundation conditions.
Table 4-2
Foundation Criteria for Plinth Width Selection

<table>
<thead>
<tr>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Non-erodible</td>
<td>1/18</td>
<td>&gt;70</td>
<td>I to II</td>
<td>1 to 2</td>
<td>&lt;1</td>
<td>1</td>
</tr>
<tr>
<td>II</td>
<td>Slightly erodible</td>
<td>1/12</td>
<td>50-70</td>
<td>II to III</td>
<td>2 to 3</td>
<td>1 to 2</td>
<td>2</td>
</tr>
<tr>
<td>III</td>
<td>Erodible</td>
<td>1/6</td>
<td>30-50</td>
<td>III to IV</td>
<td>3 to 5</td>
<td>2 to 4</td>
<td>3</td>
</tr>
<tr>
<td>IV</td>
<td>Highly erodible</td>
<td>1/3</td>
<td>0-30</td>
<td>IV to VI</td>
<td>5 to 6</td>
<td>&gt;4</td>
<td>4</td>
</tr>
</tbody>
</table>

Where:

A = Foundation Type
B = Foundation Class
C = Minimum Ratio: Plinth Width/Depth of Water, full reservoir
D = Rock Quality Designation, RQD, in %
E = Weathering Degree: I equals sound rock; VI equals residual soil
F = Consistency Degree: 1 equals very hard rock; 6 equals friable rock
G = Weathered Macro Discontinuities per 10 m
H = Excavation Classes:
   1 = requires blasting
   2 = requires heavy rippers; some blasting
   3 = can be excavated with light rippers
   4 = can be excavated with dozer blade

Alternative Concepts for Achieving Design Width

If treatment of special foundation conditions is restricted to increasing the width of the plinth, substantial additional excavation will result. Simple geometric analysis will quickly demonstrate that doubling the plinth width will also greatly increase the height and volume of the excavation by a factor of two or more. Increasing the height of the cut may require flatter slopes to assure stability, which will further increase the height and volume of the excavation.

A doweled plinth of constant width was used at Babagon Dam in Malaysia resulting in a hydraulic gradient of 15 to 20. The plinth width was effectively increased and the hydraulic gradient decreased to a desirable value by providing a 6 m wide downstream concrete slab. The thinly interbedded sediments (some friable) that crossed the plinth demanded conservative treatment and a longer seepage path.

A doweled reinforced plinth, 4 or 5 m wide provides sufficient space to construct a three-row grout curtain. Downstream, the plinth can be extended beneath the body of the dam for a distance considered adequate to treat any special foundation condition. The downstream or interior plinth extension should be reinforced and connected to the upstream or exterior plinth with a waterstop. Placing a filter cover of material similar in gradation to the 2A zone (see Chapter 8) adjacent to the perimeter joint will preclude migration of silt-sized particles into the body of the dam, even if the downstream plinth were to crack.
This concept was first suggested by J. B. Cooke (Cooke, 1999) to reduce rock excavation for plinths along steep abutments in high dams. The concept can also be applied to advantage in flatter topographic conditions. The external plinth width is defined by the conditions for a practical grouting platform, while the internal slab width supplements the requirements for the allowable hydraulic gradient through the foundation. Specifications for groutable rock quality remain the same under the entire width of the plinth, both external and internal. The result is greater flexibility for excavation geometry and the potential reduction in the volume of excavation in steep valleys, as illustrated in Figure 4-1.

![Figure 4-1, Typical Internal Plinth Cross Section (from Marulanda and Pinto, 2000)](image)

In addition to Babagon, this concept has been applied with success in recent dams such as Corrales Dam in Chile (San Martin, 1999), Ita Dam in Brasil (Sobrinho, et al, 1999), and Pescador Dam in Colombia. Corrales Dam is partially founded on weathered granite, a low cohesion material that could allow particle migration if concentrated flow paths under the plinth were to occur. Within the left abutment, a 6 to 8 meter-wide plinth, 200 mm thick, was extended below the rockfill of the dam to prevent erosion and to reduce the volume of excavation.

The external and internal plinth concept was also used at Ita, where the original plinth design width varied between 6.5 m and 4.0 m using the relationship H/20. Important savings resulted by the adoption of a standard plinth width as shown in Figure 4-2 that included a reduction of the excavation volume and up to 0.75 m³/m of concrete.

A slightly different design was adopted for an internal/external plinth for a 17 m high shotcrete faced dam on the Sugarloaf Dam (formerly Winneke), Australia, where the internal and external sections of the plinth were monolithic, (Casinader and Watt, 1985).
A method suggested by Materon, 2002, for evaluating plinth width selection has been applied at several CFRDs around the world. The method adopts the concept of external and internal plinths and applies the Rock Mass Rating (RMR) developed by Bieniawsky in selecting the combined width of the external and internal plinth. A summary of the method follows:

- Select an external plinth width allowing sufficient width to execute the grouting, 4 to 5 m.
- Classify the plinth foundation using the RMR system.
- Determine the Plinth Design Index as follows (note that the index is equal to the hydraulic gradient, H/L, where H is the reservoir head in m, and L is the dimension in m from the upstream edge of the external plinth to the downstream edge of the internal plinth at the contact of the plinth with the foundation).

<table>
<thead>
<tr>
<th>Rock Mass Rating, RMR</th>
<th>Plinth Design Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt;80</td>
<td>20</td>
</tr>
<tr>
<td>60-80</td>
<td>16</td>
</tr>
<tr>
<td>40-60</td>
<td>12</td>
</tr>
<tr>
<td>20-40</td>
<td>6</td>
</tr>
<tr>
<td>&lt;20</td>
<td>2*</td>
</tr>
</tbody>
</table>

For saprolite, and depending on foundation quality, using a diaphragm cutoff wall or excavating to a better foundation are alternatives.

- Determine the maximum reservoir pressure, H, for the plinth sector and calculate the required total plinth width by dividing H by the corresponding Plinth Design Index.
- Calculate the internal plinth width as the difference between the total plinth width and the external plinth width.
In addition to the above, areas that are susceptible to erosion should be treated with shotcrete and filters to a distance of approximately 40% of the hydrostatic head.

4.2 Geometry Downstream of the Plinth

Foundation geometry downstream of the plinth influences the behavior of the perimeter joint. The depth of the fill at the perimeter joint should be 0.6 to 1.0 m as a convenient cushion for the slab; high fills adjacent to the joint are to be avoided. The change in depth of fill should be gradual with distance from the plinth. This is often achieved with the use of a sloped backfill concrete block placed below the plinth. Figure 4-3 illustrates the geometry of a concrete block at Mohale Dam where excavation resulted in a larger-than-normal concrete block to support the plinth. This geometry avoids the high fill adjacent to the perimeter joint and provides a gradual increase in fill thickness with distance from the joint. Similar geometry is also illustrated in Figure 3-4, Chapter 3, for the Sugarloaf Dam in Australia.

