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THE EFFECT OF DISCONTINUITIES ON THE ERODIBILITY OF ROCK IN UNLINED SPILLWAYS OF DAMS

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by

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ABSTRACT

Problems with erosion in unlined spillways of dams, have caused environmental and safety hazards, and necessitate the development of reliable scientific methods to assess the erodibility of the bedrock material.

In an attempt to analyze the causes and effects of different geological, geotechnical, hydraulic, hydrological and engineering design parameters on erosion of unlined spillways, a number of dam spillways in the R.S.A. were studied. All the parameters considered important for the erosion resistance of the rock were collected and correlated with hydraulic factors and observed extent of erosion.

Rock type, strength, weathering and uniformity of the geological conditions are important geotechnical considerations while velocity and energy of the water flow are the main hydraulic parameters. The main consideration of the study was the effect of the discontinuities. All the properties of the jointing of the rock mass, such as number of joint sets, RQD, joint spacing, joint separation, joint orientation, roughness and filling material, were surveyed. Joints weaken the rock mass and induce removal of the rock blocks if the joint properties are unfavourable to stability. The jointed bedrock is much less resistant to flowing or falling water, the result being an extensive and quickly formed scour.

Various rock mass classification systems have been applied and a reasonable correlation between rock class and extent of erosion was established by means of slightly modified Kirsten (1982) and Weaver (1975) rippability classifications.

A number of methods for the prevention and repair of erosion damage have been proposed.

OPSOMMING

Probleme met erosie in onbeklede damoorlope veroorsaak skade aan die omgewing en bedreig die veiligheid van strukture. Dit is derhalwe noodsaaklik dat wetenskaplike metodes ontwikkel word om die erodeerbaarheid van rots te kan voorspel.

In 'n poging om die invloed van verskillende geologiese, geotegniese, hidrouliese en ingenieursontwerp parameters op erosie van onbeklede oorlope te bepaal, is 'n aantal damoorlope in die RSA ondersoek. Alle parameters van belang vir die erosieweerstand van rots is versamel en met hidrouliese parameters en die waargenome omvang van erosieskade gekorreleer.

Gesteentesoort, sterkte, verwering en eenvormigheid van geologiese toestande is belangrike geotegniese oorwegings, terwyl die snelheid en energie van die waterstroom as die belangrikste hidrouliese parameters geïdentifiseer is. Al die eienskappe van die nate in die rotsmassa, insluitend die aantal naatstelle, RKW, naatspasiëring, naatwandskeiding, naatoriëntasie, ruheid en vulmateriaal, is aangeteken. Nate verswak die rotsmassa en kan lei tot beweging van rotsblokke indien die naateienskappe ongunstig is ten opsigte van stabiliteit. Die genate rotsmassa is minder bestand teen die effek van bewegende water en kan lei tot uitgebreide en snelvormende erosieskade.

Verskillende rotsmassaklassifikasiestelsels is toegepas en 'n redelike korrelasie tussen rotsklas en omvang van erosie is met behulp van effens gewysigde uitgraafbaarheidsklassifikasies volgens Kirsten (1982) en Weaver (1975) verkry.

'n Aantal metodes vir die voorkoming en herstel van erosieskade word bespreek.

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1.1 Scope of the study.

The design of spillways for dams requires that flood waters with high energy be passed without causing any damage to the structure or the area immediately downstream. This research is aimed at estimating the role of discontinuities on the rate and extent of scour and erosion of different rock types in unlined spillways of dams. Erosion and/or scour are processes that involve the removal of the in situ rock material through the action of water flow. They cause both environmental and safety hazards and while both are important, the consideration of safety is the main justification for further studies.

In order to make a dam structure or a spillway safe against damage or destruction by flood events, consideration should be given to

- the extent to which human life would be endangered by damage to or failure of the dam,
- the value of property that would be destroyed, and
- the inconvenience resulting from the failure.

Protection against erosion can be provided to some extent with concrete lined channels or stilling basins, but cost implications often necessitate the consideration of unlined spillways. The need for this research is illustrated by the number of dams with unlined spillways that had suffered severe headcutting and channel erosion in a way which threatened spillway structures, while at many dams, unlined spillway channels or plunge pools have performed successfully for many years. These cases demonstrate the necessity for a method to assess the erosion resistance of bedrock material downstream of the spillways of dams. The method of safety evaluation of a spillway should be applied not only to newly constructed dams but to the existing dams as well, so that the capacity of an unlined spillway and the need, if any, for precautionary measures, can be assessed.

In an attempt to analyse the causes and effects of erosion downstream of unlined spillways, geological, geotechnical, hydraulic, hydrological and engineering design parameters are

considered. This study is aimed specifically to
- develop better documentation of rock behaviour in unlined spillways,
- foresee the response of the area downstream of the spillway to the effects of water flow or impact,
- study the role of discontinuities in the rock mass on the extent and rate of erosion,
- indicate basic plans for remedial work.

1.2 Methodology and study area.

To achieve the overall objective of this study, i.e. to develop methods for detection, prediction, prevention and repair of rock erosion in unlined spillways, certain investigations have been undertaken. Some of the problems experienced and methods used are described in the following paragraphs.

Many spillways do not operate regularly and others have never experienced a flow. In addition, the variety of spillway types introduce a number of dissimilar hydraulic parameters. This complex variable nature of spillway flow makes it difficult to determine the extent, rate and mechanisms of erosion and its impact on structures and downstream areas.

Exploration data acquired during pre-construction and construction phases, for the detailed geological or geotechnical evaluation of the erodibility of some unlined spillways and specific published literature, on this particular theme, are often not available. Despite the above obstacles, relevant reports, articles and many references were used to advance the research.

Case histories of erosion in unlined spillways of dams in other countries provided methods to study and examine the process of erosion downstream of a number of selected spillways in the Republic of South Africa.

Site visits to dams in the R.S.A. are listed within groups, according to the similarity of their spillway operation:
- Unlined rock channel downstream of concrete overflow wall. (Kammanassie Dam, Donkerpoort Dam, Hans Strydom Dam, Marico

Bosveld Dam, Koos Raubenheimer Dam).

- Partially lined uncontrolled overflow spillway. (Bell Park Dam).
- Uncontrolled free fall spillway on arch dam. (Wagendrift Dam, Graigie Burn Dam, Roodeplaat Dam).
- Flip bucket structure spilling on unlined rock. (Goedertrouw Dam, Vygeboom Dam,Gamkapoort Dam).
- Gated side-channel spillway with partially lined concrete chute. (Hartebeespoort Dam).

The localities of the above dams are shown in Fig.1.1.

Poor weather conditions and spilling at some of the dams during the site visits, reduced the number of cases available for study.

For the compilation of research data, surveys of the geological and geotechnical properties of the in situ rock mass downstream of the spillways were conducted. These included the hardness of the rock material, by means of the Schmidt Hammer Test and the geological pick, and a study of the joint parameters e.g. number of joint sets, joint frequency, joint spacing, joint aperture, orientation of joints, weathering, filling material and joint wall roughness.

Classification of the rock masses, by means of different existing systems, was attempted and the results were compared. To overcome the differences between the classification ratings and observed erosion in the field, a new method for the evaluation of the rock properties and classification system has been developed.

Available topographic and geological maps, spillway plans and design information, as well as hydrological information - daily and monthly records, hydrographs - and initial research reports, were used.

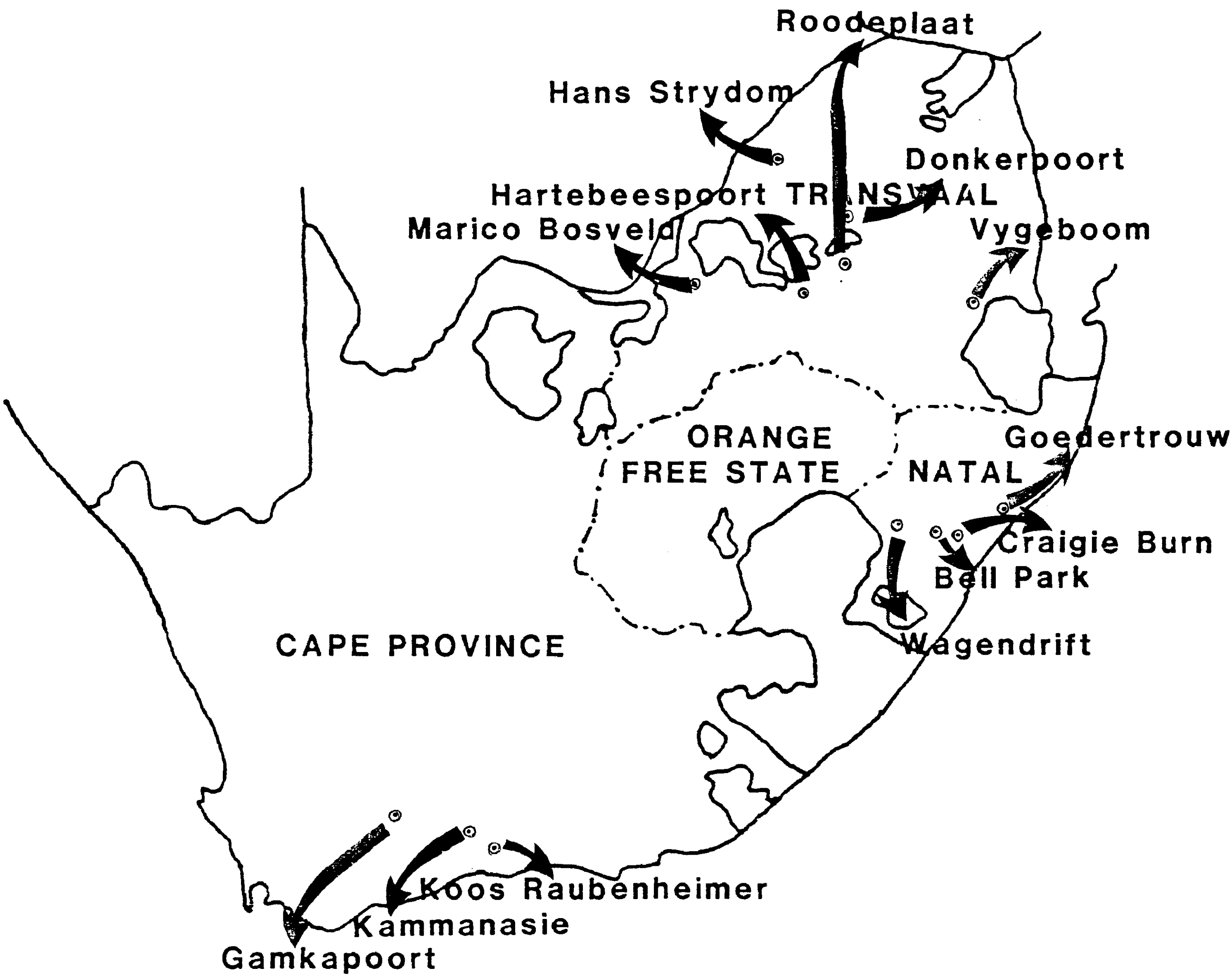


Fig.1.1 Locations of visited dams in the R.S.A.

CHAPTER 2 SPILLWAY TYPES AND EROSION PROCESSES.

2.1 Spillways.

A spillway is a structure that conveys flood water from the valley upstream of the dam to the valley downstream. The provision of adequate spillway facilities can in many instances pose more problems than the design of the dam itself. The freeboard and spillway capacity should be sufficient to ensure that the non-overspill crest of the dam will not be overtopped by a flood of a specified return period and that erosion at the foundation or the toe of the dam will not endanger the safety of the structure. The types of spillways most commonly provided at dams in the R.S.A. are described in the following paragraphs.

A spillway may be designed in the form of an overflow structure leading into a lined or unlined open channel of rectangular or trapezoidal cross section that permits direct flow of excess flood water from the reservoir to the downstream valley. This type of spillway is called a normal spillway. A side-channel spillway is a structure wherein the flood water spills over a wall which is perpendicular or at an acute angle to the dam axis and is carried past the dam by an open channel or tunnel running practically normal to the dam axis. In a shaft, morning glory or glory-hole spillway, the overflow water drops vertically or obliquely into a funnel and is conducted in a concrete pipe or tunnel generally located below the body of the dam (Krynine et al., 1957).

