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Special factors in design of high RCC gravity dams

Part I

G. S. Sarkaria and F. R. Andriolo***

The methods of design and the technology for construction of RCC dams have made significant progress during the last ten years. Objective analyses of the performance of existing RCC dams indicate the need for substantial improvements in some aspects of design, construction techniques and quality control, to ensure that high RCC dams are equal in long-term quality, safety and performance to comparable dams of conventional mass concrete (CMC).

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Based on the construction experience and the available performance data from completed RCC dams, several high (>120 m) and large (>1million m^3) RCC dams are currently under design or are proposed for construction in the near future. The economic advantages of RCC dams over CMC dams, due to rapid rate of concrete placement, shorter period of construction and lower costs, are now well recognised. However, these advantages are sometimes overemphasised, while some significant problems that have occurred at RCC dams are either overlooked or their consequences considered acceptable.

During the definition stage of a project, when the type of dam is selected, the deficiencies of, or incidents that have occurred at, some smaller RCC dams should be carefully evaluated. The consequences of similar mishaps at a high and large dam during its lifetime can be catastrophic. For the purpose of this paper, a high RCC gravity dam is >120 m in maximum height, with a straight axis and a cross-section similar to that of a typical conventionally built concrete gravity dam.

Structural Equivalence criteria

A completed RCC gravity dam should function as a monolithic elastic structure, integrally bonded to its rock foundations, that is, its structural performance should be equivalent to that of a CMC gravity dam with a similar configuration. For the two types of gravity dams to be equal in quality, safety and durability, they should have equivalent margins of safety against cracking, rupture, overstressing, shearing-sliding and leakage through the concrete and construction or layer joints.

The degree of monolithicity and elastic isotropy of a RCC gravity dam depends on several factors. If there are vertical transverse cracks or ungrouted contraction joints, the structure's blocks would act as individual cantilever gravity dams, each independent of its neighbours. If no transverse contraction joints are provided, and no transverse cracks occur, the dam will function as a three-dimensional monolithic plug, transmitting some load in all directions, including longitudinally to the abutments. If the transverse cracks are inclined or curvilinear, the structural behaviour of the dam could be a hybrid between a cantilever gravity and a three-dimensional monolithic dam. If longitudinal cracking occurs, it would affect monolithicity in the transverse direction and cause internal tensile stress concentrations. All cracks alter, to varying degree, the stress field in the elastic mass with tensile stress concentrations occurring at the terminus of each crack within the body of the dam. Other factors which could significantly affect monolithicity are the bond, shear and tensile strengths of the horizontal construction joints.

The current design, construction and performance of completed RCC gravity dams indicates several issues which need to be resolved if the structural performance of a high RCC gravity dam would be equivalent to that of a CMC gravity dam. These issues are:

- Quality of the foundations.
- Elastic monolithicity of construction joints.
- Stability against shearing-sliding.
- Structural cracks
- Transverse contraction joints.
- Longitudinal cracks and joints.
- Quality of RCC.
- Large spillways over an RCC dam.
- Drainage and seepage control.

Quality of the foundations

High concrete gravity dams, whether built of RCC or CMC, require rock foundations, which either in the natural state or after appropriate treatment, have adequate strength to receive the loads imposed upon them by the dam and the reservoir, without undergoing excessive deformations or instability. Invariably, rock foundations for high gravity dams need various types and degree of beneficiation treatment, and in some cases, foundation treatment can be as important an aspect of design and construction as the dam itself.

It is erroneous to assume that, because concrete in an RCC dam is placed in layers and compacted by rollers in a manner similar to earthfill or rockfill in an embankment dam, it would have the "flexibility" of an embankment to adjust to differential settlement or deformations of the foundation without adverse consequences for its stability. The response of an RCC gravity dam to such foundation behaviour and the effects on its stability would be similar to that of a CMC gravity dam. The consequences may comprise unacceptable reduction in reserve strength against shearing-sliding, cracking at the dam-foundation contact, increase in hydraulic uplift pressures, and cracking and overstressing in the dam itself. At the 90m high Upper Stillwater Dam[2], a transverse crack which extended through the entire section of the dam was attributed to movement along a weak layer in the foundation which was either unanticipated or inadequately treated. While provision of a transverse contraction joint in the dam would have "controlled" the cracking, it would not have prevented the other adverse consequences of excessive foundation movement.

For high gravity dams, the weaker foundation features, such as shear zones, faults, contacts filled with gouge or clayey materials, or

The Yellow River used to be called 'China's sorrow' because of its severe and frequent flodding. Work has now started on an ambitious multipurpose project whose main aim is flood control.
Janet Dansie reports.

Taming the Yellow River

Location map of the Xiaolangdi project. Source: Hochtief

The Xiaolangdi project on the middle reaches of the Yellow River, 600km southwest of Beijing and 40km north of Luoyang, will be one of the largest multipurpose schemes in China on completion.

Although it will eventually have a 1800MW power generating capacity, this is not the chief function of the project. One of those will be to protect some 100 million of the local population from flooding. The Xiaolangdi dam has been designed to control a 1000 year flood, in contrast to the current 60 year flood protection afforded by the river's ancient dykes.

Its second vital function will be to control sediment flow downstream, by means of three sediment flushing tunnels. The waters of the Yellow River are the most silt-laden in the world (see p50) and caused serious problems at the reservoir of the Sanmenxia hydroelectric project, built in the late 50s, 130km downstream of Xiaolangdi.

The 154m high rockfill dam will be constructed by a joint venture, led by German contractor Hochtief and including Italy's Impregilo and Italstrade and China's 14th Construction Bureau.

A particular technical feature of the project is the 80m deep concrete cut-off wall, built early last year under the upstream cofferdam by a Chinese contractor specialising in dam foundation works (see photograph). Otherwise, according to Hochtief's Edmund Lipgens, project manager for the Xiaolangdi dam, the construction technology will be 'state-of-the-art', but not 'innovative'.

'At present, site installation works are in full swing,' says Lipgens, 'The expatriates' camp, main office and main workshop have been completed and excavation for the right bank dam abutment has started.' Apart from this, other essential preliminary infrastructural work includes camps, offices, workshops, stores, crushing and screening plants, concrete batching plant, networks for electricity and water supply.

Project management

Canadian International Project Managers (CIPM Joint Venture), a consortium of SNC-Lavalin, Acres International, BC Hydro International and Hydro-Quebec International began to get involved at Xiaolangdi under a consulting services contract for the project preparation financed by the World Bank in July 1990. In the spring of 1991, assistance was provided to the Design Institute of the Yellow River Conservancy Commission of the Ministry of Water Resources of China, to prepare tender documents for international competitive bidding for the civil works and technical spcifications for the turbines. In September that year, a further agreement was signed for CIPM JV to carry out studies on the environmental, resettlement and economic components of the project, in consultation with Chinese colleagues at Zhengzhou.

In 1993, CIPM JV was invited to negotiate on a sole-source basis for a contract for the consulting services for the project construction supervision. This contract, for 500 man months of Canadian engineers, spread over a period from 1994-97, was awarded to the consortium in June 1994

Resettlement

Although the project involves the resettlement of over 180 000 people, Xiaolangdi does not seem to have attracted the same opposition as unfavourably dipping joints, can require extensive, expensive and time-consuming treatment. Often the critical consideration in the design of gravity dams is to assure adequate margins of safety against shearing-sliding and excessive non-elastic deformations at the weaker layers or zones in the foundations and the abutments. Two pertinent examples of high CMC gravity dams, where extensive treatment was necessary to strengthen weaker features in the foundations are the 226m high Bhakra Dam in India, completed in 1963, and the 196m high Itaipu Dam in Brazil-Paraguay, which was completed in 1982.