The zone 2A filter material, placed immediately adjacent to the perimeter joint, should be well compacted to strict specifications to minimize settlement. In early projects, vibratory roller access adjacent to the perimeter joint was difficult. This often resulted in imperfect compaction at this location. The use of rectangular backhoe-mounted vibratory compactors is commonly specified to assure effective compaction adjacent to the plinth.

In narrow valleys with steep abutments, it is difficult and expensive to provide favorable foundation geometry for the plinth. Excavation is minimized, and the plinth is dowelled to the steep abutment walls. The depths of rockfill below the perimeter joint are large and substantial joint movements cannot be avoided. In narrow canyons, Goliaths for example, the movements at the perimeter joint normal to the face (settlements) were much larger than those parallel to it (opening). Under these conditions, waterstops can be easily torn off.

For high dams, excavation for the external plinth should extend more deeply into the abutment, similar to a road cut. Horizontal contours of the external plinth excavation should be perpendicular to the perimeter joint to provide reasonable access to construct a practical grout cap. Excavation for the internal slab should result in a gradual increase of fill height away from the perimeter joint as much as practical to minimize joint movements. Where necessary, a concrete block is constructed to create this geometry and to support the external plinth. Sliding and overturning stability of the plinth and concrete block must be satisfied as described below.

4.3 Geometric Layout of the Plinth

The following description of the geometric layout of the plinth is taken from Marulanda and Pinto, 2000.

The most common and practical design is to have the plinth laid out with horizontal contours normal to the plinth alignment. For horizontal plinths aligned parallel to the dam axis, such as those located at the maximum section on the valley floor, the basic geometry is clearly defined by the vertical cross section normal to the plinth alignment, Figure 4-4.
As the rockfill settlement under water load is essentially normal to the upstream face, plane AB is situated at a right angle to the face slope. For sloping plinths at the abutments of the dam, the
same condition holds. Graphical representation becomes more difficult as the plinth alignment changes both in plan and elevation.

Figure 4-5, a plan view of the plinth alignment, presents the development of a practical relationship between the alignment angle "θ" and the plinth slope "n" (n horizontal:1 vertical). If El₁ and El₂ are the elevations of reference points Y₁ and Y₂, the following equations hold:

\[
\tan \theta = \frac{(E_{l1} - E_{l2})m}{L_{1-2}} \\
\sin \theta = \frac{m}{n}
\]

where "m" is the face slope, most commonly varying between 1.3 and 1.5, (1.3 to 1.5 horizontal: 1 vertical).

The cross section of a sloping plinth taken normal to the plinth alignment is shown in Figure 4-6. If the plinth is designed as a constant thickness slab, the majority of the cross section is essentially equal to that of the horizontal plinth. The plane AB remains normal to the dam face. The angle “α” varies in accordance with the alignment of the plinth.

Designers have found various ways, often quite involved, to define the plinth geometry. A recent simplification of the procedures for plinth layout has been proposed by J. B. Cooke (Cooke, 1999) and is illustrated in Figure 4-4. Point "X₁" is located on the foundation surface at the vertical projection of point "Y". The vertical distance, h₀ = Y – X, is established as a constant value, commonly about 0.8 m. This provides sufficient space for Zone 2A filter placement and compaction adjacent to the perimeter joint. For sloping plinths, the rockfill thickness normal to the foundation surface, “h”, is reduced. The height, h, for the inclined plinth can be calculated from the equation:

\[
h = \frac{h_0}{\sqrt{1 + \left(\frac{\sin \theta}{m}\right)^2}}
\]

For h₀ = 0.8 m and m = 1.3, the minimum value of h equals 0.63 m for θ = 90°, which is satisfactory for filter construction and performance.
The procedure for setting the plinth alignment remains unchanged. The reference plane for location of point "X", is now a plane parallel to the face reference plane and lower by the dimension "h0", say 0.8 m. The coordinates of points "X" and "Y" are the same; the plinth in plan is fully defined by the coordinates of the polygonal vertices. The main cross section of the plinth remains constant. Setting plinth face AB normal to the plane of the dam face is also required to establish plinth geometry. The angle "α" can be calculated from the equation:

\[
\tan \alpha = \frac{\sqrt{m^2 + \sin^2 \theta}}{m^2 \left(1 + \frac{1}{m^2}\right)^{\frac{1}{2}} \tan^2 \theta + 1}
\]
4.4 Stability of the Plinth

In optimizing plinth alignment, excavation, and foundation treatment, a compromise is reached between rock excavation and concrete filling of depressions. Abrupt changes in abutment slopes or excavations for access and haul roads may result in construction of large concrete blocks to support the plinth. Access roads constructed across the plinth alignment and large concrete blocks should be avoided wherever possible. Appropriate scheduling and use of ramps within the body of the dam will minimize the need for access road construction across the plinth alignment. Because of environmental considerations, quarry sources are often required to be located within the reservoir. This requirement may result in the unavoidable use of access roads that cross the plinth.

Stability of the large resulting concrete blocks must be checked against sliding and overturning. The stability of the block should be analyzed assuming the uplift pressure under the block is zero at the downstream toe and varies linearly to the full reservoir head at the upstream toe. No support should be assumed from the face slab on the understanding that the perimeter joint may have opened, and, although conservative, no resistance from the passive dowelled anchors is normally assumed in the analysis.

The following text is quoted from ICOLD Bulletin 70, Rockfill Dams with Concrete Facing:

“It is necessary to ensure that the plinth is stable under the forces acting upon it. Dowels anchoring the plinth to the rock foundation are usually designed only to resist a nominal uplift pressure arising from foundation grouting. In the absence of any particular blanket downstream of the plinth to increase the seepage path, as previously described, the uplift pressure can be assumed to be zero at its downstream edge. Passive resistance from the rockfill or from the concrete face slab must be neglected because excessive movement of the plinth would be required to develop it and the face slab pulls away from the plinth when the water load is applied. In the absence of weak seams in the foundation, a sliding factor of 0.6 to 0.7 (Ø = 30°-35°) may be assumed. Under these conditions and a plinth of usual design thickness, it is not difficult to ensure the stability of the plinth.