Passage of water through and over a concrete dam can be achieved in different ways. Where radial or sector gates are used, the spillway is controlled, while without gates, the spillway is an uncontrolled one. The crest of the dam wall has an ogee shape and such a spillway is called an overflow spillway. Free fall spillways with a lower plunge pool are associated with arch dams in narrow canyons. Free fall and overflow spillway crests are often provided with aerated splitters in order to dissipate the energy.

A spillway may release discharge water into a stilling basin, a chute, a water cushion or a plunge pool. A stilling basin is a deep basin located at the toe of the spillway structure and is designed to reduce the turbulence of the water flow and its velocity before it enters the river channel. Stilling basins are often provided with concrete lining and "dentates" for energy dissipation. Spillway chutes, either concrete-lined or unlined, transfer water farther downstream of the dam wall. Chutes under high velocity flow often suffer from cavitation or erosion damage. In case of a free overflow nappe which drops vertically on the river bed, a plunge pool is often formed by scouring.

For dissipation of the water energy into the air and protection against erosion at the toe of the dam, stepped energy dissipators, ski-jumps or flip buckets may be used, so that the discharged water lands at a safe distance from the toe.

Whatever arrangement is adopted, it is essential to consider not only the static loading of the dam but also the dynamic loading, the possibility of vibration in gates, the erosion resistance of the river bed below the dam, the type of debris likely to be carried in floods, the rate of change in river bed level downstream of the dam and the overall cost.

2.2 Scour and erosion.

Scour can be defined as the enlargement of a flow section by the removal of material through the action of the fluid in motion. Implicit in this definition is the fact that the moving fluid exerts forces on the material composing the boundary. Though the terms scour and erosion are practically synonymous in geotechnics, there is a little refinement of their usage i.e., scour of the bottom of the channel and erosion of the banks of the channel (Krynine et.al., 1957).

The term erodibility of rocks and soils is defined by Cameron et.al. (1986) as the quality, degree or capability of them of being eroded or yielding more or less readily to erosion. The scouring potential of a water jet may be described by its energy per unit area at the point of impact and to a lesser extent by its dynamic pressures over and above the bedrock threshold strength (Spurr, 1985).

A term used by Kohl (1968) is erosion strength, defined as the energy absorbing capacity of the material up to fracture under the influence of erosive forces. When the erosion strength is known, then the erosion intensity, defined as the power absorbed by the unit area of the eroded portion of the material, is given by

$$I_e = (i/t)\, S_e \tag{1}$$

where I_e is the erosion intensity (power/area), i is the depth of erosion (length), t is the exposure time (seconds) and S_e is the erosion strength (force/area).

According to Mason (1984) the erosion resistance, S_e(MPa), of the bedrock may be presented as follows

$$S_e = f\ [\sigma_c\ (RQD)] = I_r\ (T/D) \tag{2}$$

where σ_c is the uniaxial compressive strength, RQD is the rock quality designation, I_r is the threshold absorption or deflection power per unit area of the bedrock beyond which the rock will degrade, T(seconds) is the given period for the scour process and D(metres) is the depth scoured. He agreed with Kohl(1968) that the erosion strength of the bedrock may be characterised by the effective uniaxial compressive strength of the bedrock mass, which is obtained by using the empirical failure criterion suggested by Hoek (1983).

Erosion resulting in scour is caused by the impact of concentrated, high velocity flow, by turbulence as the water looses energy and slows to flow in a larger channel and by boundary shear between the soil particles or the rock blocks and the flowing water. The "initial movement of the bed", frequently called the critical condition or initial scour, is related to the impact of the water on the particles, the frictional drag of the flow on the rock particles and the lift force criteria based on the pressure differences due to the velocity gradient (Graf, 1971). Detachment is accomplished through the dissipation of the kinetic energy of flow, while particle transport is accomplished by the kinetic energy of the flowing water (Mathewson, 1981). The quantity of the material which the fluid can move or transport per unit time is termed the transport capacity of the flow.

The rate of the scour in a plunge area will equal the difference between the capacity for transport of material from the scoured area and the rate of supply of newly scoured material to that area. It will decrease as the flow section is enlarged. The limiting extent of scour will be approached asymptotically.

Initially, the depth of the scour is influenced by the erodibility of the bedrock or by the grading of loose material in the pool. Also the depth to which a plunge pool will be scoured is related to the height of the fall, the depth of the tailwater and the concentration of the flow. The increase of scour depth as a function of time is shown in Fig.2.1, using data from Kariba(1), Picoti(2) and Farchad(3) Dams. The equilibrium scour depth for a given magnitude of spillway discharge is the depth beyond which no significant increase will be caused by more spills of the same magnitude and of significant duration.

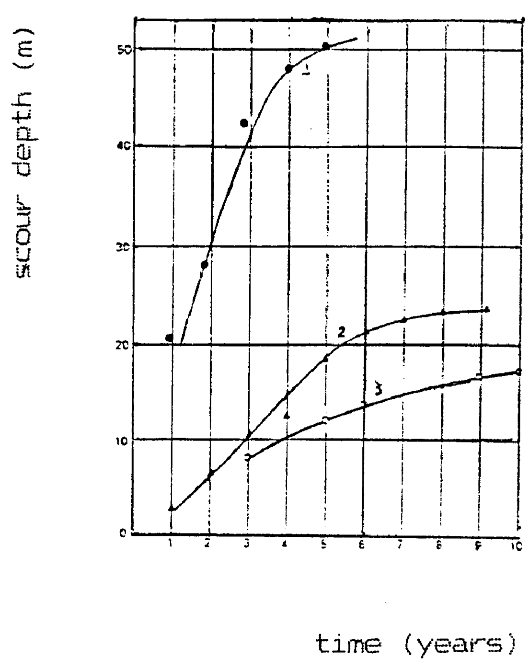


Fig.2.1 Scour depth as a function of time (Akhmedov, 1988).

According to Cameron et al. (1986) and Clemence (1988), one common type of erosion in unlined spillway channels is the knickpoint erosion. A knickpoint is an abrupt change in the slope of a channel, commonly formed in a stream channel where a resistant bed overlies a more erodible unit. Knickpoint erosion occurs when the capping rock is undermined by erosion of the lower strata and fails. Structural characteristics such as bedding thickness and joint spacing influence the knickpoint erosion.

Another phenomenon which causes erosion is cavitation due to the formation of vapor bubbles in a flowing liquid, where the local pressure is reduced below the vapor pressure of the liquid. When such cavities collapse, as the vapor condenses, destructive pressures may be created in the vicinity of the collapse. This leads to the erosion of material exposed to such collapsing cavities. Not even the hardest metals known can entirely resist this erosive force (Kohl,1968).

CHAPTER 3 LITERATURE STUDY ON THE ASSESSMENT OF SCOUR AND EROSION.

Previous attempts to calculate the extent of scour downstream of a spillway structure, were mainly based on hydraulic parameters such as discharge over the spillway, height of water fall, tailwater depth, width of spillway crest, wetted perimeter of the channel, cross-sectional area of the channel, the angle with which the water falls on to the impact area and the channel gradient. The different approaches for the estimation of scour may be simple or complex.

3.1 Spillway channels.

Woodward (1984), studied 14 spillways of dams in New South Wales, Australia, with respect to geological, topographical and operational factors. He suggested that spillways on rock masses with RQD values of less than 50 per cent and containing erodible seams should be provided with complete energy dissipators. If the RQD is greater than 50 per cent but without erodible seams, a flip-bucket structure would be a necessity. If the rock downstream of the spillway is durable and lowly stressed with RQD greater than 70 per cent without erodible seams, an unlined or a partially lined terminal structure would be safe enough to withstand the flows. Unlined spillways cut into rock are usually associated with rock mass fracture frequency less than 4 fractures per metre. In cases of gated spillways or where engineering structures such as road bridges are located across the spillway crests, and where the fracture frequency is greater than 4/metre and the rock is open-jointed with permeability greater than 5 Lugeons, the provision of a short concrete-lined chute downstream of the control structure is required.

Considering erosion of rock downstream of spillways, Reinius (1986) performed model tests to measure water pressures around a simulated rock block. Pressures caused by water propagation into cracks of the rock, act on the sides and bottom of a rock block whereas pressure from flowing water, acts mainly on the surface of the top fragments. The resistance to uplift force depends on the shape, weight of rock blocks and shear forces between adjoining blocks. Because of turbulence of the flow, the forces affect the joints and induce instability of the block. Rocks with planes or joints dipping downstream as well as rocks of poor quality with closely spaced, open joints, are more prone to erosion than rocks with upstream dipping planes.

The evaluation of information obtained from experimental and field data by Cameron et al. (1988), resulted in the identification of two criteria which are considered to be measures of the response of a channel to the hydraulic forces applied. These criteria pertain to the volume of the material eroded and distance of headward erosion. The volumetric ranking is defined as the ratio (x100) between the actual volume of erosion and the volume of erosion which could cause failure of the spillway, calculated by extending the actual eroded gully upstream to the spillway wall. The horizontal erosion ranking is expressed as the ratio (x100) between the horizontal erosion (length of the eroded area) and the distance between the end of the eroded channel and the spillway wall. These parameters show how imminent the erosion threat to a particular dam is, and may be used to give priority to remediation.

These two criteria exhibit stronger correlation with the geometric parameters (inflection points or knickpoints in the channel, channel width, excavated length, length of steep section, gradient of channel and steep section, elevation drop from the end of the excavated portion to the flood plain) than with the hydraulic parameters (peak flow, cumulative flow, hydraulic attack, flow duration, flow depth, maximum velocity).

An index of erodibility has been proposed by Cameron et al. (1988) for cases where erosion occurred in emergency spillway channels in the U.S.A.. The Erosion Probability Index (EPI) is based on the concept that the key geotechnical factors controlling erosion during spillway flow are contained in the rock mass rippability and the lithostratigraphic continuity and that these factors can be assessed in a semi-quantitative manner. The Geotechnical Erosion Probability Index (EPI_g) is assessed by using rippability (E_r) and continuity (E_c) parameters, as indicated in Table 3.1. This table is based upon the parameters used in the rippability rating chart by Weaver (1975) but with slightly different ratings. The Hydraulic Erosion Probability Index (EPI_h) is determined from hydraulic studies in the laboratory. Finally, the EPI is calculated as follows:

$$EPI = EPI_g + EPI_h.$$

GEOTECHNICAL EROSION PROBABILITY INDICES (EPI_g)	
RIPPABILITY PARAMETERS (E_r)	RATINGS
Rock mass parameters	
Rock hardness	0 - 10
Rock weathering	1 - 15
Joint spacing	5 - 30
Joint continuity	0 - 5
Joint separation	1 - 5
Strike/dip	3 - 15
Seismic P-wave velocity	5 - 20
Total rating	15 -100
CONTINUITY PARAMETERS (E_c)	RATINGS
Vertical continuity (bed thickness)	5 - 15
Lateral continuity	5 - 25

Table 3.1 A method for calculating the geotechnical EPI ($EPI_g=E_r+E_c$) (Cameron et al., 1988).

3.2 Free fall spillways.

Free trajectory jets, formed by free overfalls over the crests of concrete dams, discharge under pressure from gates set in the dam wall or flows deflected from some point below the non-overspill crest, enter the tailwater and their energy diminishes with depth of plunge. The jet produces dynamic pressure fluctuations on the floor and walls of the plunge pool, which may be transmitted into and along fissures in the rock, causing it to break up into blocks which are swept away by the flow, until an ultimate scour depth is reached.

Blaisdell, Anderson and Hebaus (1981), for their research on scour at cantilevered pipe spillway exits, described a hyperbolic logarithmic mathematical method for determining the ultimate depth of the scour.

Laursen (1952), in his paper on the nature of scour, indicates that the nett rate of scour is equal to the difference between the transport capacity of the flow and the rate of supply of material from the floor and sides of the plunge pool.

Spurr (1985) suggested that differences in phase between the turbulent hydraulic pressure fluctuations, which act at the bedrock surface and those in the bedrock itself, develop differential pressures within the fractured mass.

Akhmedov (1988) divided the development of a scour hole into three stages, according to the relation between the hydrodynamic force of the flow (P), the rock fragments' weight in the water (G), and the cohesive force between the fragments and the surrounding rock (Pc).
In the first stage, the destruction process is mainly influenced by jointing of the rock. This stage involves the removal of the rock fragments and the relation that governs that case is $P > G + Pc$.
The second stage can be represented by $P \geq G + Pc$, during which the scour hole depth decreases as the bedflow energy decreases, but it is still large enough to cause vibration-induced hydrodynamic pressure fluctuations. If the rock mass is highly fractured and the force of gravity is approximately equal to the cohesive forces, the character of the rock erosion is similar to the scouring of unconsolidated coarse-grained material.
As the rock is agitated by flow velocity pulsations, abrasion of the faces of the cracks, widening of the cracks and reduction of the sizes and weights of fragments is caused, so that the movement of each fragment increases. Eventually, during the third stage, the flow can no longer disturb the rock fragments from their equilibrium condition and $P < G + Pc$.
Scouring continues by using the previously separated fragments as tools of erosion, steadily abrading the walls of the scour hole. The rock strength is important, because the susceptibility of rock to abrasion is inversely proportional to its strength.