The foundations of Bhakra Dam[4] are predominantly sandstone and claystone with several major strata of siltstone and gouge filled shear zones. Without special treatment of the weaker zones, excessive differential settlements, as well as high uplift pressures would have occurred and impaired the stability of the dam. A typical sample of the treatment was that of a claystone zone averaging 35m wide, located about 25m upstream of the dam in the river bed. Extending into both banks and parallel to the axis of the dam, it dipped 70° downstream under the dam. The claystone, when exposed to the atmosphere and submerged in water, weathered and weakened rapidly. The treatment consisted of excavation of the claystone to a depth of 22m, backfilling with concrete, capping it with a 15m thick concrete slab extending from the dam, and consolidation and contact grouting around the concrete plug.

At Itaipu Dam[5], where the foundations for the hollow gravity dam are composed of sound basalt and breccia, a weak contact zone between basalt flows was encountered about 20m below the foundations of the highest blocks in the river bed. The weak contact zone was nearly horizontal and continuous for an area of 170m x 200m and overlain by sound basalt about 20m thick, which was suitable for the dam foundations. The weak zone is a few centimeters to 0.5m thick layer of fractured dense basalt with a predominance of horizontal open joints, which are sometimes imbricated. Pockets of highly fragmented rock with a film of clay occurred along 60% of the layer. Because of time constraints a plan of underground treatment was adopted. The treatment consisted of mining out the weaker rock from the shear zones and gouge seams through a grid of tunnels and backfilling them with concrete, improving the shear resistance of the weaker zone in the foundation. The total volume of excavation and concrete required for the shear key grid was 20 000 m^3, covering 25% of the foundation area.

The cases of special foundation treatment at Bhakra and Itaipu dams show that if RCC gravity dams were to be built at those sites, the same amount of foundation treatment would be required to assure margins of safety and quality as for the conventionally built dams. They also show that foundation treatment can be a critical item in the construction schedule, because most of it must be completed before the start of concrete placement. In some cases, in addition to curtain grouting, other supplementary foundation treatment may have to be carried out from the galleries in the dam, sometimes after filling of the reservoir.

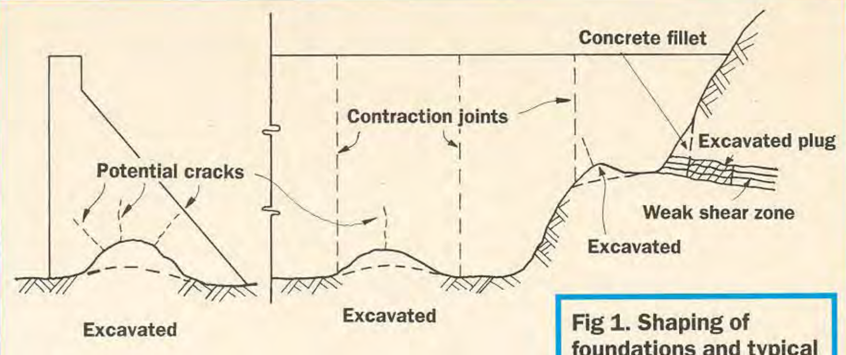


Fig 1. Shaping of foundations and typical treatment of irregularities

The importance of thorough foundation exploration and treatment for high RCC gravity dams, may also be learned from experiences in the design and construction of some relatively smaller ones. For the 68m high Concepcion Dam[6] in Honduras, the selection of an RCC dam was made before analyses were completed for resolution of such problems as: "zones of poor foundations; very poor quality cement and pozzolan; low strength aggregate with high absorption and low density concrete." For non-engineering reasons, construction had to be commenced before final drawings or specifications were available and "before the foundation level and axis of the dam were established." A similar programme would not be appropriate for a high and large RCC gravity dam, posing an unacceptable degree of risk to its safety and durability.

Shaping the foundations and abutments to eliminate site irregularities of the type shown in Fig 1, is equally important for high RCC and CMC gravity dams. If not removed or rectified, they would cause internal stress concentrations, resulting in cracking and progressive deterioration of concrete.

Stability of the abutments of a high RCC gravity dam also requires special attention, if there is axial transfer of loads from the dam. Three-dimensional stability analyses of the abutments would be necessary to determine the type of treatment required; it may also influence the structural design of the dam. While stability of a gravity dam may depend more on the quality and behaviour of the weaker parts of the foundation, the conjunctive stability of the structure and the foundations can also be degraded by poor quality and performance of the dam. Structural cracking, high internal pore pressures, uncontrolled leakage, leaching and alkali-aggregate reaction in the dam concrete can alter the stress patterns in the foundations and abutments to such a degree that the margins of safety against failure in the foundations are reduced below acceptable limits. Therefore the conjunctive stability of the dam-foundation complex must be studied as thoroughly for high RCC gravity dams, as for comparable CMC dams.

Elastic monolithicity of construction joints

Current experience indicates that the bond, shear and tensile strengths of typical untreated

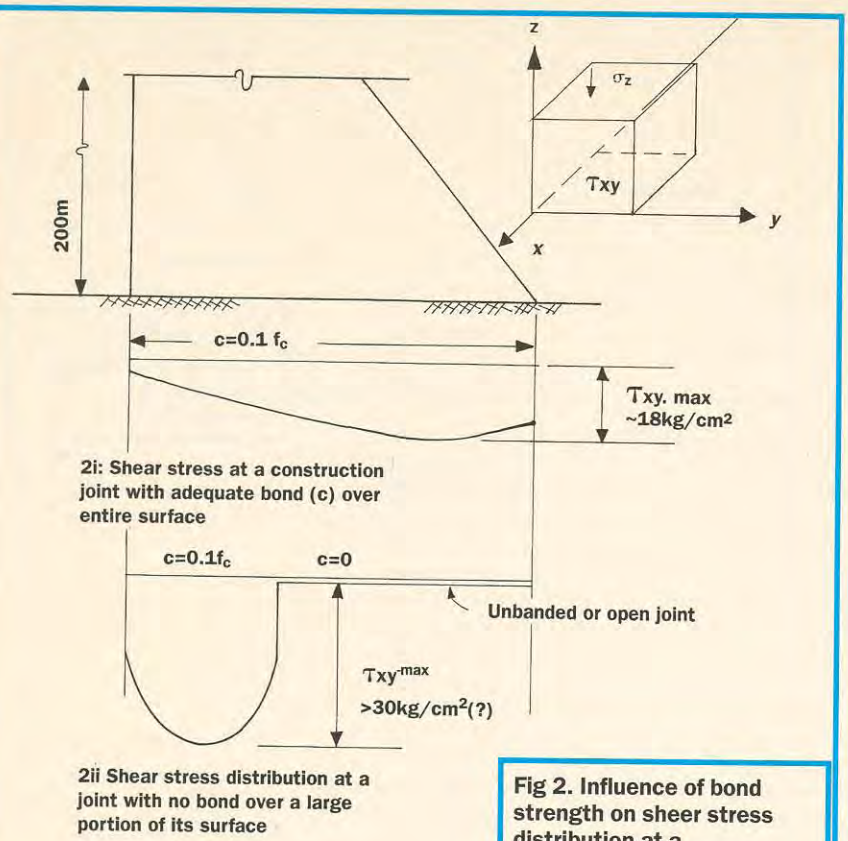


Fig 2. Influence of bond strength on sheer stress distribution at a construction joint

construction joints between layers are considerably less than that of the RCC itself. This is particularly true when no bedding mix or mortar layer is used on the entire joint surface and the time elapsed between placement of layers exceeds 8 hours[3]. On the other hand, in a CMC gravity dam, where construction joints receive such treatment as hydroblasting, sand blasting or green cutting, and without a new mortar layer, the shear, bond and tensile strengths of the joint are almost the same as that of the mass concrete. Special treatment of construction joint surface in CMC is required to remove laitance, contaminants and inferior material not compatible with cement, and to clean the surface of exposed aggregates and the already set mortar. Laitance and excess water may not seem a problem with the no-slump, low cement content RCC. But the paucity of mortar on the compacted construction joint surface would result in low and unevenly distributed bond, which is accomplished by cement grain and is not due to either the roughness of the surface or intimacy of surface contact between aggregates or hardened concrete,[3] and the construction joints would become planes of relative weakness in the dam.