Excessive height of plinth due to overexcavation or other reasons causes the standard plinth to become unstable if the head exceeds about 30 m. Conventional overturning and sliding analysis must be conducted under this condition, taking also into consideration the least favorable geological discontinuities and the design criteria that the water pressure acting on the toe block is passed straight through to the rock foundation, without calling on the rockfill dam for any stabilizing force. If overbreak is minimal, say less than 0.50 m it is possible to ensure stability by installing sufficient grouted dowels properly oriented and prestressed rock anchors.

If the plinth turns out to be excessively high across a sharp irregularity in the fresh rock surface, such as across a notched road cut excavation in a steep rock abutment or fault zone or other reasons, in addition to the grouted dowels and prestressed anchors, it may be required to provide a buttress at the downstream side of the plinth of such a length that pressure from the embankment fill can be relied upon. Under any abnormal...
circumstances a conventional stability analysis is required and stabilizing measures adopted accordingly.

Another critical problem arises in connection with higher than normal plinths. A high plinth is associated with a greater depth of compressible rockfill in the starter slab area which, in consequence, brings greater vertical offsets than normal along the most important perimeter joint upon reservoir filling. The high plinth is equivalent to a very steep abutment where the water load is transmitted through a higher column of rockfill. Golillas dam experience, Amaya and Marulanda, 1985, is rather significant in this respect. An analysis of the embankment deformations after first impoundment in Golillas has demonstrated that the magnitude of the fill settlements close to the abutments was similar to that in the center of the canyon and, therefore, movements along the perimeter joint in this very steep canyon tend to be mainly vertical.”

**Plinth Stability, Mohale Dam, Lesotho**

The Mohale Dam in Lesotho is founded on a series of hard, competent basalt flows. The contacts between flows are tight; no weak horizontal seams exist. Several shears and zones of doleritic basalt cross the foundation and a construction access road crosses the plinth foundation on the left abutment. These features locally produced foundation irregularities that caused sloping rock surfaces on cross-sections taken perpendicular to the abutment contours, as illustrated in Figure 4-3. Where these conditions occurred, the plinths and underlying backfill concrete were analyzed for sliding and overturning using the following criteria:

- Cohesion between concrete and rock was assumed to be 300 kPa.
- The friction angle between concrete and rock, $\phi$, was assumed to be $45^0$.
- Horizontal rockfill pressure on the sloping face of the concrete backfill was assumed to be equal to 0.25 times the reservoir pressure at the downstream side of the plinth. At Mohale, the rockfill pressure was included to increase stability.
- Vertical pressure on the plinth and on the sloping face of the concrete backfill was assumed to be equal to the reservoir pressure at the downstream side of the plinth.
- Vertical gravity force was assumed to be equal to the sum of the weight plinth, the concrete backfill, and the rockfill overlying the sloping portion of the concrete backfill.
- Uplift at the base of the block was assumed to be linear from the upstream heel to the downstream toe of the backfill concrete.
- For the two-dimensional analysis of sliding on the concrete/rock interface, a factor of safety of 1.5 on the frictional component and a factor of safety of 3 on the cohesion component was assumed. The ratio of resisting forces on the plane of sliding using the above factors of safety to the driving forces on the plane of sliding should be greater than or equal to 1.0.

$$\frac{(N - U) \tan \phi + cL}{15T} + \frac{cL}{3T} \geq 1.0$$
where:

\[ N = \text{Summation of total normal forces on the plane of sliding}, \]
\[ U = \text{Uplift on the plane of sliding}, \]
\[ \phi = \text{Friction angle on the plane of sliding}, \]
\[ c = \text{Cohesion on the plane of sliding}, \]
\[ L = \text{Length of the plane of sliding}, \]
\[ T = \text{Summation of driving forces parallel to the plane of sliding}. \]

• In addition, the sliding analysis was checked using the frictional component only. In this case, the ratio of the resisting forces to the driving forces should be greater than or equal to 1.0.

\[ \frac{(N - U) \tan \phi}{T} \geq 1.0 \]

• Base pressures on the backfill concrete block were calculated and evaluated for the effects of overturning. The anchor bars were assumed to provide no resistance. If tensile stresses occur on the upstream edge of the plinth/concrete backfill, pre-stressed anchors should be considered.

The above criteria were applied to various cross-sections along the plinth alignment. In evaluating the results of the analyses, it was concluded that no pre-stressed anchors would be required at those locations that yield a tensile stress at the upstream heel of 100 kPa or less and where sliding criteria is satisfied. In addition, the percentage of sliding plane in tension did not exceed 10%. At Mohale, the criteria for the sliding analysis were found to be less critical than the criteria for overturning. Pre-stressed anchors were required over a length of about 35 m.

The above criteria are considered to be quite conservative. In steep-sided canyons, it may be more reasonable to evaluate cross-sections taken perpendicular to the axis of the dam rather than perpendicular to the abutment contours and to give some credit to the passive anchor bars.

For details of a similar analysis at Sugarloaf (formerly Winneke) Dam, see Figure 10 of Casinader and Stapledon, 1979.

**Shiroro Dam**

The 110-m tall Shiroro CFRD, constructed in the early 1980s, is founded on granite and diorite. Locally, deeply weathered zones, poorly sorted deposits of boulders and sand, and fault zones resulted in excavations to depths of a few meters to as much as 13 m below the plinth alignment. The resulting combined plinth and underlying concrete block was constructed with a vertical upstream face and a steeply inclined downstream face that, in cross-section at some locations, approximated a square.

Use of the above criteria for overturning indicates large tensile stresses over most of the base. At some locations, the resultant of all forces falls close to the downstream edge of the toe block. The height and shape of the toe block was probably the major cause of the compression failures.
in the face slab at locations above the perimeter joint. Other factors that are believed to have contributed to the face slab cracks include local stress concentrations caused by the interaction between the flexible fill and the rigid foundation, steepness of the abutment slopes, and interaction between adjacent face slabs.

4.5 Reinforcement, Waterstops, and Anchors

Current practice is to provide one layer of reinforcement in the plinth equal to 0.3% each way. The reinforcement is located 150 mm clear distance from the top surface. The purpose of the reinforcing is to reduce cracks to inconsequential hairline widths. Local steel reinforcement is provided to prevent spalling at the perimeter waterstop prior to reservoir filling. Subsequent to reservoir filling, the perimeter joint opens and offsets and the spalling load disappears. Additionally, with no lower layer of reinforcement, the rock and the concrete are more compatible in the event of slight movement of the foundation upon reservoir filling.

Current practice is to construct the plinth of any length as determined by topography and construction convenience. Reinforcement is continuous through the construction joints; waterstops are not provided at the construction joints.