Because local scour is a three-dimensional phenomenon, in which the air entrainment and the characteristics of the rock mass play an important role, it is very difficult to predict the extent of scour by a free-falling jet. Martins (1973) suggested that the effect of air entrainment on the scour capacity of jets should be taken into account, on the side of safety, in calculating the scour depth, by reducing the latter by 25 per cent in case of high air entrainment, or by 10 per cent in the case of intermediate air entrainment.

Mason and Arumugam (1985), in trying to analyze the process of scour and calculating the scour depth, gathered and put into five groups all the existing formulas for scour estimation (Table 3.2).

GROUP	FORMULA BY WRITER	YEAR
I	Schoklitsch	1932
	Veronese-(A)	1937
	Veronese-(B)	1937
	Eggenburger	1944
	Hartung	1959
	Franke	1960
	Damle-(A)	1966
	Damle-(B)	1966
	Damle-(C)	1966
	Chee and Padiyar	1969
	Bisaz and Tschopp	1972
	Chee and Kung	1974
	Martins-(B)	1975
	Taraimovich	1978
	Machado	1980
	SOFRELEC	1980
	INCYTH	1981
II	Jaeger	1939
	Martins -(A)	1973
III	Cola	1965
	Davis and Sorensen	1969
	Hartung and Haustler	1973
IV	Mikhalev	1960
	Rubinstein	1965
	Solovyeva	1965
	Yuditsky	1965
	Mirtskhulava-(A)	1967
	Mirtskhulava-(B)	1967
	Mirtskhulava-(C)	1967
	Zvorykin et.al.	1975
V	Thomas	1953

Table.3.2 List of authors who proposed various formulas for calculating scour depth (Mason et al., 1985).

Group I contains the equations that express the scour depth D(m) in terms of the head drop from upstream to downstream water level H(m), the unit discharge of the jet at the point of impact $q(m^3/s/m)$ and the characteristic particle size of the bed material d(m). The general form of this equations is:

$$D = (K\ q^x\ H^y)\ /\ d^z \qquad (3)$$

where K, x, y, z are coefficients for any given formula with average values about 0,6 for x, values varying from 0,2 to 0,3 for y and values ranging from 0,0 to 0,5 for z. The values for the constant K suggested by different authors vary from 0,5 to 2,76.

The group II formulas express the scour depth D(m) in terms of $q(m^3/s/m)$, H(m), and d(m) and the tailwater depth h(m). One equation is as follows:

$$D = 0{,}6\ q^{0{,}50}\ H^{0{,}25}\ (h\ /\ d)^{0{,}333} \tag{4}$$

and another form is:

$$D = 0{,}14N - 0{,}73\ (h^2\ /\ N) + 1{,}7h \tag{5}$$

where

$$N = [(Q^3\ H^{1{,}5})\ /\ d^2]^{1/7} \tag{6}$$

The group III formulas can be classed as highly simplified:

- Cola (1965) claims for wide sheet flow, maximum scour depth will be 40 times the width of the jet;
- Hartung and Hausler (1973) suggest that for a concentrated circular jet, the scour depth will be 20 times the jet´s diameter;
- Davis and Sorensen (1969) suggest that the scour depth will be 2/3 of the height of the fall.

The group IV formulas comprise those by Russian authors and are generally more complex than those proposed by others

- Mikalev (1960): $$D = \frac{1{,}8049\ \sin a}{1 - 0{,}215\ \cot a}\left(\frac{1}{d_{90}^{0{,}33}\ h^{0{,}30}} - \frac{1{,}128}{H}\right) \tag{7}$$

- Rubinstein (1965): $$D = h + 0.19\left(\frac{H + h}{d}\right)^{0.75}\left(\frac{q^{1{,}20}}{H^{0{,}47}\ h^{0{,}33}}\right) \tag{8}$$

- Mirtskulava (1967): $$D = \left(\frac{0{,}97}{d^{1/2}} - \frac{1{,}35}{H^{1/2}}\right)\frac{9\ \sin a}{1-0{,}175\ \cot a} + 0.25h \tag{9}$$

where D(m) is the maximum scour depth, d(m) is the particle size of the bed material, H(m) is the height difference between the upstream and downstream water surface, h(m) is the tailwater depth and a(degrees) is the angle with which the falling stream enters the downstream area.

Group V was reserved for formulas developed using time as a parameter. Experiments were undertaken, considering the rate of scour rather than the ultimate depth and the concept that scour was proportional to the geometrical progression of time and, as such, a final equilibrium depth could not be expected. The scour equation produced by Thomas (1953) for long-term, ultimate scour depth, for widely graded bed material is as follows:

$$D = h + \frac{2h}{3}\left(\frac{q}{H\ Wm}\right)^{2/3}\left(\frac{H}{h}\right)^{2(q/H\ Wm)^{1/6}} \qquad (10)$$

in which Wm is the fall velocity of the water.

After testing experimentally most of the above formulas and analyzing scour data from prototype dams, Mason et al.(1985) suggested the following general equation as the most accurate for estimating the depth of scour under a free falling jet:

$$D = k\,\frac{q^x\ H^y\ h^w}{g^v\ d^z} \qquad (11)$$

where h is the tailwater depth,(m),

d is the bed particle size,(m), where the mean particle size is more appropriate than D_{90} and is usually taken as 0,25,

$k = 6{,}42 - 3{,}10\ H^{0,10}$

$v = 0{,}30$

$w = 0{,}15$

$x = 0{,}60 - H/300$

$y = 0{,}05 - H/200$

$z = 0{,}10$.

According to Mason et.al. (1985), for permanent scour to take place, three conditions must be met:

- there must be sufficient jet power at a base or sides of the hole to remove the bedrock;
- there must be sufficient jet power throughout the hole in general to enable scoured material to be lifted to the top;
- there must be sufficient power at the top of the hole to enable material to be ejected sufficiently clear of the hole so as not to fall back again.

3.3 Spillways with flip buckets.

Calculations for maximum scour depth, d_s(m), downstream of flip buckets, proposed by Locher and Hsu (1984), follow the general form:

$$d_s = (C\ q^x\ H^y\ a^w)\ /\ d^z \tag{12}$$

where d_s(m) is the maximum scour depth, q(m^3/s/m) is the discharge per metre at the crest of a drop structure or at a flip bucket exit, H(m) is the height drop from upstream to downstream water surface, a(degrees) is the angle that the flip bucket makes with the horizontal, d(mm) is the particle size of rock material, C is a coefficient and w,x,y and z are exponents. Application of the previous equation (12) by many authors, shows that the ultimate scour depth is primarily a function of unit discharge rate (q). The head (H) plays a secondary role and the particle size (d) and the lip angle (a) are less significant (Locher et al.,1984).

From a geological point of view, it is clear that interlock and cohesion of strong rock blocks in a rock mass with widely spaced joints is not taken into account in any of the above formulas.

CHAPTER 4 CASE HISTORIES FROM OTHER COUNTRIES.

Since 1970, various authors have studied the problems of erosion in unlined spillways of dams. The available data base from cases in other countries, where surveys, discussions and proposals have been made, are important to the current research. Comparisons between the cases could be made and information for the evaluation of results could be obtained.

The aspects of erosion for every case from the literature are listed in groups according to the country to which the dams belong. Depending on the available references, some of the cases are described in brief and others in detail, but the attempt is to include information about the engineering design, the geological conditions, the hydrological history, the observed erosion and finally the remedial work.

4.1 Portugal.

PICOTE DAM-1958 (Mason,1984).

This 100m high arch dam includes gates with a discharge capacity of 11 000m^3/s, a tapering chute and a flip bucket.

The latter concentrates flow downstream into a narrow canyon in excellent granite.

After a flood in 1962, a pit 20m deep had been formed in the granite, causing a 15m high bar of eroded material downstream which raised tailwater levels locally and reduced power output.

Remedial works included extension of an existing diversion tunnel, so that flows could bypass the plunge pool and bar.

4.2 New South Wales - Australia.

BURRINJUCK DAM (Woodward,1984).

The partially lined, gated spillway includes no dissipators and has a unit discharge capacity which exceeds $200m^3/s/m$.The head difference between full supply level and bed level is about 65 meters.

The dam is located on very good granite with 0,3-5,9 fractures per meter.

During a major flood in 1974, the spillway experienced a unit discharge of $102m^3/s/m$, (49 per cent of design flood outflow) resulting in scour that destroyed a penstock to a downstream power station.

COPETON DAM-1976 (Woodward,1984; Thomson,Woodward,1980).

The dam is a 113m high earth and rockfill structure on the Gwydir River, near Inverell, in northern N.S. Wales. The spillway consists of a 156m wide concrete ogee, supporting 9 gates. The drop from the design flood level to the river bed downstream is 130 metres, with an average overall gradient of 1 in 4. The chute downstream is lined for 55 metres. The design outflow capacity is 14 $800m^3/s$.

The structure is located in predominantly coarse-grained granite but more deeply weathered than in the general area. Some scour potential at the downstream end of the chute was realised and this was one factor favouring the nine gates, rather than a lesser number.

In 1976, the first discharge of $460m^3/s$, scoured a narrow - 10m wide - but unusually deep channel, along the existing gully downstream of the concrete chute. In addition to the main scour channel, the floods also produced some minor undermining of the backfill concrete at the downstream end of the chute. For a long period following the cessation of flood discharge, the rock in the main scour channel showed significant deterioration, due both to continued rock-popping failure in the floor of the channel and the opening up on exposure of very extensive, vertical, laumontite ($Ca(Al_2Si_4)O_{12}.4H_2O$-zeolite group) coated joints in the rock forming the walls of the scour channel. The unweathered granite in the floor of the scoured channel had a high scour resistance before fracturing due to stress failure, but once the failure occurred, the rock slabs could be removed by even the smallest flow. The quantity of the rock scoured by this mechanism during a flood event would thus depend more on the duration of the flood rather than on the peak discharge, in contrast with the normal (not stress-related) scour of resistant rock, where the flow is capable of scouring rock for only a relatively short period at the peak of the flood, and the quantity of removed rock depends mainly on the peak discharge.

The major factors whose combined effect was responsible for scour can be summarised as follows:
spillway discharge flowed directly into an existing gully; the topography and geology of the gully area permitted the developent of the initial, notch-like, scour channel; high in situ, horizontal compressive stress of 15-20 MPa, in the near-surface rock in the spillway area; the natural stress concentration effect on the original gully profile probably meant that the pre-existing stress relief joints, parallel to the topography, were more intensely developed beneath the gully than elsewhere. However, the Copeton spillway site had a high scour potential due principally to the high head and steep slope of the discharge path. The high in situ rock stresses and unfavourably orientated geological structures were additional factors favouring erosion.

Remedial works included a training wall between the fifth and sixth gate, so that four gates could be operated as a service spillway - because of the massive rock mass in that area, and the five remaining gates could be operated as an emergency secondary spillway. In addition to the training wall, concrete lining of the upstream half of the scour channel floor, of the right hand wall of the spillway cut downstream of the chute and of areas within the service spillway immediately downstream of the chute, was undertaken.

4.3 Zambia/Zimbabwe.

KARIBA DAM-1962 (Mason, Arumugam,1985; Mason,1984).

The dam is a double curvature, mass concrete arch 130m high, 620m long along the crest, located on the Zambesi River. The two additional spillway openings besides the four flood sluices, brought the total flood capacity to 9 400m^3/s.

The openings discharge free jets on an area immediately downstream of the dam and even with the rock being sound gneiss, erosion was considered inevitable. For this reason, the structural arch of the dam wall was thickened at mid height to attract load laterally to the abutments and thus reduce the cantilever stresses at the base of the dam.

By 1967, the plunge pool, developed by annual spilling from the sluices, reached a depth of 50m below river bed level and almost 400 000m^3 of rock was removed. By 1985, the scour hole extended 70m below the river bed level. A more immediate threat to the stability of surface material proved to be the spray associated with discharges from the flood gates. Spilling often took place for several months at a time and it was estimated that the disseminated spray effects arising from the spilling cloud could amount to as much as 100mm/day, over a large part of the abutments.