Low shear and bond strengths at a construction joint also mean lower effective modulus in shear along the joint. The relatively weaker joints make the concrete mass laminated or anisotropic and elastically heterogeneous; it would have a different type of response to sustained, as well as dynamic, loads than a truly monolithic structure. Ideally, to ensure elastic and monolithic response of the dam to all types of loads in all directions, including shearing forces, the bond, shear and tensile strengths of the construction joint should be equivalent to that of the concrete. A joint substantially weaker in shear than the concrete, would alter the deformations and magnitude and distribution of normal and shear stresses in the concrete and, particularly, at the joint. In high RCC gravity dams, the resulting shear and tensile stresses may exceed the permissible limits. Thus, sufficient bond and shear strength at the horizontal construction joints is necessary not only for adequate safety against shearing-sliding, but also against overstressing, cracking and subsequent deterioration of concrete, for both RCC and CMC gravity dams.

Typical distribution of horizontal shear stress along a construction joint or at a horizontal plane in mass concrete of a CMC dam is shown in Fig 2.1. In an RCC gravity dam, shear stress distribution at a construction joint could be entirely different. If a bedding mix is used only near the upstream face, bond strength in that part of the joint would be nearly equal to that of the concrete. If the rest of the joint surface is untreated and it has very low or no bond, the distribution of shear stress would be altered with "concentrations" occurring in the upstream part of the joint, as shown in Fig 2.2. Conversely, if the bond in the upstream portion is much lower than along the rest of the joint and tension in the upstream portion opens it, critical shear stresses would build up in the downstream portion with a risk of ultimate shear failure commencing there.

The effective modulus of elasticity in shear would vary similar to the variation of bond along a construction joint. For a 200m high CMC gravity dam, the maximum horizontal shear stresses would be about 18 kg/cm^2. If the average bond strength of a joint is 60% of that of the RCC, or that of a joint with a bedding mix over its entire surface, then the maximum shear stresses might be 50% higher than in a comparable CMC gravity dam. If the maximum shear stress exceeds the effective bond strength of a construction joint, then that joint could be the weakest feature in the structure where shear failure might commence. For high RCC gravity dams such a situation would pose a high risk.

To better appreciate the impact of the above discussed "lamination effect" on the structural behaviour of the dam, data on two 200m high RCC and CMC gravity dams with identical cross-sections are compared. Assuming that RCC is placed in 0.30m layers and conventional concrete in 2.5m lifts, the total area of construction joints in the RCC dam would be more than eight times that in the CMC dam. Because of the fast rate of RCC placement, it is likely that bond and shear strength will vary considerably over the joint surface, and with values >50% of that of concrete over perhaps 30% of the area. For a 100m length of the dam, it represents joint surface area of about 1.5×10^6 m^2 which may have deficient shear strength, and hence lower margins of safety against a shear failure, than considered acceptable for a high gravity dam. This deficiency and the corresponding risk, increases with the height and size of the RCC gravity dam.

Stability against shearing-sliding

Stability of a gravity dam against shearing-sliding, at construction joints, through weaker or cracked portions of the concrete, and at or near the dam-foundation contact, is a primary consideration. However, it is not the only factor governing the safety and stability of the dam. Since typical shearing-sliding analyses are considered relatively simple, there is a noticeable trend in current design practice for RCC gravity dams to emphasise this aspect while not addressing other equally important factors, such as cracking, which also affect stability of the dam. For example, the ACI report, "Roller Compacted Concrete"[1] states that "gravity dams are designed essentially for stability against overturning and sliding."

A typical shearing-sliding analysis involves the determination of the shear-friction factor (SFF) defined as follows:

$$SFF = \frac{CA(\sum N - \sum U)\tan\varnothing}{\sum H}$$

where:
C = unit shear resistance or bond or cohesion.
A = uncracked area of potential sliding surface.
Σ^{N} = summation of normal forces.
Σ^{U} = summation of hydraulic uplift force.
tanø = coefficient of internal frictional resistance.
Σ^{H} = summation of shear forces.

This apparently simple factor for assessing the stability of concrete gravity dams, is predicated on monolithic behaviour of the entire structure with elastic continuity at the construction joints. For monolithic behaviour, bond or cohesion along the entire surface of the construction joints is indispensable. Since sliding-shearing is essentially a progressive phenomenon, frictional resistance is fully mobilised only after the bond has been overcome by differential shear. Several questions need to be considered regarding sliding-shearing along construction joints: Where will it commence? At what rate will it progress? How much differential shear movement would render the dam block unstable? Can "residual" cohesion at a joint which has already undergone differential shear deformation be relied upon to assure an adequate margin of stability?

A more realistic method of assessing stability against shearing-sliding is to determine values of "unit" shear-friction factor, that is per unit area of the potential sliding plane. The unit SFF will vary from the upstream to the downstream side of the dam, being low or even negative in the critical upstream part, where low normal compressive forces, or even tension, is likely to occur and the uplift forces or pore pressures should be higher. In some RCC dams only a small upstream portion of the construction joints is covered with a bedding mix to provide sufficient bond, which when combined with frictional resistance over the entire joint would obtain the desired shear-friction factor. However, the distribution of the frictional resistance, $[\Sigma^{(N-U)}\tan\varnothing]$, would not be uniform along the joint due to the variation of the vertical normal stress and uplift. If bond is lacking over a large portion of the joint surface, frictional resistance over that part can not be relied upon to provide additional resistance to shearing; the upstream bonded part would tend to shear first and the actual factor of safety against shearing-sliding may be unacceptable. In order to assure adequate stability against shearing-sliding at the construction joints, in high RCC gravity dams sufficient bond should be provided over the entire surface of each joint.

Effective bond and shear resistance of a construction joint in an RCC dam is influenced mainly by the time interval between placement of RCC layers, and type and extent of treatment of the joint before placement of new layer of concrete. Large scale field and laboratory tests have been made for several RCC dams to assess the effectiveness of various types of treatment of construction joints.[3,7] The results for Capanda[8] dam, Fig 3, show great improvement in shear strength when a bedding mix is placed on the joint. Significant conclusions drawn from results of tests at several relevant dams are:

- Without a layer of bedding mix, the effective tensile and shear strengths at the joint would be 50 - 60% of that of RCC, depending upon the time interval between layers.
- Use of a layer of bedding mix immediately before placing the new RCC layer improved the bond of the joint about 40%, to almost that of RCC itself, regardless of the time interval between layers.
- Cleanup of joint surface with low pressure (about 7 kgf/cm^2) air-water jet before placement of the new RCC layer, increased the bond strength 5 - 10%.