The anchor bars are arbitrarily designed; no specific analysis is performed to select anchor spacing and size. The anchors assure good contact with the foundation. Three rows of 26 mm diameter bars at 2 m centers are commonly specified.

4.6 References


San Martin, Luis, “Plinto en Granito Descompuesto y su Materializacion en el Embalse Corrales”, *II Symposium on CFRD dams*, Bracold-Engevix-Copel, Florianopolis, Brazil, October, 1999.


Chapter 5

PERIMETER JOINTS AND WATERSTOPS

5.1 Introduction

The perimeter joint connects the concrete face slab and the plinth of the CFRD to complete the upstream water barrier of the dam. The main function of the perimeter joint is to maintain a watertight seal against full reservoir load while allowing for anticipated movements between the plinth and face slabs. The plinth is anchored to rock and fixed in place. The face slab rests on the rockfill body of the dam, and will move and deform as the rockfill it rests on settles beneath it. Figure 5-1 illustrates how the face slab can move relative to the plinth in three different directions: normal to the perimeter joint (opening), normal to the face slab (settlement), and parallel to the perimeter joint (shear). Movement in any of these directions separates the face slab from the plinth.

Figure 5-1, CFRD Perimeter Joint Movements (from Pinto and Mori, 1988)

Table 5-1 below summarizes measured perimeter joint movements for several modern CFRDs of compacted rockfill. Maximum total perimeter joint movements in the plane of the joint, calculated as the square root of the sum of the squares of the movements normal to the joint and normal to the face, are plotted versus dam height in Figure 5-2. As would be expected, there is a general trend of increasing joint movement with increasing dam height; however, there is a large amount of variability and scatter. The large variability in perimeter joint movements for higher CFRDs relates to variability in rockfill materials, placement, and treatment in the vicinity of the perimeter joint. This aspect is treated in greater detail in Chapter 8, Fill Materials. From examination of Figure 5-2, it can be concluded that for well-constructed CFRDs, total perimeter joint movement can be expected to be under about 30 millimeters for dams less than about 100
meters in height. For dams over 100 meters in height, total perimeter joint movements may be as high as 100 millimeters or greater.

### Table 5-1

**Perimeter Joint Movement**

<table>
<thead>
<tr>
<th>Dam</th>
<th>Country</th>
<th>Year completed</th>
<th>Height, m</th>
<th>Rock type</th>
<th>Perimeter Joint Movement, mm</th>
<th>Data Source**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aguamilpa</td>
<td>Mexico</td>
<td>1993</td>
<td>187</td>
<td>Gravel</td>
<td>19  16  5  2</td>
<td></td>
</tr>
<tr>
<td>Tianshengqiao</td>
<td>China</td>
<td>1999</td>
<td>178</td>
<td>Limestone and mudstone</td>
<td>16  23  7  6</td>
<td></td>
</tr>
<tr>
<td>Foz do Areia</td>
<td>Brazil</td>
<td>1980</td>
<td>160</td>
<td>Basalt</td>
<td>23  55  25  1</td>
<td></td>
</tr>
<tr>
<td>Salvajina</td>
<td>Colombia</td>
<td>1984</td>
<td>148</td>
<td>Gravel</td>
<td>9   19  15  1</td>
<td></td>
</tr>
<tr>
<td>Alto Anchicaya</td>
<td>Colombia</td>
<td>1974</td>
<td>140</td>
<td>Hornfels-Diorite</td>
<td>125 106 15 1</td>
<td></td>
</tr>
<tr>
<td>Xingo</td>
<td>Brazil</td>
<td>1994</td>
<td>140</td>
<td>Granite</td>
<td>30  34  --  2</td>
<td></td>
</tr>
<tr>
<td>Golilllas</td>
<td>Colombia</td>
<td>1984</td>
<td>130</td>
<td>Gravel</td>
<td>100 36  --  1</td>
<td></td>
</tr>
<tr>
<td>Khao Laem</td>
<td>Thailand</td>
<td>1984</td>
<td>130</td>
<td>Limestone</td>
<td>5?  8?  --  1</td>
<td></td>
</tr>
<tr>
<td>Cirata</td>
<td>Indonesia</td>
<td>1988</td>
<td>126</td>
<td>Breccia-Andesite</td>
<td>10  5  8  7</td>
<td></td>
</tr>
<tr>
<td>Shiroro</td>
<td>Nigeria</td>
<td>1984</td>
<td>125</td>
<td>Granite</td>
<td>30 &gt;50 21 1</td>
<td></td>
</tr>
<tr>
<td>Reece</td>
<td>Australia</td>
<td>1986</td>
<td>122</td>
<td>Dolerite</td>
<td>7   70  --  1</td>
<td></td>
</tr>
<tr>
<td>Cethana</td>
<td>Australia</td>
<td>1971</td>
<td>110</td>
<td>Quartzite</td>
<td>11  --  7  1</td>
<td></td>
</tr>
<tr>
<td>Kotmale</td>
<td>Sri Lanka</td>
<td>1984</td>
<td>97</td>
<td>Charnokite</td>
<td>2   20  5  1</td>
<td></td>
</tr>
<tr>
<td>Xibeikou</td>
<td>China</td>
<td>1991</td>
<td>95</td>
<td>Dolomite</td>
<td>14  25  5  4</td>
<td></td>
</tr>
<tr>
<td>Murchison</td>
<td>Australia</td>
<td>1982</td>
<td>89</td>
<td>Rhyolite</td>
<td>12  10  7  1</td>
<td></td>
</tr>
<tr>
<td>Sugarloaf</td>
<td>Australia</td>
<td>1982</td>
<td>85</td>
<td>Sandstone</td>
<td>9   19  24  1</td>
<td></td>
</tr>
<tr>
<td>Crotty</td>
<td>Australia</td>
<td>1991</td>
<td>83</td>
<td>Gravel</td>
<td>2   27  --  9</td>
<td></td>
</tr>
<tr>
<td>Mackintosh</td>
<td>Australia</td>
<td>1981</td>
<td>75</td>
<td>Graywacke</td>
<td>5   20  3  1</td>
<td></td>
</tr>
<tr>
<td>Bastyan</td>
<td>Australia</td>
<td>1983</td>
<td>75</td>
<td>Graywacke</td>
<td>5   21  --  1</td>
<td></td>
</tr>
<tr>
<td>Chengbing</td>
<td>China</td>
<td>1989</td>
<td>75</td>
<td>Lava tuff</td>
<td>13  28  20  5</td>
<td></td>
</tr>
<tr>
<td>Pichi-Picun-Leufu</td>
<td>Argentina</td>
<td>1999</td>
<td>40</td>
<td>Gravel</td>
<td>2   12  1  3</td>
<td></td>
</tr>
<tr>
<td>Serpentine</td>
<td>Australia</td>
<td>1972</td>
<td>39</td>
<td>Quartzite</td>
<td>1.8  5.3  --  1.8</td>
<td></td>
</tr>
<tr>
<td>Palooona</td>
<td>Australia</td>
<td>1971</td>
<td>38</td>
<td>Argillaceous Chert</td>
<td>0.5  5.5  --  1.8</td>
<td></td>
</tr>
<tr>
<td>Tullabardine</td>
<td>Australia</td>
<td>1982</td>
<td>26</td>
<td>Greywacke</td>
<td>--  0.7  0.3  1.8</td>
<td></td>
</tr>
</tbody>
</table>