As a result of the plunge pool development and surface bank slides, the 1967 Consultants' review recommended the construction of a bypass spillway to reduce the discharge requirements on the main flood openings. This spillway has not been constructed.

4.4 Pakistan.

TARBELA DAM-1975 (Mason,1984; Lowe III,1982; Lowe III et.al.1979)

The 143m high embankment dam is sited on River Indus, in the Himalayan Foothills of northern Pakistan. Each year, considerable quantities of water are passed downstream via the two spillways. The service spillway, with a capacity of 18 600m^3/s consists of a gate-controlled ogee weir, a chute 530m long at a 2:1 slope and a flip bucket. An auxiliary spillway with much shorter chute and a flip bucket at its downstream end is designed to provide an additional capacity of 24 300m^3/s.

Both spillways are sited in areas of weak siliceous limestones and limestones interbedded with phyllites. The service spillway pool has the added complication of a band of hard dark igneous rock running obliquely across the downstream end.

The Tarbela spillways (117m from headwater to tailwater) are subjected to high flows in July and in August, during which time snow melt from the headwaters in the Himalayas, causes flow in the 6 000 to 18 000m^3/s range. During spillway operation over the six years since completion, the jets from the buckets scoured plunge pools downstream of both spillways. Spilling via the service spillway commenced in August 1975. Lateral erosion of the plunge pool occurred mainly on the right flank, and developed dramatically on 12 June 1976, with the partial collapse of the steep slope adjacent to the pool. Approximately 400 000m^3 of material was displaced. The non-uniform erosion of the generally weak rock area, and the natural backward action on either side of the pools, caused them to erode towards the flip bucket foundations, and to endanger them partially.

Remedial works were embarked upon which lasted for several years and included stabilization of high slopes around the plunge pool to avoid major slope failures; post-tensioning the bucket structure into the rock immediately upstream, to ensure stability in the event of undermining; lowering the level of the igneous intrusion at the end of the plunge pool in the hope of reducing the return currents on the sides of the pool; lining the sides of the pool with massive walls of rolled concrete incorporating drainage galleries and stressed anchors.

4.5 U.S.A.

ALDER DAM-1945 (Mason,1984).

The 100m high dam has a spillway, situated on its left bank. The spillway, with a flip bucket at its end has a 2 265m^3/s capacity.

Flood water discharges on an area of blocky andesite.

The maximum recorded outflow until 1953 was 566m^3/s. The extensive use of the spillway resulted in the erosion of a plunge pool, about 30m*45m*24m deep, downstream of the bucket.

Remedial work carried out in 1952, comprised of grouting and installation of anchor bars. A wide fault zone, leading back from the plunge pool to the bucket, was backfilled with concrete and a barrier of rock, separating the plunge pool from the main river, was also grouted and anchored. A weir was built across the barrier to add to stabilization and to maintain the water level in the plunge pool.

SAYLORVILLE LAKE DAM-1975 (Cameron et al.,1986).

It is a 31,5m high and 2 025m long earth dam, located on the Des Moines River, in Polk Country, Iowa. The spillway is an uncontrolled gravity concrete ogee weir, 129m wide, with 60m of paved chute and approximately 1500m of unlined trapezoidal chute.

The unlined spillway channel is underlain by gently dipping, indurated shales, calcareous siltstones, thin limestones, coals and sandstones. Overlying unconsolidated glacial and aeolian deposits flank the spillway both to the east and west.

The poorly cohesive sedimentary rocks, the lack of lithological and stratigraphical continuity of the sedimentary rocks and the narrowing of the downstream portion of the channel focusses flow and accelerates channel bank and bottom degradation. These are the main factors that lead to a dramatic "stair-step" erosional landscape, with up to 9m of local relief, in the downstream channel, during the period 18 June to 3 July 1984, when a peak flow of $476m^3/s$ (which is 90% of the spillway capacity) reached velocities of approximately 2,25m/s.

The remedial work recommended, included filling, levelling and seeding all the irregularities in the sod-covered reach of the flat channel, maintaining the erosion resistant grass and soil cover and filling the upper end of the erosion gully with lean concrete.

LAKE BROWNWOOD DAM-1932 (Cameron et al.,1986).

The dam is located on a tributary of the Colorado River, in Texas. It is a rockfill embankment 546m long, 39m high. The uncontrolled broad-crested spillway is 144m long and about 0,6m high, and discharges water in an unlined channel, 195m long and 141m wide, located 240m left of the left abutment of the dam.

The channel consists predominantly of soft to moderately hard, multicoloured shales and $\pm$6m of thick sandstone, above the base of the shales. On top, a 36m thick formation of two resistant limestone sequences, each separated by 9m of shale with sandstone interbeds, occur.

In July 1932, during a relatively small flood event of $364m^3/s$, considerable erosion of the channel occurred. Successive overflows, eroded the shales preferentially and caused headward migration of several knickpoints. The relative lack of jointing or other structural discontinuities in the resistant units of these formations, as well as their good lateral stratigraphic continuity, may partially explain the relatively slow rate of headcutting in the spillway channel.

A series of recommendations, regarding the repair of the spillway channel, included excavation and reshaping of the eroded area; protection of the vertical surfaces of exposed shale beds with shotcrete and wire mesh; usage of drainage material or pipe drains through shotcrete, where required, and leaving existing boulders in place on ledges in the discharge channel.

GRAPEVINE DAM-1952 (Cameron et al.,1986).

The Grapevine embankment dam, located in Terrant and Denton counties, Texas, is 3 855m long, with a 8,4m wide crest, and is 41,1m high above the stream bed. The dam has a conduit outlet, controlled by electrically operated sluice gates. The spillway is an uncontrolled off-channel chute-type concrete structure, 150m in length, 32,7m high, and of 5 348m^3/s maximum discharge capacity.

Bedrock strata dip gently southeastwards, at an average rate of about 2 percent. Minor jointing and fracturing is present in the rocks underlying the spillway discharge channel. The strata consist of alternating beds of variable thickness and continuity, soft to moderately hard, fine grained, weakly to moderately cemented sandstone, soft carbonaceous shale and occasional thin seams of hard sandstone.

A flood event in 1981, the second in the dam´s history, lasted 21 days and the discharge of 254,8m^3/s (50 per cent of design capacity) produced severe headward erosion in the channel and a rugged erosional landscape was formed with up to 9m of local relief.

The factors that lead to the development of the scour are the lack of dissipation structures at the toe of the spillway; the substantial gradient change in the spillway channel; a pre-flood gully which incised the spillway channel downstream of the sharp gradient change; relatively soft, weakly cohesive sediments characterized by poor lithologic and stratigraphic continuity. In addition, a paved road crossed the spillway channel in close proximity of the structure, which dammed the spillway overflow and contributed to the initiation of knickpoint erosion in the channel.

Remedial action involved the construction of a concrete chute and stilling basin on the downstream toe of the spillway, placement of excess excavation on the flood plain section of the main embankment to help prevent sliding, and relocation of the road.

LEWISVILLE DAM (Cameron et al.,1986).

Lewisville reservoir is 18,52km from Grapevine Lake. Their spillways are quite similar.

The channel downstream of the Lewisville spillway consists of rocks of the Cretaceous shale unit with excellent durability and lithologic continuity. In addition to the latter, the smooth, gently sloping channel and wide, flat channel bed contributed to less severe erosion, than in Grapevine spillway, during the flood of 1981.

At Lewisville, the flood lasted from 15 October to 10 November 1981. The grass and soil, lining the channel, and the weathered shale "skin" of minor extent at the toe of the apron, were removed during the flow.

This minor damage would be corrected by construction of a concrete slab downstream to inhibit undercutting of the apron during future spillway discharges.

BLACK BUTTE DAM-1963 (Cameron et al.,1986).

It is an earthfill dam, located on the Stony Creek River. A flat, reinforced concrete slab was constructed at the channel floor crest with channel gradients, of 0 per cent and 9 per cent upstream and downstream of the crest respectively.

The spillway channel consists of basalts, interbedded with pyroclastic material. In the lower portion of the channel the lithology changes from basalt to the underlying sediments.

The channel flowed for 66 hours during March 1983. The maximum height of water above the crest was 0,81m and the maximum spillway discharge was 45,92m^3/s. Because of the highly hetereogeneous structure of channel rock, with cooling cracks varying in intensity from place to place, the steep downstream channel gradient and the narrow width of the channel, the channel downstream of the crest experienced moderate erosion, which did not endanger the structure.

SALINAS DAM-1940 (Cameron et al.,1986).

It is a thin wall arch structure with the spillway adjacent to the dam on the right abutment and a curved concrete apron which is superelevated to distribute high water flows and which terminates at a lip. The spillway overflows one to three times per year.

The rock mass is massive, Cretaceous sandstone with very few shaly interbeds and the rock surface is a dip-slope, dipping south-eastwards, roughly parallel to the lower portion of the apron.

After high flows of 1969, cumulative erosion damage occurred, particularly along the right rock bank.

A remedial program of rock bolting, wire-mesh installation, guniting and placing of mass concrete in the rock floor was carried out, in late 1969. Since the repair, gunite has held up against the erosion. The wire-mesh is torn in places and some of the mass concrete in the floor has been eroded, but the spillway and the channel appear to be in excellent condition.

BLUE RIVER DAM (Cameron et al.,1986).

It is a rockfill embankment, with an impervious earth core, a regulating tunnel, an intake tower, a gated concrete spillway with maximum discharge capacity of $880m^3/s$ and a stilling basin.

The spillway channel is excavated in hard to moderately hard andesite. Joints, shear zones, fractures and zones of hydrothermal alteration highlight discontinuities in this highly variable and complex rock.

The spillway had been used for a period of 25 days, at a discharge of 72,8 to $78,4m^3/s$. Because of the lithologic heterogeneity, structural discontinuities and high design velocities, severe erosion occurred at the discharge end of the channel.

The recommendations were the removal of small trees from the spillway and the placing of concrete or asphalt lining in the channel to prevent further erosion.

TENKILLER FERRY DAM (Cameron et al.,1986).

The spillway structure of the dam is a gated weir with a downstream apron and flip bucket.

Massive jointed sandstone, underlain by shale and thinner sandstone units form the foundation of the structure.

A spillway outflow, in 1957, produced considerable erosion of this rock. Only minor flows have occurred since 1957 and none of these has damaged the structure.

WISTER DAM-1949 (Cameron et al.,1986).

This dam has an uncontrolled overfall spillway structure.

The spillway channel is underlain by interbedded sandstones and shales, that dip downstream at approximately 30 degrees.

It has operated three times. During the last two overflow events, it flowed for 10 days with peak flows of 62,26m^3/s. Loose fill with vegetation was removed and rock eroded along both concrete layback walls and in the middle of the channel. Erosion took place by differentially lifting and stripping thin sedimentary strata along weak bedding planes and fractures.

LAUREL RIVER DAM (Cameron et al.,1986).

The dam has a spillway with an "inverted-U" shape. This design utilizes aprons in the downstream and upstream surfaces of the spillway sructure. The sill is capped by a reinforced concrete slab.

The channel is situated in massive sandstone, underlain by an alternating sequence of thinly bedded sandstones, shales and massive sandstone. A knickpoint is formed, where the upper durable sandstone is being undercut by erosion of the poorly cohesive shale unit. The rate of erosion is constant, 0,9m/year, and it will not threaten the spillway structure in the near future.

CHAPTER 5 FACTORS AFFECTING ERODIBILITY OF THE ROCK MASS.

5.1 Introduction.

It is generally acknowledged that flood events, resulting in high discharges through dam spillways, can induce erosion in the downstream area.

Scour within and downstream of unlined spillways of dams has been discussed by many authors. Some of them looked only at the characteristics of the water discharge, while others added, in their research, the influence of the geometry of the area bearing the water impact, and few others determined geological and geotechnical properties and their ratings to be used for the classification of the rock mass. Attempts to predict scour, either in unlined channels, downstream of overflow spillways, or in pools developed downstream of free-fall jets, from the characteristics of the water jet alone and unrelated to the local geology and spill duration (Table 5.1), are not representative of the real situation where the observed erosion depth is less than the calculated one.

Thus, a variety of factors control the response of an unlined impact area, within and downstream of spillways. These factors are geological, geotechnical, hydraulic and/or geometric and of engineering design. They are common in all types of spillways and only their individual contributions may vary. The factors are grouped into two large categories; one includes the geological factors and the other includes the hydraulic factors (Table 5.2).