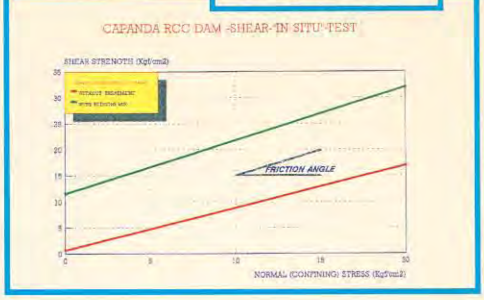
An additional benefit of using a bedding mix over the full area of the joint is the improvement in its impermeability, offsetting the need for a thicker upstream zone of CMC, provided in some completed RCC dams[16]. The Capanda tests also showed that with bedding mix, the shear strength of the joints was considerably higher than that of the dam-foundation contact; the foundations at Capanda dam are predominantly meta-sandstone. The shear strength of the construction joints without bedding mix was about the same as that of the dam-foundation contact.

Considering the above to obtain adequate monolithicity and shearing resistance along construction joints, for high RCC gravity dams, the following criteria are suggested:

- Adequate bond and shear strength over the entire surface of each construction joint is essential for monolithic action of the dam block. In areas of high seismicity, the construction joints should also have adequate tensile strength, particularly those upstream.
- Before placing a new layer of RCC, the entire surface of the old compacted layer should be cleaned with low pressure air-water jet.
- Regardless of the interval between placement of RCC layers, place a thin layer of bedding mix over the entire surface of the joint, immediately before placing the new layer of concrete.

While these measures would increase the costs and slow down RCC placement, the improvements in quality and longterm performance of the high dam would be cost-effective, with reduced maintenance costs and stability factors equal to those of a comparable CMC dam.

Fig 3. Capanda RCC dam - shear "in-situ" test



Part II of this paper will be published in Our June issue.

Special factors in design of high RCC gravity dams Part II

In the second part of the paper, the authors discuss the different types of cracks which may occur in a high RCC gravity dam and the role played in their prevention by contraction joints and the quality and composition of the materials used.

By G S Sarkaria and F R Andriolo***

Structural cracks are defined as those that occur within the body of the dam and alter its monolithic, isotropic and elastic behaviour. Shrinkage cracks, which mostly occur in the exposed surfaces of concrete, have a random pattern and do not penetrate more than a few centimeters from the surface, are not considered structural cracks.

The most common cause of structural cracking is the high tensile stresses that occur during the relatively rapid rate of cooling and consequent contraction of concrete. The higher the constraint against contraction, the higher the tension and cracking commences where tensile stress exceeds the tensile strain capacity of the concrete. Structural cracks can also be caused by excessive, differential and rapid foundation deformations, tensile overstressing of concrete, such as around large openings or due to an earthquake, or can be due to alkali-aggregate reaction. In the worst scenario, structural cracking may be caused by a combination of several factors.

The undesirable consequences of structural cracking in a gravity dam can be multifarious: leakage into the galleries, or through the dam, to the downstream side, leakage into the dam from the foundations, leaching and weakening of concrete, restart or spreading of dormant alkali-aggragte reaction, acceleration of weathering and deterioration of concrete, particularly due to freezing-thawing, impairment of the structural integrity and stability of the dam; high internal uplift pressures if the cracks link with saturated or leaky construction joints and difficult and expensive repairs, particularly if the reservoir cannot be emptied.

Transverse cracks in a gravity dam generally commence at the concrete-foundation rock contact and progress upward and inward into the dam. The cracks may start during the first colder season, six to twelve months after concrete was placed on the foundations, and may not be visible for several months thereafter. The cracks travel faster when the weekly drop in ambient temperature exceeds 20°C. If placement of RCC is interrupted for periods exceeding 30 days during the cold season, even with surface insulation, the cracks may extend to the surface, yet not be easily discernible. Cracks can also start at a surface exposed to rapid and large drops in daily ambient temperatures and progress into the concrete.

Sometimes the cracks may stay dormant in the body of the concrete for a long period and reach the surfaces only when external loads such as reservoir pressure, foundation deformations, prolonged freezing weather and rapid drop in temperature of the outer zones of concrete, or earthquake increase the tensile stress concentrations at the termini of the cracks.

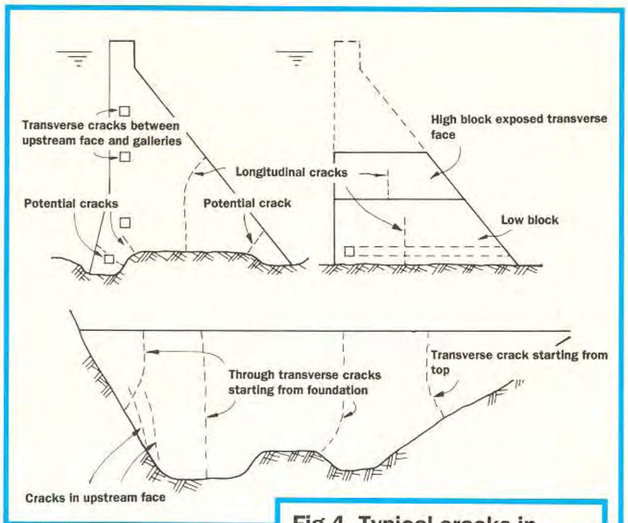


Fig 4. Typical cracks in concrete dams

The orientation, configuration and extension of transverse cracks may be neither a vertical plane normal to the axis or the upstream face of the dam, nor extend across the full cross section of it. Fig 4 shows the various types of cracks that have occurred in RCC and CMC gravity dams. The configuration and extent of a crack may radically change as the reservoir is filled and continue to change over a period of several years. In colder climates, the seasonal changes in the opening and extension of a crack can be pronounced.

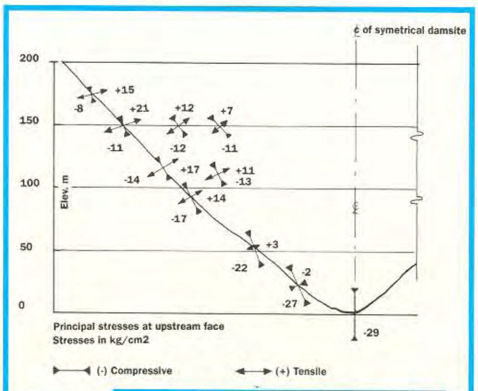


Fig 5. Typical stresses at upstream face of a 200m high monolithic gravity dam (Reservoir full + earthquake)

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Table I Crack in RCC dams

DAM	CREST LENGTH M	MAX HEIGHT M	RCC VOLUME 10(3)m³	CEMENTITIOUS CONTENT kg/m³	DESCRIPTION OF CRACKS
Copperfield, Australia	340	40	140	95C + 15FA	Three contraction joints provided. One through transverse crack with seepage.
Elk Creek, USA (1st)	365	25	260	118C + 56FA	Cracks between contraction joints at 90m spacing.
Galesville, USA	290	51	170	52C + 54FA	Seven transverse cracks; some started from top of dam. Leakage.
Knellport, South Africa	200	50	59	61C + 142FA	Through induced cracks where planned. Some leakage.
Riou, France	308	26	45	40C + 120FA	No contraction joints. Several cracks.
Upper Stillwater, USA	815	90	1070	80C + 175FA	Several through transverse cracks at 5 to 15m spacing
Uruquai, Argentina	690	76	600	60C	Four through transverse cracks.
Wolwedans, South Africa	268	70	210	58C + 136FA	Through induced cracks-joints. Some leakage.
Zaaihoek, South Africa	527	47	134	36C + 84BFS	Through induced cracks-joints. Some leakage.

C = Cement; FA = Fly ash; BFS = Blast furnace slag

Cracks along a sloping abutment are likely to commence normal to the abutment slope and curve into a near vertical direction, which may not be normal to the axis. Transverse cracks can also occur in the upper part of the dam placed against a near vertical abutment, because of the high restraint against deflection of the structure. Transverse cracks in concrete upstream of a longitudinal gallery can be caused by steep temperature gradients between the gallery and the upstream face of the dam exposed to cold air or water.