* O = Opening, normal to joint, S = Settlement, normal to concrete face, T = Shear, parallel to joint

** 1  ICOLD, Bulletin 70, 1989
2  Pinto and Marquez, 1998
3  Marques, Machado, et al, 1999
4  Peng, 2000
5  Wu, Hongyi, 2000
6  Wu, G.Y. et al, 2000
7  Kashiwayanagi et al, 2000
8  Fitzpatrick, et al, 1985
9  Knoop, B.P. 2002 (personal communication)
Due to the requirement to maintain a watertight seal while accommodating these movements, and the high pressure heads it can be subjected to under full reservoir load, the perimeter joint has received a great deal of attention by designers. This attention has focused primarily on design of the water-retaining barriers that form the perimeter joint. Design concepts used for CFRD perimeter joints are typically incorporated into the design of the vertical contraction joints of the CFRD face slabs as well.

![Perimeter Joint Movement vs. Dam Height](image)

**Figure 5-2, Perimeter Joint Movements vs. Dam Height**

- **(D)** Dam height (meters)
- **(M)** Movement (millimeters)

### 5.2 Perimeter Joint Designs

The basic design concepts used for modern CFRD perimeter joints are relatively similar from one project to the next; however, the details that are developed based on these concepts can vary substantially.

**Evolution of the Perimeter Joint**

Pinto and Mori (1988) summarized the evolution of the perimeter joint in modern CFRDs of compacted rockfill, starting with the 110-meter high Cethana Dam in Australia. Cethana Dam, completed in 1971, was the first compacted CFRD over 100 meters in height. Two water barriers were used to provide a watertight but flexible connection between the plinth and face slab under full reservoir load. A W-shaped copper waterstop was used at the bottom of the joint between the concrete face slab and plinth. The shape of the waterstop was selected so that it could deform as the perimeter joint opens without tearing or rupturing. A rubber waterstop was placed at the middle of the joint. Measured leakage upon first filling of the reservoir was limited to about 50 liters per second (l/s), and reduced to about 10 l/s within five years.
Due to the small joint displacements and minor leakage measured at Cethana Dam upon its completion, only one rubber waterstop was placed in the middle of the perimeter joint of the 140-meter high Alto Anchicaya Dam in Columbia. Upon its completion in 1974 and subsequent filling of the reservoir, a leakage rate of 1,800 l/s was recorded. Some investigators thought that a lack of sufficient bonding between the concrete slabs and the rubber waterstop allowed water to pass around the waterstop and through the joint, and that this condition may have been a primary cause of the leakage. For others, the main reason for the leakage was the very steep abutment and the erosion that occurred during construction, as there was no protection for the support zone. Deep erosion occurred along the contact with the abutments. The backfill material was not properly compacted, had insufficient density and, therefore, was more deformable. To help reduce leakage around the joint, damaged concrete was replaced, and a reservoir of mastic joint sealing material was placed in the open joint. The mastic was covered by a wire mesh reinforced sand-asphalt mixture and clay, and held in place by steel plates. Leakage was dramatically reduced upon refilling of the reservoir, and the success of the repair was attributed to the performance of mastic joint sealing material, which would fill any opening that formed between the plinth and face slab.

Perimeter joint designs for subsequent high CFRDs incorporated not only the two-barrier waterstop system successfully used for Cethana Dam, but also the mastic reservoir concept used to help successfully seal the perimeter joint of Alto Anchicaya Dam. The perimeter joint for the 160-meter high Foz do Areia Dam in Brazil was the first to use the three-barrier system. The successful performance of the perimeter joint of Foz do Areia has lead to the use of this same three-barrier concept on many high CFRDs. The perimeter joint of the 148-meter high Salvajina Dam in Columbia, completed in 1984, is shown in Figure 5-3.

Perimeter Joint Design for Low CFRDs

Although the three-barrier concept in use since Foz do Areia has been used for many CFRDs, regardless of height, CFRDs less than 100 meters in height and some that are more than 100 meters tall have successfully used only two barriers in the perimeter joint. The perimeter joints for the 80-meter high Mangrove Creek Dam in Australia and the 63-meter high Boondoma Dam in Australia both utilized bottom waterstops and middle rubber waterstops without the upper water barrier that is commonplace in high CFRDs (Mackenzie and McDonald, 1985 and Rogers, 1985). The perimeter joints for the 22-meter high Macagua Dam in Venezuela, and the 24-meter high CFRD for the Keenleyside Powerplant Project in British Columbia, Canada, both have a bottom waterstop and a reservoir of mastic at the joint surface (Prusza et al., 1985). The middle rubber waterstop was eliminated. The Pescador dam in Colombia (43 m tall) and the Antamina Dam in Peru (135 m tall), also eliminated the middle rubber waterstop. Both dams used a copper waterstop at the bottom and a reservoir of very erodable material on top of the joint. The dams have performed extremely well, leakage at Pescador Dam is less than 3 l/s, and leakage at Antamina Dam is about 1 l/s.
Perimeter Joint Design for Very High CFRDs

Several CFRDs are currently planned that will exceed 200 meters in height, including the proposed 233-meter high Shuibuya Dam in China. These CFRDs can be classified as very high dams, and the high pressures and deformations that the perimeter joints of these dams will be exposed to require close attention. Several numerical studies and model tests have been conducted in China to test the effectiveness of various perimeter joint design concepts under high head conditions. These studies have been conducted primarily in support of design efforts for the 233-meter high Shuibuya Dam proposed for construction in China, and have concentrated on the effectiveness of cohesionless fines, mastic materials, and copper waterstops as water barriers for perimeter joints under high heads. Some of the more important preliminary conclusions reached by the various researchers involved in these studies include the following:

Figure 5-3, Perimeter Joint for Salvajina Dam (from ICOLD, 1989)

1. Copper waterstops with a thickness on the order of 0.8 to 1.2 millimeters are reasonable for use on CFRDs between 100 and 234 meters in height; however, seepage is shown to occur around standard bottom copper waterstops under pressures of approximately 1.5 Mpa. When the base of the copper waterstop in contact with the plinth or face slab concrete is treated with mastic materials, the composite waterstop was shown to be watertight to a pressure of 2.5 Mpa with 10mm of deformation (Jia et al., 2000).
2. For upper water barriers consisting of mastic, the composition of the mastic material is important, particularly at very high heads. In China, mastics have been developed that maintain a watertight seal and are capable of large extensions (greater than about 600%) without breaking at both high and low temperatures under heads exceeding 200 meters (Jia et al., 2000 and Wangxijiong, 2000).