When a jet discharges along the downstream face of the spillway wall (Fig.5.1), jet velocities of up to 50m/s may occur and produce such shear stresses on the floor that, with time, even the best concrete and rock are attacked, abraded and eroded on the surface. With material of suitable strength and durability, however, a longer spilling time is necessary before substantial abrasion and scouring take place. Erosion occurring in the spillway channel is from a geomorphological standpoint similar to that which may occur in natural stream channels where a knick-

PARAMETERS	Free Falling Jets			Flip Bucket Structures	
	WAGENDRIFT	CRAIGIE BURN	ROODEPLAAT	GOEDERTROUW	VYGEBOOM
$q(m^3/s/m)$	687,1/120	366,5/121,9	961/143	589,5/160	129/41,5
H(m)	35	35,3	50,4	10,5	42,3
d(m)	0,300	0,250	0,300	0,250	0,200
a(degrees)				33	40
h(m)			1,5-2,3		
$g(m/s^2)$	9,81				
k or C	$6{,}42-3{,}10H^{0{,}10}$				
v	0,30				
w	0,15				
x	0,60-H/300				
y	0,05+H/200				
z	0,10				
D(m)	11,66	8,71	8,58	11,37	17,37

Table 5.1 Calculations of the maximum scour depth during peak outflow; where q is the unit discharge; d is the particle size (otherwise the average joint spacing); a is the angle that the flip bucket exit makes with the horizontal; h is the tailwater depth; k or C is a coefficient; v,w,x,y,z are exponents; D is the maximum scour depth equal either to $k\ q^x\ H^y\ /\ d^z$ for free falling jets, or to $k\ q^x\ H^y\ h^w\ /\ d^z\ g^v$ for free falling jets with tailwater (Mason et al., 1985), or to $C\ q^x\ h^y\ a^w\ /\ d^z$ for spillways with flip bucket structures (Locher et al., 1984).

GEOLOGICAL/GEOTECHNICAL FACTORS	HYDRAULIC PARAMETERS
rock type	maximum spillway capacity
structure and texture	height of water fall
dry density	length of the spillway crest
void ratio	downstream slope of the spillway
natural moisture content	radius of the arch wall
permeability	characteristics of energy dissipators
natural or primary stress	roughness of the downstream area
weathering	in case of a channel
uniformity of geological conditions	width
jointing	length
number of sets	slope gradient
j.spacing-minimum and average	wetted perimeter
j.separation	cross sectional area
j.roughness	in case of a pool
fill type and properties	cross sectional area
j.orientation	radius of impact area
hardness/strength	depth of tailwater
cohesion	peak outflow
friction angle	discharge volume
	duration of maximum outflow
	height of water above spillway crest
	angle of water fall
	velocity and energy of water

Table 5.2 Factors determining erodibility of the rock mass below spillways.

point erosion developes, either through a lowering of the base level or channelization (Cameron et.al., 1988).

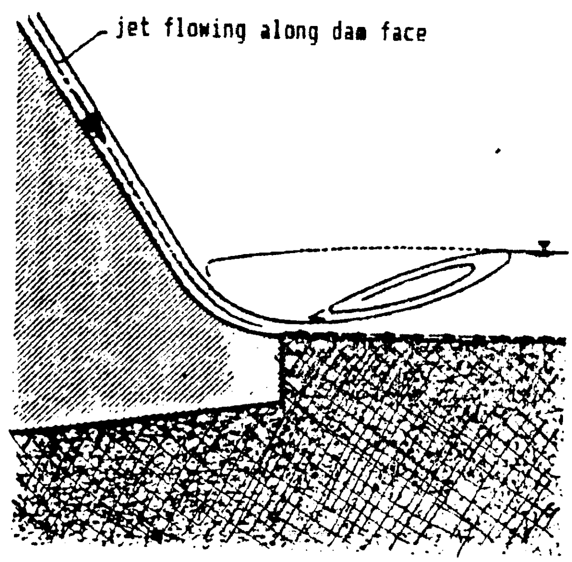


Fig.5.1 Water discharge along the face of a spillway wall into a channel (Hartung et al., 1973).

In the case of a free-falling jet (Fig.5.2), shear forces, causing abrasion and erosion on the downstream area, occur due to the deflection of the jet on the floor. Also as a result of the deflection of the jet, additional high dynamic pressures are created. The maximum values of these pressures increase according to the drop height of the jet. These dynamic pressures significantly increase the scouring, when they are transmitted through the cracks in the bedrock and build up beneath the rock surface (Hartung et.al.,1973).

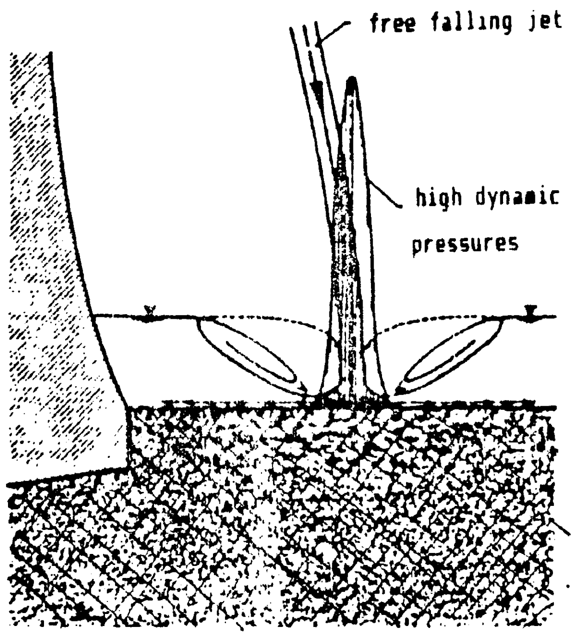


Fig.5.2 Free falling jet in the area downstream of a spillway (Hartung et al., 1973).

For a free-fall spillway, below which a plunge pool may develop, the type and the size of the material play a role in the initial scouring (Spurr, 1985). The more fractured the rock, the more rapid the development of the live-scour hole, and the smaller the ultimate size of block formed. The harder the predominant rock type, the longer the last stages of scour development take to reach equilibrium. The greater the angle between the line of jet impact and orientation of the main joints and bedding planes ($>70^{o}$), the more likely it is that hydrofracture will occur and the resistance to erosion will be greater than in cases of smaller angles (70^{o}-35^{o} or $<35^{o}$), where exfoliation may occur.

To solve the problem of local scour in areas below spillways with flip buckets, it is necessary to know the characteristics of the flow for both the water jet and the spillway structure itself, in addition to the rock mass parameters.

None of the factors, either the geological, or the hydraulic, has an individual effect on rock mass behaviour. There is a strong correlation among them and this complicates the assessment of the contribution of individual factors.

5.2 Geological/Geotechnical Factors.

The previously mentioned geological properties of rock within and downstream of the spillways, concern the rock mass which includes the rock material properties and the characteristics of the discontinuities. In the following paragraphs, each one of the properties is analyzed and its correlation with other geological parameters is indicated, in particular the effect of discontinuities, using references and field observations.

5.2.1 Rock type.

It is important to determine, to which of the three main rock type groups (igneous, sedimentary or metamorphic), the rock within and downstream of the spillway belongs. The rock type plays a role, mainly in the initial stage of scour of the rock mass and influences its strength, durability, permeability.

Almost all igneous rocks, excluding pyroclastic rocks, offer high resistance to erosion. In particular, acidic igneous rocks such as syenite, granite, quartz porphyry and also diorite and gabbro are very competent rocks. Similarly resistant are the volcanic rocks such as andesite, basalt and trachyte.

Metamorphic rocks such as gneiss are very resistant. Schists with high proportions of mica, as well as brittle phyllite with clay must be avoided because they are easily eroded.

Crystalline sedimentary rocks such as non-porous crystalline limestone and dolomite are usually highly to marginally resistant. Sedimentary rocks cemented with quartz are moderately resistant, while the quartzites and the greywackes are more resistant. Mudrocks, depending upon the grain size and the percentage of clayey material, behave similar to either weakly cemented sandstones or to clay-schists. They are generally of low strength and may deteriorate with time due to wetting and drying cycles.

5.2.2 Fabric.

The grain size (very coarse, coarse, medium, fine or very fine grained rocks) and the degree of grain interlocking influence rock material properties e.g. strength, deformation, durability and permeability. Generally, coarse-grained rocks weather more rapidly than fine-grained types of similar mineralogy, and so they are more easily affected by the erosion process. The more strongly the minerals are bonded together, the greater they resist weathering and erosion.

5.2.3 Porosity.

The value of porosity governs other geotechnical properties of the rock. Porosity depends upon the degree of interlocking between minerals and when the grains are poorly interlocked, the rock may be weak and relatively non-elastic. If the grains are well interlocked, the rock will exhibit high strength and high elasticity values. Porosity is closely related to the degree of saturation of the rock. High porosity will lead to high moisture content which in turn affects other geotechnical properties of the rock.

5.2.4 Specific gravity/unit weight.

The weight of rock blocks is required for the calculation of the forces acting on them e.g. gravity, frictional resistance and kinetic energy. The ability of flowing water to transport rock fragments, depends on various factors, including the specific gravity of the rock. When rock blocks are lifted up and carried away by the water, they acquire kinetic energy and become power tools for breaking down other obstructions and for grinding other rocks.

5.2.5 Moisture content.

High moisture content results in an increased unit weight of the rocks and causes loss of strength. The unconfined compressive strength of quartzitic shale, quartzdiorite, gabbro and gneiss can be reduced by as much as 50 per cent by saturation in water as compared with oven dried specimens.

5.2.6 Permeability.

Passage or seepage of fluids through voids of a porous intact rock determines the permeability of the rock. The characteristics of permeability of the rock are important to be recorded, because movement of water through the rocks might reduce their resistance against erosion and may cause other geotechnical and environmental problems.

5.2.7 Strength.

Strength is an important parameter for estimating the rock response to water impact. Rock with low strength favours erosion and scour. Softer rocks will break up along microcraks, but stronger rocks will yield larger fragments which are effective tools for abrasion. In some cases, rock strength decreases with time.

Rock hardness is closely related to the compressive strength. Many authors have proposed various correlations. Miller (1965) showed the relation between the unconfined compressive strength, σ_c(MPa) and the Schmidt Hammer Rebound Index (R), taking into consideration the rock density (Fig.5.3).

5.2.8 Cohesion.

Cohesion is the no load shear strength of the rock and depends on rock composition, structure and strength. It is correlated with the erosion process in the same way as compressive strength.

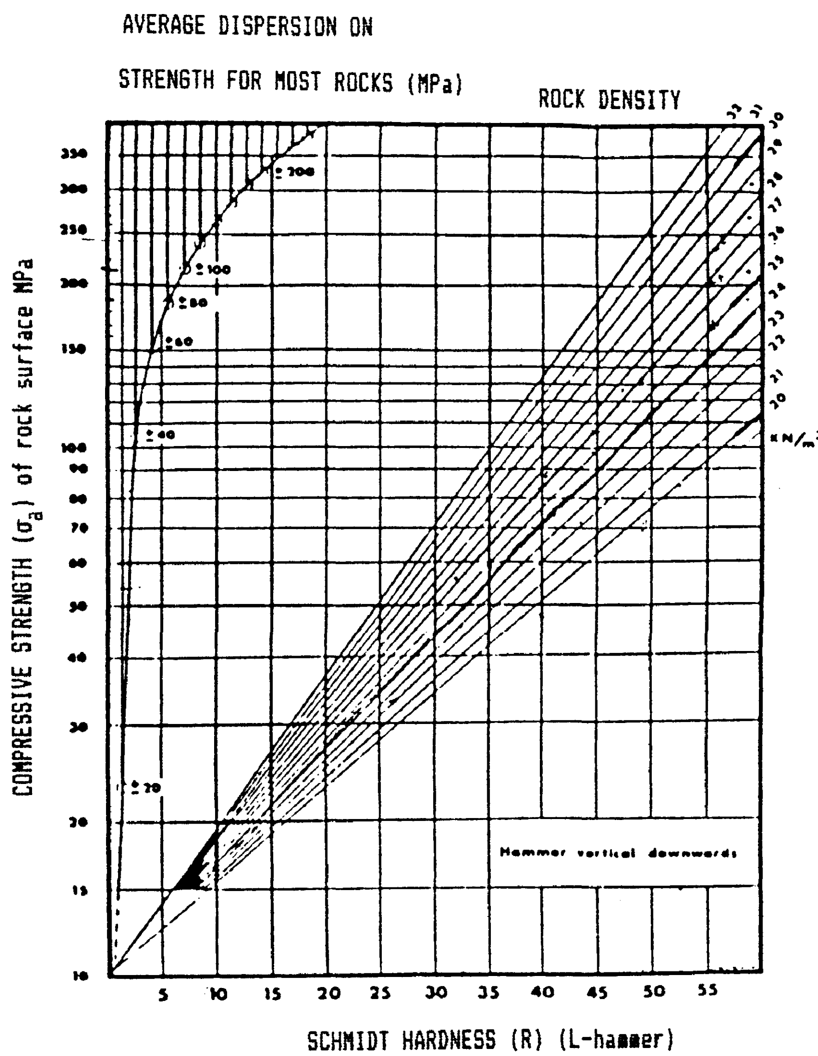


Fig.5.3 Correlation chart for Schmidt Hammer tests, relating compressive strength, rock density and rebound number (Miller, 1965).