Typical ranges of principal stresses which may be expected in the upstream face of a 200m high, fully-monolithic gravity dam, that is with contraction joints keyed and grouted, are shown in Fig 5. Semi-horizontal tensile stresses are indicated in the upper half near the abutments; if the dam is located in a wide valley, these tensile stresses would be more pronounced. In long RCC gravity dams, such as Upper Stillwater[1] and Uruguai[2], the cracking due to thermal effects may have been augmented by the structural tensions due to transfer of loads to the abutments by beam action and torsion. The longer the monlithic dam, the larger the contribution of such tensions to cracks in the dam.

Because of the above stated consequences, structural cracks are considered unacceptable in high CMC dams. Unfortunately, the implications of cracking in RCC gravity dams are often misunderstood and not appropriately emphasised. For example, the definitive ACI report 'Roller Compacted Concrete'[1], states that, 'the principal concern for cracking in gravity dams without contraction joints is appearance and leakage control'. Such statements seem to imply erroneously that a cracked gravity dam is as good as a non-cracked one, as long as leakage through the cracks is stopped or controlled and the appearance of the exposed faces of the dam is pleasing to the eye. For high RCC gravity dams, such a rationale regarding the importance of crack control can result in dams of inferior quality and durability, with shorter operational life and higher maintenance costs.

To reduce the risk of cracking, several provisions are adopted in the design of both RCC and CMC dams. These are: low cement and adequate pozzolan content, use of low heat cement, precooled aggregate to control placement temperatures, restricting concrete placement to cooler periods and provision of contraction joints. Full-section transverse contraction joints at 15-20cm spacing have been unversally provided in all CMC dam of various heights and sizes. However, in a large number of already completed RCC gravity dams, 90m in height, contraction joints were either not considered necessary, or only a few were provided. It was reasoned that zero-slump RCC has much lower shrinkage than conventional concrete, and with very low cement content and heat of hydration, continuously placed RCC would have adequate tensile strain capacity and, with proper control of placement temperatures, it should not develop structural cracks. However, experience at several completed RCC dams has shown that without properly spaced full transverse contraction joints, cracking could not be prevented. Table I lists nine completed RCC dams, where transverse cracking has been reported.

Transverse contraction joints

The efficacy of transverse contraction joints in preventing structural cracks in CMC gravity dams of various sizes has long been proven. However, for RCC dams, some designers have been reluctant to accept the necessity of full transverse contraction joints at 15-20m spacing, mainly because joint construction could slow down RCC placement and increase cost. Instead, partial measures, such as crack inducers or joints in the upstream conventional concrete were adopted. In some RCC dams, a few full transverse contraction joints were provided only at locations where differential foundation settlement was expected or where there were pronounced irregularities in the foundation profile. Another line of reasoning would accept cracking in a dam storing water infrequently, or for short durations.

Parallel to the construction of jointless RCC dams during the past 12 years, several RCD gravity dams were built in Japan with transverse contraction joints. The RCD type is considered a conservative version of the standard RCC dam. Since no completed RCD dam has suffered cracking, the additional cost of providing contraction joints, if any, is considered justified, particularly for high RCC gravity dams. Table II lists RCC dams with transverse contraction joints which have either been completed or are proposed for construction in the near future.

Engineers in several countries have conduct-

Table II Crack free RCC dams with transverse contraction joints

DAM	CREST LENGTH M	MAX HEIGHT M	RCC VOLUME 10(3)m^3	CEMENTITIOUS CONTENT kg/m^3	CONTRACTION JOINT SPACING	REMARKS
Aoulouz, Morocco		70	610	100C	45	Stone leakage
Asahiogawa, Japan	260	84	361	96C + 24FA	15	No cracking
Mano, Japan	239	69	219	96C + 24FA	15	No cracking
Miyagase, Japan	400	155	2000		15	Proposed
Pirika, Japan	910	40	365	84C + 36FA	15	No cracking
Quail Creek, USA	609	42	131	135C + 152FA	40-100	No cracking where joint spacing $\leq$ 60m, except along abutments
Shimajigawa, Japan	240	89	317	91C + 36FA		15 No cracking
Shuikou, China	646	50	300	50C + 100FA	30	No cracking between block joints

ed studies to determine the optimum spacing of transverse contraction joints to prevent cracking in RCC dams and in theoretical conclusions verified against the experience of completed dams. A summary of the current practice and experience regarding crack prevention and contraction joints in various countries is as follows (see also Tables I and II):

- **Japan[13]** Full joints at 15m are standard provision. No cracking reported in ten completed dams.
- **Spain[15]** Full joints at 40-100m in valley part of dam and closer spacing along abutments. Partial joints or crack inducers not considered beneficial.
- **Morocco** (Aoulouz dam) - Full transverse joints at 45m and primary joints in upstream conventional concrete at 15m.
- **USA** (Willow Creek, Galesville, Middle Fork, Monksville, Copperfield and Upper Stillwater dams) - No full contraction joints provided. Through cracks, some with seepage, occurred in all these jointless dams.
- **USA** (Elk Creek dam) - Contraction joints at maximum 91m spacing. Cracking has occurred between several joints in the completed first stage of the dam.
- **USA** (Quail Creek dam) - Full contraction joints cut at 14.6m spacing. No structural cracking reported.
- **China** (Shuikou Diversion Wall)[19] - Joints provided at about 30m spacing. No cracking reported.
- **South Africa** Crack inducers installed at 10m spacing in Zaaihoek gravity dam and in Knellpoort and Wolwedan arch-gravity dams[12]. Cracks with some leakage occurred in all three dams where the inducers were located. It is not proposed to provide full contraction joints in future dams.

In five relatively high RCC gravity dams which are scheduled for construction in the near future, Miel-I (185m), Miel-II (141m) and Porce II (118m) in Colombia, Platanoryssi (95m) Greece and Pangue (115m), Chile, contracton joints are to be provided selectively to respond to changes in foundation conditions and to form the spillway blocks. Spacing of joints is 60m in Miel I and II, 40m in Porce II and about 50m in Pangue and Platanoryssi dams. It is also pertinent to note that thermal stress analyses for Upper Stillwater[20] dam had concluded that a joint spacing of 60m would be adequate to prevent cracking. Actually, cracking occurred at closer spacing than that anticipated.

The authors believe that provision of full transverse contraction joints at closer spacing of 15-20m, would be prudent for high, long and large RCC gravity dams. The Japanese experience in building the highest RCC dams and that at other dams such as Quail Creek[22], USA, has shown that the joints can be cut without slowing down construction and at a nominal additional cost. Such contraction joints also provide assurance against cracking when ambient temperatures may not be ideal at the time of RCC placement, or when cement content may have to be increased to compensate for poorer than specified quality of pozzolanic materials. This in turn can avoid delays in completion of the dam, while insuring a better quality crack-free RCC structure.

Longitudinal cracks and joints

Before 1965, most CMC gravity dams ≥150m in height, were built with longitudinal contraction joints. The world's highest CMC gravity dams, Grande Dixence, Switzerland (285m) and Bhakra, India (226m) have two or more longitudinal contraction joints. The principal purpose of the longitudinal joints was to prevent longitudinal cracks. These joints had to be grouted before filling the reservoir and after the concrete had been slowly cooled down to the average ambient temperature, by circulating cold water through embedded pipes placed on top of each lift. Longitudinal contraction joints were also provided in several high thick arch dams. Performance of these high dams over 40-50 years has proven the efficacy of longitudinal contraction joints in preventing longitudinal cracks and facilitating essentially monolithic behaviour of the structures as anticipated in design.