3. Model tests on prototype joints consisting of a copper bottom waterstop treated with mastic, and a membrane covered mastic reservoir, indicate little to no measurable leakage at pressures up to 2.5 Mpa and joint openings up to 100 millimeters (Tan, 2000).

4. Use of cohesionless fines in lieu of mastic as an upper water barrier is feasible as an alternative means for sealing perimeter joints of CFRDs greater than 200 meters in height (Ding et al., 2000).

5.3 Lower Water Barriers

Nearly all of the CFRDs constructed after Alto Anchicaya Dam have utilized a bottom waterstop of copper, stainless steel, or sometimes PVC as the lower water barrier in the perimeter joint. The basic shape of the bottom waterstop is shown in Figure 5-4, and it is designed to safely accommodate any deformation that may take place between the plinth and the face slab without rupturing. The shape is generally referred to as a W-shape or F-shape depending on whether both vertical tabs are present.

![Figure 5-4, Typical Bottom Waterstop Shape](image)

**Figure 5-4, Typical Bottom Waterstop Shape**

**Shapes and Sizes**

As shown in Figure 5-4, the basic shape of the bottom waterstop is designed with a high center section, or rib, to allow the waterstop to deform without tearing as movement of the perimeter joint occurs. The tabs on one or both sides of the waterstop are embedded into the concrete of the face slab and plinth, and serve to increase the seepage path around the waterstop. On some CFRDs the vertical tab is omitted or shortened in the plinth to permit better reinforcement placement, concrete placement and concrete consolidation.
Sizes of bottom waterstops vary somewhat from project to project, and the size of a waterstop selected for any one project should be based on anticipated joint movements and concrete and reinforcement placement requirements. The height of the center rib should be tall enough to allow the waterstop to deform without rupture. The height of the wings at either end of the waterstop should be as high as possible without interfering with reinforcement in the face slab or plinth. The base width of the bottom waterstop should be large enough to allow for proper placement and consolidation of concrete at the perimeter joint. Base widths for most bottom waterstops vary between about 250 and 500 millimeters.

**Metal Waterstops**

For CFRDs above 100 meters in height, either copper or stainless steel waterstops are used for the bottom water barrier at the perimeter joint. The selection of copper or stainless steel is typically based on factors such as ease of handling, water quality, and material costs. Copper is more easily formed and welded than stainless steel. Stainless steel has greater corrosion resistance and less likely to be damaged during construction than copper. Prices for stainless steel or copper will vary greatly depending on the project location and other economic factors. The metal sheets used to fabricate copper or stainless steel waterstops are typically on the order of 0.8 to 1.2 millimeters thick. Thinner sheets can be used on CFRDs of about 100 meters in height or less. Thicker sheets are recommended for higher CFRDs.

Metal waterstops require special attention to prevent damage during construction. A pad of asphalt concrete or mortar is typically constructed under the waterstop to provide an even surface for placement of the waterstop, and to protect the waterstop from puncture or tearing due to sharp aggregates in the concrete face slab supporting zone. Use of asphalt concrete also provides a surface that has some ductility in order to protect the waterstop should joint movement occur. A strip of bituminous felt or PVC is also used to cushion the waterstop on the pad. The center rib is filled with neoprene or foam inserts to prevent the center section from being crushed under pressure from fluid concrete or external water pressure. A wood or steel box is placed over the portion of the waterstop protruding from the plinth prior to placement of the face slab concrete to protect it from damage during placement of adjacent rockfill materials. This point cannot be over-emphasized. All too often, damage to the copper waterstop has occurred because of poor construction practice. When damage occurs, a poor repair or no repair is often the result.

Copper and stainless steel waterstops are typically formed in long, continuous pieces to minimize field splices. At the 145-meter high Mohale Dam in Lesotho, copper waterstops were formed as they were placed on site using a specially designed, continuous feeding and forming machine. Field splices are made by overlapping successive sections, and welding using a high fluidity electrode to ensure full penetration between the overlapping sections (ICOLD 1989). Spark testing is used to check field splices for quality.

**PVC Waterstops**

Bottom waterstops of PVC can offer several advantages over copper or stainless steel waterstops when they are used; however, their use has historically been limited to CFRDs lower than 100
meters in height. Figure 5-5 shows the PVC bottom waterstop used for one of the two CFRDs constructed for the 63-meter high Boondooma Dam in Australia. PVC bottom waterstops have also been used at the Macagua and Caruachi dams in Venezuela and the Keenleyside Powerplant Project in British Columbia.

PVC waterstops are substantially thicker than copper or stainless steel waterstops in order to resist pressures under full reservoir head. PVC waterstop thickness can range from 5 millimeters for low CFRDs to over 12 millimeters for higher CFRDs. The greater thickness can simplify placement of the waterstop during construction, and reduces the risk of damage during construction as well. For the CFRD at the Keenleyside Powerplant Project, a 12-millimeter thick PVC waterstop was used for the bottom water barrier. The thick waterstop allowed for the elimination of both the support pad and the PVC or bituminous cushion strip normally required for thinner PVC waterstops or metal waterstops. The thick PVC section also eliminated the need for a protective cover during placement of the adjacent rockfill. The protruding portion of the waterstop was temporarily bent upwards against the plinth to allow for placement and compaction of the face supporting zone beneath the waterstop at the perimeter joint.

As shown in Figure 5-5, a thin tab is left in place along the bottom of the center section of the waterstop to help keep the center rib open and clear prior to joint movement. This eliminates the inserts that are required for metal waterstops. Sections of PVC waterstops are typically spliced in the field by butt-welding adjacent sections together. No additional welding materials are required for splicing of PVC waterstops.