5.2.9 Angle of internal friction.

Friction determines the internal shearing resistance to sliding along planes of weakness e.g. geological discontinuities, cracks, joints and faults. Maximum values for the angle of internal friction, ϕ(degrees), may be as high as 75 to 80 degrees. However, residual values rarely exceed the range of 25 to 30 degrees (Janardhanam et al., 1985).

5.2.10 Deformation.

Modulus of Elasticity, E_t(MPa), is the value used to determine the elastic behaviour of rock (deformation) under stress. The rock origin, mineralogical composition, fabric, joints and degree of alteration, moisture content and time control the rock deformability. Various authors have correlated the Modulus of Elasticity with the hardness and dry density of the rock.

Knowing the value of the Modulus of Elasticity of a certain rock and the forces to which it is subjected, the strain could be determined. In the case of high values of strain, considerable changes in the internal stresses of a confined rock mass take place. Relief of these stresses towards an open face of the rock mass is usually accompanied by fracturing of the rock along existing planes of weakness and may result in unfavourable conditions downstream of a spillway.

5.2.11 Discontinuities.

Introduction

A discontinuity is defined as any interruption in lithological and physical properties (Cameron et al., 1988). Only rarely will one encounter bedrock which is continuous within the area of study and which exhibits no discontinuities.

Discontinuities affect not only the application of rock mechanics theory but they will, in most cases, lead to problems with water. Problems associated with the presence of water and its movement through the rock masses relate to the weakening of the rock mass and alteration of the geotechnical properties (Legget et al., 1988). Obviously such loosened and weak bedrock is much less resistant to flowing or falling water, the result being in general, an extensive and quickly formed scour. Failure will generally take place along discontinuity surfaces or may select the combination of joints yielding the greatest continuity to form a failure surface.

Discontinuity surfaces within the rock mass require careful study. They come in a variety of types, depending on their origin and are characterized by their size, shape, frequency and orientation. They can be classified under two broad categories (Cameron et al., 1988):

- stratigraphic discontinuities which are limited to stratified rock sequences, such as the sedimentary rocks and their metamorphic equivalents, including those hosting or admixed with volcanic igneous rocks and
- structural discontinuities which occur in all rock associations.

Stratigraphic discontinuities.

Under this heading are included depositional features such as bedding planes, bed contacts, unconformities, sedimentary structures and textures, bed pinch-outs and facies changes in the same lithostratigraphic unit and non-uniform lithology. In general, uniform geological conditions are likely to provide a much more scour resistant rock mass than would a highly variable site (Woodward, 1984). A brief description for each of the stratigraphic types is following:

- Bedding planes.
  The method of formation of sedimentary rocks leads to the creation of a surface of deposition that visibly separates each successive layer of stratified rock from its preceding or following layer. These planes may separate beds less than 10mm in thickness called laminations, or beds thicker than 10mm called stratum. Bedding planes behave as planes of weakness, along which the rock will tend to break apart.

- Bed contacts.
  They are surfaces which separate beds of the rock composed of several laminations or strata. They behave similarly to the bedding planes, and both mark changes in the period and conditions of deposition.

- Sedimentary structures and textures.
  They interrupt the internal homogeneity of beds. Large scale cross stratification, bioturbation, root mottling together with gradded bedding, zones of fossil accumulations and of intraformational conglomerates may cause a lack of bed continuity.

- Pinch-outs and facies changes.
  Pinch-outs are changes in the thickness of a bed, while facies changes are lateral or vertical variations in the lithology or paleontology of a rock unit. Both can cause confusion in the interpretation of the geology of the study area, changes in the mechanical and engineering properties of the rock and consequently, in rock erosion resistivity.

- Unconformities.
  They are particular surfaces below which the strike and dip of bedding differs from the strike and dip of the bedding above. A surface of unconformity juxtaposes two distinct bedding attitudes and represents a time interval during which the upper surface of the older rocks was eroded and the landscape was tilted before the younger beds were deposited. A special case of unconformities is the disconformities, where the surface indicates only a period of erosion without tilting. They represent, in a vertical sense stratigraphically, rocks of different compositions and thus, different response to erosion. Their identification is therefore, a necessity.

Structural discontinuities.

Structural discontinuities are caused by movements resulting from natural compressive and tensional stress fields which affect rock masses in the upper crust of the earth. The resulting rock deformation produces folds, fractures, faults, joints and, in the case of some orogenic belts, regional metamorphism and forceful injection of molten rock and other fluids.

Such processes can change the orientation of the stuctural planes of the rock such as stratal dip, schistosity, foliation, zones of igneous contact and epigenetic veins, which are important structural discontinuities concerning the mechanical behaviour of the rock. Finally, a type of discontinuity often overlooked, involves dissolution cracks and cavities. The characteristics of the previously mentioned discontinuities are as follows:

- Igneous contacts.
  Igneous contacts form zones of various dimensions according to the compositions of the intrusive and the host rocks, the depth of the intrusion and the prevailing temperature gradients, water content and the orientation of the intrusive body in

relation to the host terrane. These zones are of different mechanical properties than the surrounding rocks and contribute to non-uniform conditions of the area, and furthermore to differing rates of erosion on either side of the zone contact. Such zones are often characterized by closely spaced joints.

- Veins.
  Veins are epigenetic fillings of fractures, composed of minerals which differ in their resistance to erosion from the surrounding rocks.

- Foliation, schistosity.
  Foliation or schistosity occurs in all metamorphic rocks in which there is a more or less parallel disposition of the minerals during metamorphism. Foliated or schistose rocks tend to split or cleave readily into slabs parallel to the schistosity, forming planes of weakness and leading to failure.

The main structural discontinuities which cause great problems downstream of spillways are fractures in the rock mass. Any break of the rock can be defined as a fracture. Since fractures are caused by stresses, it is obvious that fracturing is caused by stresses which are greater than the strength of the rock (Krynine et al., 1957). The term "fractures" encompasses all joints, shear zones, faults, and also the cracks and fissures which are designated minor fractures. These fractures are described as follows:

- Faults.
  When subjected to great pressure, the earth´s crust is subjected to shear forces. When such forces become excessive, movement along a plane of failure (fault) will proceed until the unbalanced forces are equalized (Legget et al., 1988).

  Normal faults are closely related to the dominant rock folding of the district in which they occur. Frequently, they are oblique to the fold axes, and it is difficult to detect any system of arrangement among them. In case of normal faults occuring in horizontal or gently inclined strata, it can be said that they are the result of tension. Many normal faults have been formed due to subsidence or collapse of the crust. Reverse faults are usually encountered in regions of highly folded and compressed rocks. The overthrusts of a highly disturbed region are often cut across by a series of normal faults. Such phenomena seem to suggest that, while overthrust and traverse faults are the result of horizontal movements, the normal faults in question may have come into existence when the tangential pressure was relieved (Campbell et al., 1953).

Displacement of rock masses due to faulting indicates that rocks of differing erodibility may be juxtaposed, resulting in the formation of a knickpoint. Because faults are associated with breaking and mechanical disintegration of the rock on or around the fault plane, these discontinuity surfaces at the earth´s surface are generally subject to more rapid erosion than the surrounding rock and fault traces are therefore often seen as local topographic lows, gullies or river valleys (Ramsey et al., 1987). In addition, common features observed in faulted areas are brecciated and shear zones.

- Shear fracture planes.
  Such planes may be formed during folding, parallel to the bedding planes which are opened under exceptional compression forces or in general, when the crust of the earth is being subjected to intense strains and stresses.

- Bands of cataclastic rocks.
  Under enormous pressure the constituent minerals of certain rocks may have been rendered in a sense plastic and compelled to flow. Rock bands of this type are often well developed along one or both sides of faults or dislocations of the crust. Such bands of cataclastic rocks may be mylonites, which are typically developed along lines of overthrust or reverse faults and are usually closely associated with crystalline schistose rocks into which indeed they often pass, or friction (or crushed) breccias, which are best developed in regions where the rocks have been subjected to much compression.

- Folds.
  Folds are also deformation features which contribute to non-uniform conditions of the rock mass and are of large scale or sometimes, of small local occurrence (microfolds). They occur in rocks containing planar features, such as sedimentary bedding, lithological layering in metamorphic schists and gneisses, or planar anisotropic features, such as cleavage or schistosity, produced during an early deformational event. Folding is due to the horizontal compression and lateral deflection (Mathewson, 1981). The action of the horizontal

forces should have been balanced by a vertical couple of forces. The upward acting force of the vertical couple causes a system of vertical stresses and an upward warping (anticline), whereas the downward-acting force of the couple is responsible for the creation of the syncline. There is tension at the top of an anticline and at the bottom of a syncline. As the resistance to the upward movement is less than to the downward warping of a syncline, there is greater possibility for cracking at the top of an anticline (Krynine et al., 1957). Not all folds, however, are formed by buckling processes. Folds can also be formed by non-uniform forces. Variable vertical subsidence of crustal layers, differential shearing in shear zones or by the sliding of rock masses over underlying floor irregularities can also give rise to folds (Ramsey et al., 1987).

- Joints.

Joints are by far the most common type of geological structure. They are defined as fractures or cracks in rock, along which little or no displacement has occurred. Joints may be classified as either planar or curved surfaces. In particular, joints are planar surfaces of actual or potential failure or parting in a rock mass. When joints are formed in competent rocks, they represent a complete fracture in the rock as distinct from cleavage, which may appear as an internal structural weakness in the rock. These planar surfaces never occur alone, but usually in parallel sets and they result from internal stresses, either during the cooling of the rock or during tectonic displacement (Cameron et al., 1986). Curved surfaces may develop because of shock waves passing through the rock mass, due to volcanic or crypto-explosions, meteorite impact or brittle tensile fracture (Brink et al., 1973).

Joint sets of different orientations often combine to form joint systems of variable dimensions depending on the nature of the rock and the forces which produce the fractures. For example, sedimentary rock strata often exhibit tension joint sets which are at right angles or oblique to the bedding direction and most often occur in high angle conjugate pairs. Cooling of igneous rock masses generally involves the creation of tensional joints. During the intrusive stage of some igneous rock masses, compressive and shear forces may dominate and produce additional joint sets. In granites, besides the presence of the main vertical joints, a set of cross-joints, horizontal or inclined, usually undulating, may be exhibited giving to the rock a kind of bedded appearance. As schistose or foliated rocks differ much in composition and structure, they might have been expected to show considerable variety in the character of their jointing. As a rule, the jointing of schistose rocks is irregular.

The main obvious causes leading to jointing according to Campbell et al. (1953) are as follows:

- contraction, where any moist and plastic rock, such as claysone shrinks upon drying and becomes cracked or fissured. The passage from the non-crystalline to the crystalline condition also involves contraction and this may be the reason for jointed structure of certain chemically formed deposits. The jointing structure of crystalline igneous rocks, vertical joints in granite, prismatic jointing of basalt and other eruptive rocks, are also due to contraction;
- expansion, where rocks of all kinds when subjected to heat will expand and when cooled will contract. As a result, they will become fissured. Rocks are subjected to sudden heat in cases of molten rock erupted in their immediate neighbourhood and of sun heat, especially in dry tropical and subtropical regions. The cross joints in granites may owe their origin to after cooling action, while many lavas have a tendency to split most readily in the direction of flow;
- crustal movements develop powerful mechanical stress or strain which result in the regular intersecting systems of main joints in sedimentary strata. During such movements, strain of torsion, compression and tension are caused. Vibratory movements of the crust, resulting in the buckling up and folding of strata in gigantic mountains, must often have induced earthquakes, but the fissuring and shattering due to passage of such vibrations or waves of elastic compression could not be distinquished from the ordinary effects of folding and torsion.