Since forming of longitudinal joints and their treatment increase construction costs, during the last 25 years several high CMC gravity dams were built without longitudinal contraction joints. Noteworthy examples are: Dworshak, USA (219m), Revelstoke, Canada (175m) and Piedra del Aguila, Argentina (172m). The designs of these dams incorporated several features to eliminate the risk of longitudinal cracking, such as the use of relatively lean concrete mixes with low cement and adequate pozzolan content, low concrete placement temperatures and post-cooling in the lower part of the dam. The criteria for Revelstoke[9] dam are typical: it is assumed that if the temperature gradient of the concrete near the foundation rock did not exceed 21°C, then tensile strains that would develop as the concrete cooled would not be high enough to cause cracking. Concrete mix was designed following the concept of crack-free mass concrete for Dworshak and Libby dams[10]. However, Revelstoke, Dworshak and Libby dams suffered some unanticipated structural cracking before, as well as after, filling of the reservoir[9]. In Revelstoke and Libby dams, some of the cracks were longitudinal.

Longitudinal cracks generally commence from the foundation in the central part of the block and travel upward, and can also occur in a new lift placed over older concrete. The upper part of the old concrete may have cooled considerably and with the temperature of the new concrete peaking about a week after its placement, a large temperature gradient at the joint would induce longitudinal cracks in the new layer.

Since longitudinal cracks progress upward slowly, if subsequent lifts of concrete are placed rapidly at short intervals, they may not be noticed during construction. Such cracks are generally detected in transverse galleries or unlined outlet conduits in the lower portions of the dam; sometimes several years after completion of the dam and filling of the reservoir. Longitudinal cracks may also reach the top of a block where concreting had been interrupted for a whole cold season, or appear on the exposed transverse face of a block.

Longitudinal cracks can endanger the stability of a dam, particularly if they show a tendency to progress as reservoir pressure is applied, or if they connect with transverse cracks or joints, or are penetrated by water under high pressure. Another consideration is the risk of extension of such cracks during an earthquake. Therefore, prevention of longitudinal cracking in high gravity dams, whether RCC or CMC, is as imperative as the control of transverse cracks.

The experience of high CMC gravity dams discussed before, where transverse or longitudinal cracks occurred despite all the preventive design provisions, indicates that the risk of longitudinal cracks occurring in high RCC dams cannot be ignored. Such advantages of RCC as no-slump concrete and lower heat of hydration because of lower cement content, are offset by the lack of post-cooling, which was provided in lower parts of Dworshak and Revelstoke dams. While no longitudinal cracking of RCC dams has been reported so far, it should not be a reason for complacency, because, due to the lack of transverse galleries and rapid placement of RCC, such cracks cannot be visually detected during construction, may not surface for several years, may remain hidden and may connect with transverse cracks.

The risk of longitudinal cracking in RCC gravity dams increases with height. For a 200m high dam with vertical upstream face and 0.8:1.0 downstream slope, the lower layers would be 160m wide. Considering that in RCC dams contraction joints at 15-20m intervals in the axial direction are considered necessary to reduce the tensile strains near the foundations, why would that not be not be valid in the transverse direction? Another risk factor is the concrete mix, if it is considered necessary to increase the cement content near the foundations to obtain higher strength RCC; the need for a richer mix increases with the height of the dam.

Since post-cooling for grouting vertical longitudinal contraction joints in RCC dams would be impractical and costly, an alternative would be to provide an inclined longitudinal construction joint. Essentially, the RCC dam would be built in two stages. The concept of the two-stage construction of a 200m high RCC gravity dam is shown in Fig 6. The lower 100m of the dam would be constructed in two monoliths, A and B; A being constructed first, with B following it a few weeks later, as may be convenient for construction, and to ensure that the temperature differential between concrete in the two monoliths is not excessive when second stage concrete is placed. Two practical alternatives are suggested.:

1) Monolith A is on the upstream side, with the construction joint parallel to the downstream face of the dam. Its principal advantage is that Monolith A can be completed rapidly to a height above the upstream cofferdam, providing additional security against flooding if the cofferdam is overtopped. It may also be used for earlier start of storage in the reservoir. Another positive feature is that the weight of Monolith B would exert compressive pressure on the joint, closing it and improving its shear strength and would offset the flexural tensile stress component when the reservoir pressure is applied. A possible disadvantage of this arrangement is that tensile concentrations may occur at the interface with Monolith B at the top of Monolith A, which may require steel reinforcement in that zone to prevent cracking.

2) Monolith A forms the downstream part of the dam and has an upstream slope of 0.40:1.0. Upstream inclination of the construction joint is necessary to take advantage of the weight of Monolith B and the component of the principal stress normal to the joint. The main disadvantages of this alternative are: a)the volume of concrete in Monolith A is 55% more than Monolith A of Alternative I; b) cofferdam protection will be required for constructing the second stage and c) early storage of water in the reservoir would not be possible.

Alternative 1 is preferable and more suitable than Alternative 2 from the viewpoints of construction convenience, early storage of water and overall economy. Alternative 1 is similar in concept to the two-stage construction of the Guri[14] dam, Venezuela, where the 110m high first stage was built in 1968 and the second stage raised to a maximum height of 162m fourteen years later. The reservoir was full when the second stage was added. Both stages of the dam were built of conventional mass concrete.

The sloping construction joint could be stepped or formed with a plain surface. Before placing the second stage (Monolith B) RCC,

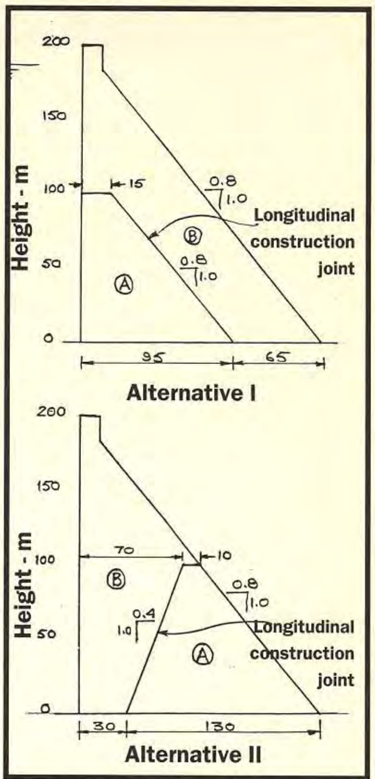

Fig 6. Two-stage construction of a 200m high RCC gravity dam

Table III Density of RCC and CMC

PROJECT OR STUDY	AGGREGATE Type	Specific gravity	DENSITY (KG/M³) RCC	CMC
Bocaina	Quartzite	2950-3000		2418-2609
Capanda	Metasandstone	2620-2640	2438-2450	2420-2445
Itaipu	Basalt	2900-2950	2507-2625	2605
New Victoria	Granite-dolerite		2500	
Saco Novo Olinda	Granite-GNEISS	2744	2295-2365	2252-2350
Serra da Mesa	Granite	2640	2325-2475	
Tucurui	Meta-sedimentary	2730-2760	2400	
Urugua-1	Basalt	2900-2950	2632-2694	2450

the joint surface would be thoroughly cleaned by high pressure hydroblasting. A layer of conventional concrete, about 0.3m wide, would be placed against the joint, as each layer of RCC is placed and compacted. Tests at Ross[3] dam, USA, Guri and other dams have shown that adequate bonding can be obtained between conventional old and new concrete by hydroblasting or equivalent treatment and that keys or steel dowels are not required. Two-stage construction with inclined construction joints was also adopted for some blocks of Capanda[8] dam in Angola.