**Bottom Waterstop during Construction**

Many projects have used wood planking and steel frames to protect the perimeter joint and the protruding waterstops prior to construction of the face slab. (Fig 5-6)
Middle water barriers for perimeter joints consist of flat or dumbbell shaped PVC or Hypalon waterstops such as those used for construction and contraction joints in concrete hydraulic structures. Each end of the waterstop is embedded into the plinth and face slab concrete. Various shapes used as middle waterstops in CFRDs are shown in Figure 5-7. Flat waterstops typically have rows of ribs along each side to provide for better mechanical interlocking with the face slab or plinth concrete. Dumbbell-shaped waterstops have solid core bulbs on either end for the same purpose. Center bulb waterstops include a hollow bulb at the middle of the waterstop. They are considered preferable to other middle waterstop shapes due to their ability to undergo greater deformation before tearing or rupturing. Additional measures are sometimes taken to protect middle waterstops from being cut by sharp concrete edges as the face slab offsets from the plinth. At 148-meter high Salvajina Dam in Columbia, neoprene cylinders were placed at alternate corners to protect the middle waterstop (ICOLD, 1989).
Although middle waterstops of several different shapes have been used successfully on high CFRDs in the past, the value of these waterstops has been questioned. As discussed earlier in this chapter, one of the reasons given for the leakage recorded during the initial filling of Alto Anchicaya was poor performance of the middle PVC waterstop. Cooke and Sherard (1987) question whether middle bulb waterstops may rupture under the high pressures and large joint movements experienced at the perimeter joints of high CFRDs, and also question whether the location of the middle waterstop can impair proper placement and consolidation of concrete at the perimeter joint. Tests conducted to determine the performance of middle PVC waterstops under large joint displacements, reported by Pinto and Mori (1988), indicated that these waterstops tended to rupture after about 25 millimeters of displacement. However, tests on center bulb waterstops reported by Guiduci et al. (2000) showed that they could undergo up to 115 millimeters of displacement without rupturing. Thus, if middle waterstops are used as a water barrier, center bulb waterstop shapes are recommended.

5.5 Upper Water Barriers

For CFRDs over 100 meters in height, the upper water barrier typically receives the most attention by designers. This water barrier is installed after the plinth and lower part of the face slab are constructed, and it provides the greatest opportunity to achieve a reliable and robust water barrier for the perimeter joint. As discussed previously, the first use of an upper water barrier was for the repair of the perimeter joint at Alto Anchicaya Dam. The successful application of mastic joint sealing materials at Alto Anchicaya resulted in the adoption of a reservoir of mastic as the upper water barrier for many future high CFRDs. Other upper water barrier concepts have been developed and successfully incorporated into high CFRDs as well; however, the general concept of providing a reservoir of joint sealing material to fill the gap left as the face slab separates from the plinth remains the same.
Mastic Joint Sealants

The application of mastic as a joint sealant for the upper water barrier of the perimeter joint is common and widespread. The most common detail consists of forming a reservoir of mastic at the top of the perimeter joint by chamfering the top edges of the plinth and face slab. Mastic is then placed into the joint until it forms a mound above the surface of the joint. A flexible and durable covering, anchored at each side to the plinth and the face slab, is placed over the mounded mastic to protect it and so that the water pressure from the dam reservoir can be applied evenly over the surface of the material. As movement of the perimeter joint occurs, the mastic is forced into the joint under the pressure of the reservoir to fill the gap that forms, thus maintaining a seal against leakage through the joint. Figure 5-3 shows the mastic reservoir detail used at Salvajina Dam in Colombia.

Results of tests conducted on the effectiveness of mastic as an upper water barrier indicate that appropriate attention must be paid to the mastic joint detail, beginning with the proper selection of the mastic materials to be used. Mastic is a compound material containing several components that affect its workability and performance as a joint sealing material. The main component of mastic is a bitumen base that increases in fluidity as it is heated, and hardens when cooled. Additives and fillers are used to increase the fluidity of hot mastic, as well as to increase or decrease the deformability of the mastic after it has cooled. Thus, in high temperature environments, a particular type of mastic may be highly deformable and may even flow under its own weight if not carefully placed and secured in the perimeter joint. In some instances mastic placed over perimeter joints has been reinforced with wire mesh to prevent it from flowing down sloping plinths near the abutments or face slab contraction joints. In environments where the temperature may fluctuate widely, and in colder regions, care must be taken to select a mastic composition that maintains its deformable characteristics at temperatures far below freezing. For acceptable performance, mastic materials should be viscous enough to flow into the perimeter joint when movement occurs, but be sufficiently strong and elastic so that it does not break down or separate under high pressures. Mastic can become brittle, crack and separate from the concrete in the perimeter joint, eliminating its effectiveness as a joint sealing material. Golillas, in Colombia, is an example of mastic materials that became brittle and cracked. The reservoir was not filled for about four years after construction because the tunnel to convey water to the city of Bogotá was not finished. Because of exposure to weather over several years and being at an altitude of about 3000 m, most of the mastic material became brittle and did not provide a defense during the initial filling. The poor performance of mastic materials in laboratory tests reported by Pinto and Mori (1988) may have been the result of poor mastic properties above any other factors. Mastics that provide several hundred percent extension under a wide range of temperatures are recommended.

Mastic is placed in the perimeter joint using several different techniques. A common method is to heat mastic in batches to a temperature such that it can be easily formed and spread in layers in the prepared concrete joint. The temperature of the mastic must be high enough so that each successive layer is properly bonded to the previously applied layer when applied in this fashion. Otherwise weak planes or laminations may form in the mastic, eliminating its effectiveness. Pre-formed lengths of mastic, or logs, can also be used, either supplied in a usable form directly from
the mastic supplier or made onsite. Pre-formed logs have some advantages over hot-applied mastic, including ease of placement in the perimeter joint and greater material quality control. Lengths of mastic are placed directly into the prepared joint. Mastic materials mixed in a factory-controlled environment, or at a suitable onsite facility, will be more uniform in composition and of a higher quality than mastics mixed in the field just prior to application. Preformed mastic logs (typically 1 to 1.5 meters long when ordered from the supplier) are joined by heating each end, and then pressing each end together. Preformed mastic logs were used in the perimeter joints of the recently completed CFRD of the Keenleyside Powerplant Project.

Careful cleaning and treatment of the concrete surfaces on which the mastic will be placed is important. Common practice is to carefully wash and clean the surface of the concrete, and pre-treat the surface with a coating of heated mastic prior to placement. Companies that supply mastic for such purposes provide instructions for proper pre-treatment of concrete surfaces. Some designers specify that a small diameter (15 to 16 millimeters) neoprene tube be placed in the groove of the joint before the mastic sealant is applied. The tube serves to prevent the mastic from flowing into the joint until a sufficiently wide gap has formed. Such a detail is not considered necessary; however, as long as enough mastic is provided to completely fill the joint as it opens.