Forces directed towards the surface may break open the rock or loosen its structural composition. Fractures give rise to a blocklike structure, however, the blocks may not be separated from each other. Joint size, shape, frequency and orientation, together with the rock mass strength parameters control the size and shape of partially unstable blocks. If the blocks are relatively small and if the fissures separating them are open then, the blocks are susceptible to movement or even complete removal (by uplift and plucking), the result being in general an extensive and quickly formed scour.

- Stress relief joints.
  Rocks formed under high confining stresses, such as granites, will have some remnant of these stresses, also called in situ stresses, preserved. In addition, when the rocks are subjected to load or stress, changes in the internal stresses are developed. If the rock masses are confined, these stresses, named residual stresses, remain for a rather long time after the loading is removed. As soon as some rock material is removed or a free surface is found towards which relief can be succeeded, stress relief joints, occur.

- Blasting fractures.
  Some of the fractures occurring on a rock mass, particularly when the latter is hard, are not naturally developed. Such fractures result because of excavation activities, e.g. blasting, and they have a distinctive pattern. The surfaces of the discontinuities are neither planar nor continuous and are radially directed around a point.

Survey of the discontinuity planes.

Common rock displays so many planes of weaknesses as to be essentially a collection of separate blocks fitted in the three dimensional mosaic (Goodman et al., 1985). We can call this material a discontinuous rock mass. One form of failure of both surface and underground excavations is destruction of the surface by erosion due to water.

Joint surveys are conducted on rock faces associated with outcrops, exposed slope faces, channels, trenches, tunnels, shafts, borehole sides and borehole cores. The joints measured (the sample) which may only be a portion of the joints exposed (the sample population) are considered to be representative of the joints within the entire rock mass (the target population) (Jennings et al., 1973).

The parameters (Table 5.3), describing the structural discontinuities which are required for the analysis of a rock mass fall, into the following groups:
- factors describing the spatial position and orientation of the

joints such as the co-ordinates and the dip and strike of each joint,
- factors describing the nature of the joints i.e., their length, continuity, spacing, apertures, gouge filling and surface properties.

JOINT SURVEY DATA
R.Q.D
Number of joint sets
Joint spacing (mm)
Joint separation (mm)
Joint orientation - dip angle and dip direction (degrees)
Joint wall roughness
Gouge and properties
Water content

Table 5.3 Main properties for joint survey data.

A joint survey data sheet has been introduced by Jennings et.al. (1973). It includes locality and general rock properties in addition to joint properties.

Rock material properties, recorded during the survey, are the rock type and the hardness according to Table 5.4. Joint continuity, joint spacing, joint separation, joint surface orientation and every other joint property are all key parameters with respect to evaluation of the rate and extent of rock erosion within and downstream of spillways.

ROCK HARDNESS DESCRIPTION	IDENTIFICAT
Very soft soil	easily moulded with fingers; shows distinct heelmarks.
Soft soil	moulds under fingers with strong pressure; faint heelmarks.
Firm soil	very difficult to mould with fingers; difficult to cut with a hand spade.
Stiff soil	cannot be moulded with fingers; cannont be cut with hand spade and requires handpicking for excavation.
Very stiff soil	very tough and difficult to move with handpick; requires pneumatic spade for excavation.
Very soft rock	material crumbles under firm blows with a sharp end of a geological pick and can be peeled off with a knife. It is too hard to cut a triaxial sample by hand.
Soft rock	can just be scraped and peeled with a knife; indentations 3mm show in the specimen with firm blows of the pick point.
Hard rock	cannot be scraped or peeled with a knife; hand-held specimen can be broken with the hammer end of a geological pick with a single firm blow.
Very hard rock	hand-held specimen breaks with hammer end of pick under more than one blow; can be scratched with a knife; slight ring under hammer blow.
Extremely hard rock	specimen requires many blows with geological pick to break break through intact material; cannot be scratched with a knife; rings under hammer blow.

Table 5.4 Categories of rock hardness (Jennings et al.,1973).

Discontinuity type.

The type of discontinuity, e.g. bedding joint, tension joint or contact should be determined, to indicate the cause of the joint development and to identify its origin. Fractures are due to local overstress of the rock material which produces rupture surfaces. If the stress source is tectonic, shear or tension joints, normal, thrust or reverse faults, or shear zones would be produced. When these tectonic joints occur in sedimentary rocks, they are often referred to collectively as bedding joints or cross joints, depending on whether they are parallel to or cut across bedding planes. Where stress occurs as a result of overbuden removal or cooling, exfoliation and cooling joints result.

Rock Quality Designation.

Measurement of the RQD provides a method of assessing the quality of the rock mass. This method yields a numerical figure between

0 and 100 per cent. It is measured per drill run and is defined as the total length of the individual core sticks greater than 100mm in length, divided by the length of the drill run and expressed as a percentage. In case there is no borehole core, determination of the RQD may be done by estimating the number of joints per cubic metre of the in situ rock mass (Table 5.5). Spurr (1985) referring to plunge pool development, stated that a rock mass with 80 per cent RQD would be more resistant to erosion than a rock mass with RQD less than 50 per cent.

NUMBER OF JOINTS PER CUBIC METRE	RQD	NUMBER OF JOINTS PER CUBIC METRE	RQD
33	5	18	55
32	10	17	60
30	15	15	65
29	20	14	70
27	25	12	75
26	30	11	80
24	35	9	85
23	40	8	90
21	45	6	95
20	50	5	100

Table 5.5 Correlation chart between RQD and number of joints per cubic metre (Kirsten, 1982).

Fracture frequency.

Fracture frequency is also a measure of the quality of the rock mass but, unlike RQD, it is not based on a specific size of core. It is a number which is obtained by counting the number of natural fractures that occur per meter length of the core. For over 20 fractures per meter, the specific number is not important and the value may be recorded as >20; the poor quality of the rock is certain. The frequency of the joints influences the strength of the rock mass. The greater the fracture frequency is, the smaller the strength of the rock mass will be. Rock Quality Designation and Fracture Frequency are parameters that reflect joint block size, one of the most important factors for the erodibility of rock masses.

Number of joint sets.

The effect of joints is that the rock mass is divided into a number of blocks by joint surfaces, along which movement may take place. Joints exist in one or more sets of various attitudes, and if there is more than one set, they intersect at various angles. The joints of the most important set, referred to as major joints, can usually be traced for tens or hundred of metres, whereas the secondary joint sets are usually of less importance and more likely to be curved or irregularly spaced.

Continuity

Continuity, otherwise called persistence, of the joint set can only be described in well exposed rock masses. It is often merely stated, if a joint set is continuous or not. The most meaningful value is the average trace length of fractures belonging to a specific set (Brink et al., 1982). Numerous persistent joints make a rock mass more liable to scour.

Joint wall separation.

Some discontinuities show tight contacts between their surfaces, but most are open to some degree. Separation of the joint walls, is described as closed, very narrow, narrow, wide, very wide, major fracture has a marked effect on the strength (Brink et al., 1982). A rock mass with numerous widely open joints is prone to be eroded more easily than a rock mass with tight joint walls. Joints thus permit entry of the surface water into the rock mass. The size of a fissure opening (aperture size) or joint block separation is often a function of the proximity to the surface, the slope of the ground and the stress regime.

Joint spacing.

Spacing and frequency of the joints influence the strength of the rock mass. According to the joint spacing and the uniaxial compressive strength of intact rock, the rock mass could be classified into different groups (Fig.5.4). The spacing between joint walls determines the quality of the rock mass. Small values of joint spacing indicate small rock blocks, which may be easily transported by water flow of any magnitude.

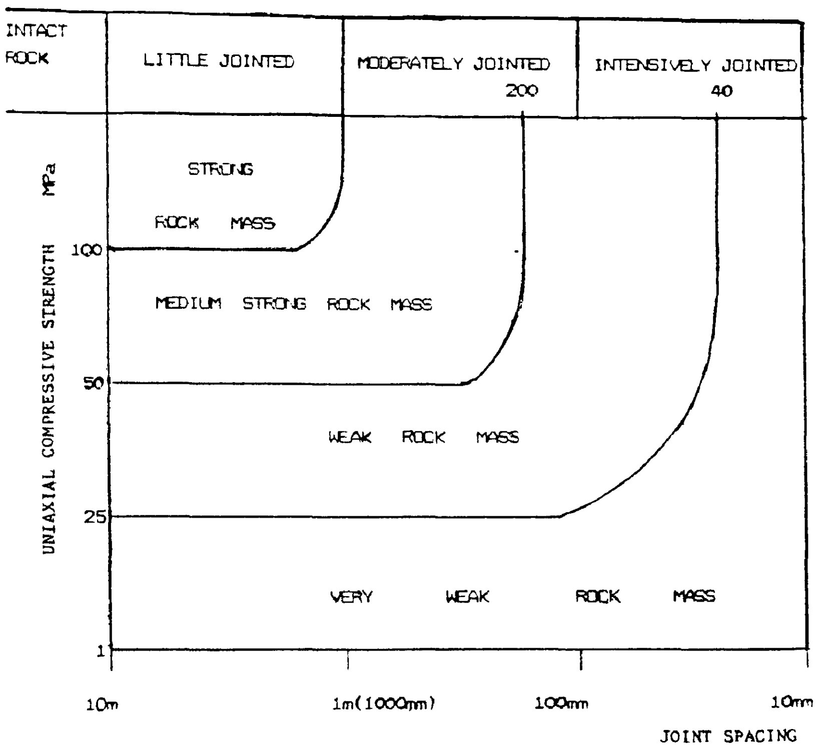


Fig.5.4 Rock mass quality, according to compressive strength and fracture spacing (Jennings et al., 1973).

Waviness of joints.

The waviness is a surface feature of the joint planes. The length or base and amplitude or offset of the wave on the joint plane is measured by using a standard straight edge placed on the exposed joint surface in a direction normal to the strike. The amplitude of the wave is recorded in centimetres and the wave length usually in metres (Jennings et al.,1973).

Roughness of the joints.

Roughness, as indicated by means of the terms slickensided surface, smooth, rough, very rough, steps and ridges is an important surface feature of joint planes. Bieniawski (1973) classified joint surfaces according to their roughness into similar categories but correlated with the friction angle, ϕ(degrees), of the joint plane (Table 5.6).

DESCRIPTION OF JOINT WALL ROUGHNESS	JOINT FRICTION ANGLE ϕ_j (degrees)
Slickensided surfaces	25°
Smooth surfaces	30°
Defined ridges	35°
Small steps	40°
Very rough surfaces	45°

Table 5.6 Correlation between roughness and friction angle (Bieniawski, 1973).

Roughness has an unquestionable effect on the development of strength of the joint surface against the shear stress created by water forces. Rough surfaces exhibit relatively high shear strength and consequently greater resistance to movement and the erosion process than smooth surfaces do. Any roughness on the parallel joint surfaces creates rock bridges that must be broken before the block can be moved along them (Goodman et al., 1985).

Shape of rock blocks.

Rock mass properties are affected by the shape of the rock blocks. The form of intervening blocks can be described as blocky, equidimensional, tabular, or columnar (Goodman et al., 1982) and is determined by the orientation of the different joint sets, while the size of the blocks depends on the joint spacing.

Alteration - gouge.

The degree of alteration along a joint can be described as clean, stained, slightly altered, or filled. Joints may be filled with various minerals, e.g. calcite, dolomite, quartz, clay minerals, or they may be open if the infilling has been washed out.

In case of infilling between joint walls, a description similar to that for a soil profile must be made. Identification of the type of the filling (composition) and its origin is important (Brink et al.,1982). Its thickness and hardness often influences the strength of the rock mass.

Under the term gouge are icluded materials derived from breakage of the country rock due to movements, in situ weathered materials, foreign infilling materials deposited between the structural planes and also intruded igneous materials which are different from the host-rock.

Water content.

The presence of water in the joints determines the pressures acting on the rock mass and especially on the rock blocks formed by the joint planes. For classification purposes, estimation of water presence may be recorded as none - where no water exists on the joint face, damp - when the joint surface is moist but there is no free water present, seepage - when water coming from the joint is sufficient to saturate the rock face but there is no observable flow, and flow - when water is flowing from the joint and forming pools of water.