Quality of RCC

How does RCC compare with CMC as a material suitable for building high and large gravity dams of the same durability and quality as existing dams which have performed well for several decades? Comparison of certain pertinent properties of RCC and CMC is made to answer this question.

Density: The density or unit weight of RCC is either the same or somewhat greater than that of CMC with the same materials. The main reason for higher RCC density is lower water content. Table III shows density of typical RCC and CMC used at some projects.

Compressive strength: Since the compressive strength of concrete depends on the propoerties of its many ingredients and other factors, comparative analysis of RCC and CMC may be based on:

$$=\text{Mix efficiency}=\frac{\text{Compressive strength (kg/cm}^2)}{\text{Cementitious material (C+P) (kg/m}^3)}$$

Mix efficiencies at various ages for 24 RCC and six CMC dams or studies are plotted in Fig 7 for Capanda, Itaipu, Tucurui, Upper Stillwater and Uruguay, where tests comparing RCC and CMC using the same constituents were made.

Generally mix efficiency is higher for RCC than for comparable CMC, meaning that desired compressive strength of RCC can be obtained using lower cementitious content, particularly portland cement. However, RCC with richer mixes, such as those tested at Itaipu and Upper Stillwater, had somewhat lower mix efficiency than CMC. This indicates that for high RCC gravity dams, cement content would have to be higher than that required for low RCC dams, to obtain compressive strength equal to that in comparable CMC dams.

Tensile strength: Like compressive strength, tensile strength of RCC and CMC also depends on the cementitious content and age. For CMC, tensile strength is considered to be 10-15% of compressive strength. Data from 15 dams or testing programmes, showing the ratio of splitting tensile strength to unconfined compressive strength, indicate that the average tensile strength of RCC is also 10-15% of its compressive strength.

Shear strength: Results of direct, biaxial and triaxial tests performed on cores obtained from test fills and completed dams, and of in situ tests, indicate that the shear strength components, c and ø, of RCC are comparable to that of CMC made from similar aggregates. While cohesion is dependent on the cementitious content, the angle ø is affected by the quality and gradation of the aggregates. Typical values of shear strength parameters for some RCC and CMC dams are shown in Table IV.

Modulus of elasticity: The main factors that influence the modulus of elasticity of RCC and CMC are: age, type of aggregate and its modulus and water-cement ratio or paste content. Fig 7 illustrates the increase in modulus of elasticity with age for five CMC and 13 RCC dams or test programmes. It is seen that the modulus of elasticity of RCC is considerably lower than that of CMC; about 50% at 7-28 days and about 65% at 90 days and later. Tests on core samples obtained from RCC used as backfill at Itaipu showed the same modulus at 3090 days as CMC with the same mix.

Creep: The coefficient of creep is mainly influenced by the modulus of elasticity of the aggregate and the amount of fines or mortar in the concrete mix. Due to the higher mortar content of RCC, its coefficient of creep is higher than that of CMC made of similar aggregates.

Table IV Shear strength of RCC and CMC

PROJECT	CEMENTITIOUS CONTENT kg/m³	SHEAR STRENGTH PARAMETERS RCC C-kg/cm²	RCC 0°	CMC C-kg/cm²	CMC 0°
Capanda-RCC	100	31.7	42		
Capanda-CMC	140			36.5-29.9	43
Elkcreek	105	8.5	57		
Galesville	105	26.8	33		
Ilha Solteira	200-350			55-70	31-44
Itaipu	100-350			22-96	40-42
Serra da Mesa	200	22	58		
Tamagawa	130	30	49		
Upper Stillwater	253	21	57		
Urugua-1	60	24.6	48		

Tensile strain capacity

Test data for tensile strain capacity for RCC for Capanda and Uruguai dams as compared to that for CMC indicate that the strain capacity of the two types of concrete using the same amount of cementitious material, at various ages, is about the same.

Thermal properties

The adiabatic temperature rise for both types of concret is essentially proportional to the portland cement content of the mix. For thermal analysis and design of concrete mix, a critical parameter is the adiabetic temperature rise per unit of cementitious content ie C/kg/m^3. Other thermal properties, such as diffusivity, specific heat, conductivity and coefficient of thermal expansion, are practically the same for both RCC and CMC composed of the same ingredients in the same proportions.

Permeability

Coefficient of permeabilty of RCC ranges from 10^{-6} to 10^{-12} m/s with cementitious content from 60-250kg/m^3, as compared to 10^{-9} to 10^{-12}m/s for CMC with similar cementitious content.

The improved RCC mixes with a higher percentage of fines, which have about the same low permeability as CMC, are better suited for high RCC gravity dams, because less permeable concrete is more durable.

Conclusion

The above comparison of important physical properties of RCC and CMC indicates that modern RCC is good concrete and that high and large RCC gravity dams of the same quality as existing major CMC gravity dams can be designed and built, providing stringent quality control is exercised in the selection of materials, design of the RCC mix and during construction.

Large spillways over dams

In most gravity dams, the ideal location for a spillway is over the dam itself. As larger and higher RCC gravity dams are built, the spillways for some of them will have large discharge capacities and large crest control gates.

The specific discharge, q=m^3/s/m length of crest, is often the governing criterion in sizing the spillway and its gates. Considerations of economy, energy dissipation and frequency and duration of spillway operation, determine the design q and the height of the crest gates. Economics generally dictate height and number of crest gates. In current practice, the largest radial gates for spillways over CMC gravity dams range 15-20m in height and 10-20m in width.

Large spillway gates are mobile dams, which transfer very large forces to the piers supporting their trunnions. In turn, the loads on the piers, which are reinforced concrete structures, are transmitted into the body of the dam. The piers and the dam blocks under them need to be designed as an integral structural unit, without overstress in the unreinforced concrete and particularly, in its construction joints.

Would a spillway structure, comprising a cap of CMC over an RCC dam have adequate margins of safety and ensure safety and operability of the gates under all emergency conditions?

For high and large gravity dams, like any other type of dam involving a large investment and a high downstream hazard potential, the overall safety and operability of the spillway should be a primary consideration.

Therefore, for high RCC gravity dams with large gated spillways, it may be necessary to increase the CMC component of the spillway blocks.

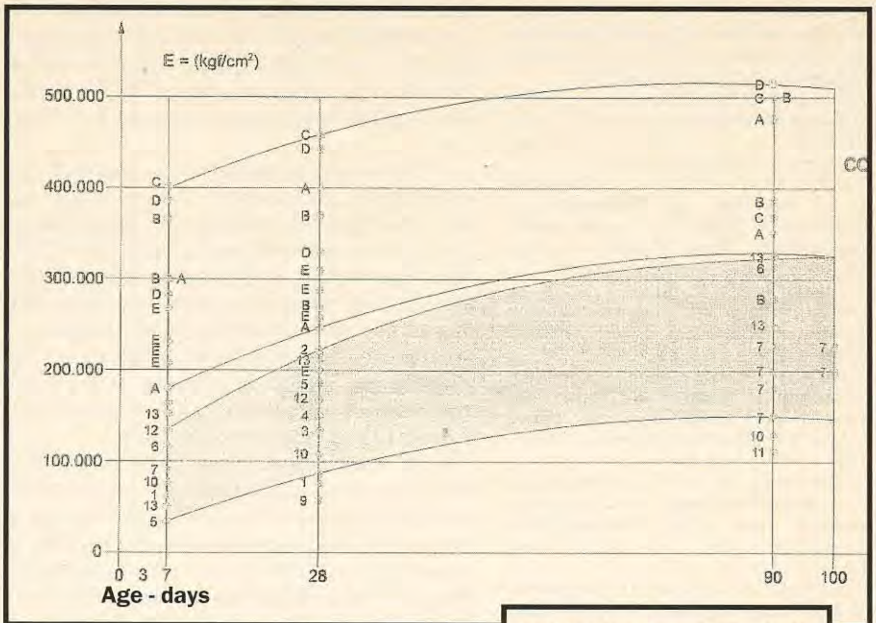

Fig. 7: Modulus of Elasticity of RCC and CMC

Conventional Concrete
A - Itaipu
B - Agua Vermelha
C - Ilha Solteira
D - Tucurui
E - Capanda