Mastic joint sealants must be properly covered and protected in order to provide a long lasting upper water barrier. Although coverings consisting of PVC membranes, and even conveyor belting material (Morris, 1985), have been successfully used in the past, Hypalon has been the preferred covering material because of its resistance to weathering and ozone attack. Only materials that offer exceptional resistance to ozone deterioration, weathering and stresses from reservoir pressure should be considered if an alternative to Hypalon is desired. The covering should be wide enough to cover the mastic material, and should form a convex upward shape, along with the mastic, when secured in place. The covering will never be in tension when the joint opens and the mastic is forced into the joint. The covering is secured to the concrete of the face slab and plinth either by galvanized rigid steel angles or galvanized flat bars anchored into the concrete at regular intervals. Flat bars are more convenient to use on rough concrete surfaces, but must be anchored at closer intervals (Cooke and Sherard, 1987). Flat bars anchored using concrete expansion anchors at 400 millimeter centers were used for the CFRD of the Keenleyside Powerplant Project.

**Joint Sealing With Cohesionless Fines**

In their 1988 paper, Pinto and Mori proposed an alternative to the mastic joint sealant system. This alternative considers replacing the reservoir of mastic covering the perimeter joint with a zone of fine sand and silt. Figure 5-8 shows their proposed alternative. The key to this alternative is the addition of a fine filter zone to the rockfill placed behind the perimeter joint. As movement of the perimeter joint occurs, the fine sand and silt is washed into the gap that forms but retained by the fine filter zone behind the slab. The fines clogged joint limits leakage through the perimeter joint.
Figure 5-8, Upper Water Barrier Using Cohesionless Fines (from Pinto and Mori, 1988)

The 187-meter high Aguamilpa Dam in Mexico incorporated the cohesionless fines concept as a joint sealing material, utilizing fly ash for cohesionless fines (Gomez, 1999). A reservoir of fly ash was placed on the surface of the perimeter joint, protected by a galvanized steel cover. The upper water barriers for the 178-meter high Tianshengqiao No. 1 Dam and the 124-meter high Heiquan Dam in China also use a zone of fly ash for the cohesionless fines (Jiang and Zhao, 2000). A similar concept was adopted for the 145-meter high Mohale Dam in Lesotho (Gratwick et al., 2000).

The Antamina Dam in Peru and the Pescador Dam in Colombia have used this concept not only for the perimeter joint but also for all expansion joints. The cohesionless fines are placed inside a hypalon cover (membrane) that is securely attached to the concrete. At both dams, the membrane was placed on top of a geotextile and secured with a steel angle anchored to the concrete. This tight connection precluded the possibility of washing the cohesionless fines during the operation of the reservoir.

The key to the success of upper water barriers using cohesionless fines is the proper design and placement of the fine filter zone behind the perimeter joint, and the proper selection of the cohesionless fines material to be used. The gradation of the fine filter zone placed behind the perimeter joint must be such that the cohesionless fines are not washed through the perimeter joint and into the rockfill body of the dam. Additional details for the design and placement of this zone are provided in Chapter 8, Fill Materials. The cohesionless fines used for this upper water barrier concept must contain enough fine particles (silt sizes or finer) to provide a low permeability barrier to flow from the reservoir, but must not have any cohesive properties that could prohibit the erosion of this material into the perimeter joint. Due to its fineness and non-cohesive properties, fly ash is a preferred material for this purpose. However, fine silty sand with non-plastic fines or any other fine, non-cohesive material should be able to perform this function as well.

Regardless of the type of cohesionless fines used, this zone must be adequately protected from the reservoir. An upstream, zoned earthfill buttress, a common feature on many of the higher
CFRDs, serves this purpose up to the top of the buttress. Above the earthfill buttress zone, a membrane cover should be used to protect the cohesionless fine, similar to those used for mastic materials. Additional details of this upper earthfill buttress zone are also provided in Chapter 8, Fill Materials.

Casinader (private correspondence, 2004), reports that at Sugarloaf and Kotmale dams, a relatively small plinth backfill of a few meters of confining fine material was taken up the whole length of the plinth. This was used instead of a larger earthfill buttress at the toe of the dam within the valley section of the dam. Where abutments are steep, this technique is not practical.

**Other Upper Water Barriers**

Although upper water barriers utilizing mastic and cohesionless fines are the most common methods used on CFRDs of all heights, other concepts for upper water barriers have been developed recently. A waterstop-type barrier was developed for the recently completed 125-meter high Ita Dam in Brazil, consisting of a triangular shaped neoprene or EPDM waterstop that can be wedged into the joint between the plinth and the face slab, or the vertical contraction joint between adjacent face slabs. The waterstop is designed so that it can expand as the joint opens. The force of the reservoir water pressure keeps the waterstop seated in the joint. According to test results reported by Mori and Sobrinho (1999), the new waterstop was successfully tested to static heads of up to 260 meters and about 60 millimeters of joint movement. It is noted; however, that the waterstop was used for the upper water barrier at vertical contraction joints only. A mastic reservoir was used for the upper water barrier in the perimeter joint. A similar concept was used for the 125-meter high Machadinho Dam in Brazil (Mauro et al., 1999).

In an attempt to include a rubber waterstop at the perimeter joint, without the complications caused by middle rubber waterstops, designers of the 122-meter high Quinshan Dam in China included an upper waterstop of corrugated rubber that spans the opening between the plinth and the face slab (Jia et al., 2000). The rubber waterstop is bolted on either side of the joint to steel angles embedded in the slab concrete, and is intended to perform the same function as the middle PVC waterstop used in other high CFRDs. This concept has also been considered for the proposed 234-meter high Shuibuya Dam in China (Jia et al., 2000).

**5.6 Additional Perimeter Joint Details**

During the construction of CFRDs, the concrete face slab will deform and move against the plinth as the underlying rockfill settles and deforms. This deformation and movement can cause stress concentrations at the perimeter joint, and may result in spalling of the concrete or damage to the water barriers in the perimeter joint of high CFRDs. To prevent damage to the perimeter joint during construction, compressible wood or asphalt board filler is typically nailed to the butt face of the plinth in order to provide a cushion on which the face slab can rest. The filler is usually about 12 to 20 millimeters in thickness, and only functions during CFRD construction. Plywood, wood planking, or asphalt impregnated press board are commonly used.
5.7 References

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