RCC Concrete
1 - Middle Fork
2 - Pamo
3 - Upper Stillwater
4 - Lower Creek
5 - Les Olivettes
6 - Itaipu
7 - Uruguai-i
8 - Rwedat
9 - Monksville
10 - Willow Creek
11 - Elk Creek
12 - Serra de Mesa
13 - Capanda

Drainage and seepage control

The durability of a wet concrete dam can be impaired by leaching of minerals, excessive weathering of exposed surfaces and damage by freezing and thawing. Damp concrete is more susceptible to alkali-aggregate reactivity than dry concrete. Also, pore pressures in the body of the dam, both CMC and RCC, can be a destabilising force similar to hydraulic uplift at construction joints or at the dam-foundation contact.

The internal drainage systems, namely a curtain of formed or drilled drains near and parallel to the upstream face, discharging into galleries in the dam from which the drainage water can flow out, either by gravity or by pumping, have proved most effective in keeping many large CMC dams dry. Using RCC with coefficients of permeability as low as that of CMC and use of a bedding mix over construction joints, the same type of drainage systems would also be effective in keeping high RCC dams dry and seepage under control.

In some RCC dams[16], the drainage curtain and the galleries are located at considerable distance downstream of the upstream face. In such dams, a large upstream portion of the dam could become saturated and develop high pore pressures. This is undesirable in high dams, because this part of the dam will have either low compression, or tension, and would be susceptible to cracking, which, combined with hydro pressure in the cracked portion, can marginalise the stability of the dam.

Summary and conclusions

1. Experience gained in design and construction of smaller RCC dams indicates that RCC can be successfully employed to build high gravity dams of the same quality as comparable CMC dams which have been in satisfactory service for several decades.

2. The acceptance standards of quality and safety for RCC dams should be the same as those currently internationally accepted for comparable CMC dams. However, the performance of several completed RCC dams has

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demonstrated the need to improve certain deficiencies with respect to selection of materials for RCC, foundation treatment, structural monlithicity, crack prevention and leakage, when compared to the standards for CMC dams.

3. The scope of exploration, analyses and beneficiation treatment of the rock foundations of a high RCC gravity dam should be the same as that required for a comparable CMC dam. The impact of foundation treatment on costs and the construction schedule of the dam should not be underestimated at the time of selection of type and layout of the dam. The foundations should be shaped to eliminate irregularities which may cause stress concentrations in and cracking of the dam.

4. Adequate bond, uniformly distributed over the entire surface of each construction joint, is essential for obtaining the necessary degree of elastic monolithicity in a high RCC gravity dam. Without such adequate bond, there may occur higher than admissable shear stresses and an unacceptable risk of shearing at a weak construction joint.

5. Treatment of the full surface of each construction joint, comprising a thin layer of bedding mix of suitable strength, regardless of the time interval between placement of layers of RCC, would provide adequate bond and shear strength and monolithicity. It would also eliminate leakage through the joints.

6. Prevention of structural cracks in a high RCC dam should be a mandatory objective. Transverse contraction joints for the full crosssection of the dam, provided at intervals not exceeding 20m for the entire length of the dam, are effective in preventing transverse cracking.

7. The risk of longitudinal cracking in the body of the dam increases with height and volume of the RCC dam and the cement content of the mix. Provision of an inclined longitudinal construction joint would be an effective and practical crack prevention measure, which would facilitate monolithic performance of the dam and also be compatible with the construction schedule.

8. All materials used in a high RCC gravity dam, including cement , pozzolana and fine and coarse aggregates, should be similar in quality to those considered suitable for a comparable CMC dam. Particularly important are physical properties relatd to specific gravity, susceptibility to AAR, or excessive thermal expansion.

9. The RCC mix should be designed with the lowest cement plus pozzolan content necessary to obtain the desired workability and specified strength in compression and shear at prescribed ages and with the lowest practicable rise in temperature.

10. For high RCC dams, in addition to the longitudinal foundation treatment gallery, intermediate horizontal longitudinal and some transverse galleries near the foundations would be desirable to provide access to the interior of the dam. Such galleries would be highly useful for treatment of deep cracks, control of leakage, grouting of contraction joints if necessary, additional foundation treatment in the future and instrumentation.

11. Depending upon its location and size, openings in the body of a high RCC dam, including galleries, may need steel reinforcement to prevent cracking due to tensile stress concentrations.

12. A system of vertical drilled drain holes along and near the entire upstream face of the dam and discharging into the longitudinal galleries is the most effective means for keeping the body of the dam, including the construction joints, dry. The drain holes should be accessible for periodic redrilling.

13. Large spillways, with high crest gates, located over a high RCC dam, affect the stability of the dam. Integration of the spillway structures, particularly the piers supporting large gates, with the RCC dam, such that the latter is not overstressed, needs special consideration in design of the dam.

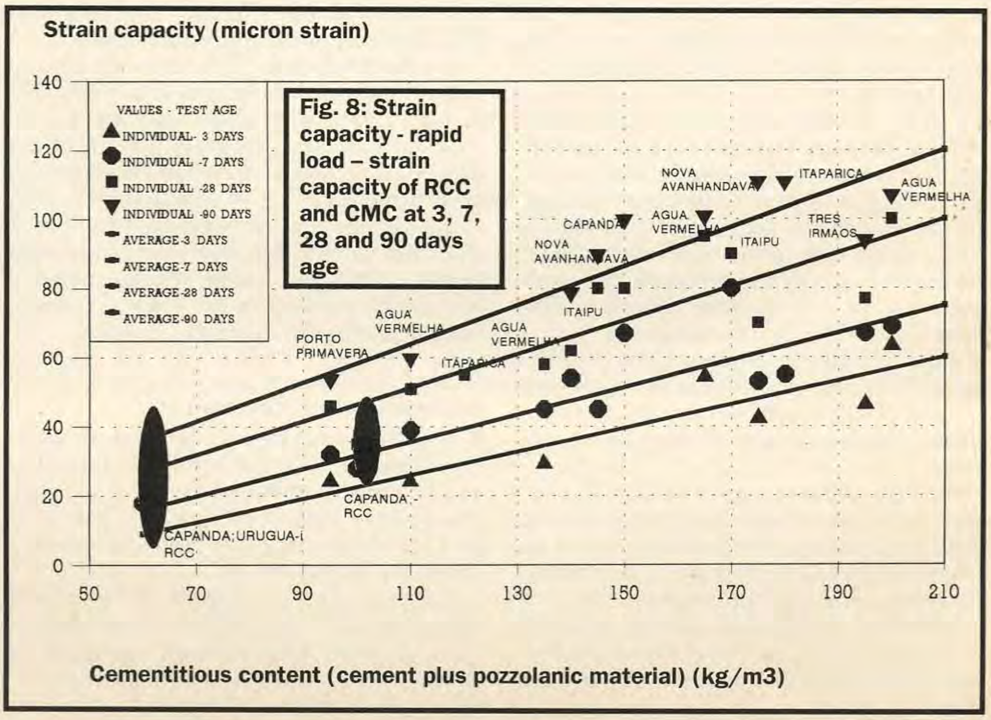

Fig. 8: Strain capacity - rapid load – strain capacity of RCC and CMC at 3, 7, 28 and 90 days age