Water content in joints generally promotes unstable behaviour of jointed rocks especially when surfaces are smooth. Water in the joints increases the sliding forces and assists the sliding movement (Reinius, 1986). The effects of flowing water on jointed rock mass will be discussed in detail in Chapter 5.3.4.

Joint orientation.

The orientation of the joint, by which is meant the dip angle and the dip direction is very important especially with respect to the direction of water impact.

Except under very severe conditions, closed joints will not be a critical factor in determining rock scour resistance while the orientation of open joints or joints with erodible filling may be of great significance, particularly in rock masses with a small number of joint sets (≤ 3). In more highly jointed rock with 4 or more sets, joint orientation is likely to be less important (Woodward, 1984).

Joints can be characterised as favourable or not, for a certain excavation or construction, in this case for resistance against water impact within and downstream of spillways of dams. Favourable joints are these which do not assist the erosion process and unfavourably oriented are those which facilitate the removal of rock blocks.

In general, unfavourably oriented joints will be those that strike about normal to the water flow and dip steeply downstream, thereby favouring the development of a near vertical face. Downstream dipping planes or joints are more susceptible to developing an internal water pressure than upstream dipping planes and selective covering or anchoring of exposed downstream dipping layers might be necessary. The most unfavourable combination of joint sets and topography is where water flow can remove joint blocks simply by sliding on one set of joints or along the line of intersection of two joint sets without having to perform the additional work of lifting blocks (Woodward, 1984).

In spillway channels, experience with supercritical flow has shown that erosion will occur to only a small extent or not at all in rock of good quality, i.e. where the rock blocks between joints or cracks are large and interlocked. Erosion has been observed in rock with parallel joints, as for example in sedimentary or gneissic rock, where the joints or layers were directed downstream (Reinius, 1986), but also in rock of poor quality with closely spaced, open joints.

In case of scour hole development, the more closely aligned the major discontinuity planes to the incidence of the plunging jet, the easier the penetration and the more rapid the hydrofracture will be. In addition, the smaller the angle between the water jet direction and joint plane orientation, the more likely exfoliation to follow.

Key block theory.

This theory has been developed by Goodman et al. (1985) to be used for the instability of the rock mass during tunnelling. The method is based on a theorem which states that in order for any block to move, it must be finite, removable and potentially unstable in the excavated space (Table 5.7). Blocks of concern may be single, convex polyedra or united, multiple blocks formed by the union of individuals. A key block is a rock block bordered by joints, and which can be removed under the influence of gravity from the roof or the walls of a tunnel. The movement of the first block from the surfaces of excavation creates a space into which previously restrained blocks may then advance.

The principles of the key block method could be applied to the study of scouring in unlined spillways.

TYPES OF BLOCKS		DESCRIPTION
INFINITE		Provides no hazard.
FINITE	NON REMOVABLE	Because of its tapered shape
	REMOVABLE	Stable even without friction. It has a favourable orientation according to the resultant forces. Its virtual movement is away from the excavated space.
		Stable with sufficient friction (potential key block). Its tendency for movement is towards the free space, but unlikely to become unstable unless the frictional resistance on the potentially sliding surface is extremely small or there are loads in addition to the block's self weight driving the displacement.
		Unstable without support (key block).

Table 5.7 Types of blocks determined by key block theory (Goodman et al., 1985).

Stereographic projection.

The stereographic projection as a graphical device for solving geological, crystallographic and other spatial problems is well accepted. Besides the use of stereonets for the projection procedure, plots can be drawn with computer graphics on a video screen or a plotter. Knowing the orientations of the joint sets and the water flow direction, the projection and the correlation among them will be easy. In addition, the developed forces acting on the rock blocks might also be plotted, and the direction of the movement could be predicted.

5.2.12 Effect of discontinuities on strength.

The strength of the rock mass is much lower than the strength of rock material, because rock is weakened by the presence of various types of discontinuities. Besides the ability of the rock mass to resist the initial penetration of water, strength along joint surfaces is also of great importance to resist movement of blocks. This depends on the strength of the intact pieces and on their freedom to move, which in turn depends on the number, orientation, spacing and shear strength of the discontinuities. Tests (Hoek,1983) have shown that the intact material strength is not achieved in a rock mass even at the most favourable joint orientations.

Geological discontinuities and their characteristics control the size and the shape of unstable blocks (Turk et.al., 1985). The Mohr-Coulomb criterion

$$\tau = c + \sigma_n \tan\phi \qquad (13)$$

appears suitable to determine the shear strength, τ(MPa), along a joint surface, where σ_n(MPa) is the stress normal to the joint surface, c(MPa) is the cohesion, and ϕ(degrees) is the friction angle of the joint plane.

The parameters which affect the shear strength along joints, are the composite friction angle of the minerals forming the walls of the joints, the irregularities of the joint surface, the hardness of the rock forming the joint walls, the effect of previous shear movement, the presence and the thickness of the joint filling materials and the water pressure. With an increase of the hardness of joint wall material, there is a corresponding increase in friction angle where little or no joint filling is present. The shear movement required to reduce strengths to their residual values, varies with the hardness of the joint wall and is approximately 30mm to 60mm for weaker rocks and 100mm to 150mm for harder rocks (Bieniawski et al., 1973). If the joint is filled, then the filling material controls the shear strength and each type of material has different shear strength parameters. Water pressure along the joints reduces the normal stress and consequently the shear strength. Joint shear strength may be estimated by direct shear field tests, laboratory testing using either the triaxial or direct shear apparatus or by joint surveys and empirically derived shear stength formulas.

5.2.13 Effect of discontinuity surface properties on friction angle.

Friction mostly depends on the quality of the contact surfaces such as old or new, rough or smooth. For smooth surfaces, the coefficient of friction, $\tan\phi$, will range generally between 0,4 and 0,8, but mostly, it will be between 0,5 and 0,6.

5.2.14 Secondary permeability.

The ability to permit passage or seepage of fluids through cracks or discontinuities in rock masses is termed secondary permeability. Secondary permeability is several orders greater than permeability through the voids of porous rock material.

The secondary permeability of a rock mass is proportional to joint parameters and, in particular, is a function of the third power of the fissure or joint wall separation. The equivalent permeability, k(m/s), of a parallel array of cracks is given by the following equation, by Hoek and Bray (1977):

$$k = (g\ e^3\ ) \ /\ (12\ v\ b) \tag{14}$$

where $g(m/s^2)$ is the gravitational acceleration, e(m) is the wall separation of the cracks, b(m) is the spacing between the cracks, $v(m^2/s)$ is the coefficient of kinematic viscosity ($0{,}0101 * 10^{-4}$ m^2/s for pure water at 20 ^oC).

Although the secondary permeability is affected by the wall separation of discontinuities and the amount of infilling, a rough estimation of permeability can be obtained from the spacing of discontinuities (Tables 5.8 and 5.9).

The presence of moisture or water can have several effects on rock mass. Water pressures induce instability, by reducing the shear strength of potential failure surfaces and by increasing the forces tending to induce sliding. In addition, freezing of groundwater during winter can cause wedging in water-filled joints. Erosion of surface soils and fracture infillings, caused by water flow, can give rise to a reduction in stability. Soft materials, e.g. clay or other decomposition products, are washed from joints or faults. Chemical action may dissolve some of the more soluble mineral components, resulting in a change in the mechanical properties of the rock.

ROCK MASS DESCRIPTION	TERM	k (m/s)
Very closely to extremely closely spaced discontinuities.	Highly permeable	10^{-2}–1
Closely to moderate widely spaced discontinuities.	Moderate permeable	10^{-5}–10^{-2}
Widely to very widely spaced discontinuities.	Slightly permeable	10^{-9}–10^{-5}
No discontinuities.	Effectively impermeable	10^{-9}>

Table 5.8 Permeability of a rock mass according to spacing of discontinuities (Anon, 1977).

	k(m/s)	Intact rock	Frctured rock	Soil
Practically impermeable	10^{-10}	Slate		Homogeneous clay below zone of weathering
	10^{-9}	Dolomite		
	10^{-8}	Granite		
	10^{-7}			
Low discharge poor drainage	10^{-6}			Very fine sands, organic and inorganic silts, mixtures of sand and clay glacial till, stratified deposits
	10^{-5}	Limestone, Sandstone	Clay-filled joints	
	10^{-4}			
	10^{-3}			
High discharge free drainage	10^{-2}		Jointed rock	
	10^{-1}			Clean sand, clean sand and gravel mixtures
	1		Open-jointed rock	
	10^{1}			
	10^{2}		Heavily fractured rock	Clean gravel

Table 5.9 Coefficients of permeability of typical rocks and soils (Hoek et al., 1977).

5.2.15 Weathering.

Weathering is the starting point in the train of events leading to erosion. The degree and rate of weathering, depends on the temperature, the moisture content and the available organic material. The rate of weathering depends not only upon the vigour of the weathering agent, but also on the durability of the rock concerned, which is governed by the mineralogical composition, texture and porosity, as well as by discontinuities. Discontinuities represent planes of weakness along which weathering is concentrated. Familiar features of weathering action along discontinuities are the joints and grikes in limestone and dolomite, tors in granites and spheroidal weathering in basalts and dolerites. Cleavage, schistosity and foliation in metamorphic rocks influence the rate of weathering.

According to the climatic N-value (Weinert, 1969), either disintegration or decomposition will prevail. Nearly all igneous and some metamorphic rocks can decompose, while nearly all sedimentary rocks will only disintegrate during weathering, no matter what the climatic envir nment is (Weinert, 1969). Other processes of alteration closely related to weathering, e.g. argillization, chloritization, sericitization, or feldspathization, silicification, dolomitization, calcification, either reduce or increase the competency of rocks.

5.2.16 In situ stress.

Rock in its virgin condition is always in a state of stress, as a result of the weight of the overlying material and tectonic forces. This stress cannot easily be predicted because the tectonic component is generally unknown. Knowledge of the stress regime will assist in establishing the most stable shape and orientation of excavations, in calculating the stress concentrations around the excavation area and in evaluating the need for support or reinforcement.

When the stress in a rock mass downstream of a spillway, exceeds the strength of the material involved, failure occurs. Slabbing or buckling failure occurs when the stress in the rock approaches a third to a half of the uniaxial compressive strength (Cook, 1976). Rupture or ultimate strength is defined as the maximum stress difference a body is able to withstand, prior to loss of cohesion by fracturing. Disturbance of the stess field due to excavations leads to the development of higher stress and possibly to unstable conditions. Excavation in rock induces a readjustment of the virgin state of stress and results in the creation of stress concentrations, the magnitudes and locations of which will affect the stability of the works.

Tensile stresses result in the opening of discontinuity planes, and the resulting loss of shear strength may give rise to removal of rock blocks by flowing water. Moderate compressive stresses will increase interblock frictional strength and will reduce block movement.

5.2.17 Uniformity of the lithology of the downstream area.

Stratigraphic discontinuities play a great role in the erosion. They control the rate and the intensity of erosion. They often produce gradient changes (knickpoints), in spillway channels, in the form of abrupt waterfalls, series of closely spaced "stair-steps", or gentle, subtle changes. The position or location of the knickpoint, which is a non-permanent feature, depends upon the relative erosion resistance of channel materials and their response to discharge conditions. In the case of the less resistant rock downstream of the contact of the discontinuity, erosion results in the formation of a fall or rapid. On the other hand, differential erosion immediately upstream of the contact may result in the formation of a hydraulic jump if the downstream reach is underlain by the harder, more resistant lithology.

In a plunge pool, the non-homogeneity of the rock may significantly affect its shape (Spurr,1985). When a rock at one side of the live-scour hole is more scour resistant than elsewhere, the development of the plunge pool may be partially confined so that it becomes asymmetrical. The more confined the pool, the more intense the hydraulic action on the pool boundaries will be, regardless of whether it is caused by the secondary flows recirculating around the jet, by the quasi-steady roller or by the free jet itself.

In general, uniform geological conditions are likely to provide a much more scour resistant rock mass than would a highly variable site (Woodward,1984).

5.3 Hydraulic Parameters.

The design of the spillway, the geometry of the impact area and the characteristics of the water jet, act in concert with the geological factors to control the response of the bedrock downstream of the spillway to water outflow.

5.3.1 Engineering design of the spillway.

The difference between the level of the spillway crest and the initial river bed level (before erosion of any significance takes place) or the surface of the tailwater (in case of a water pool at the toe of the spillway), H(m), as well as the length of the spillway crest, Z(m), are used in various calculations. These are necessary for the estimation of the dynamic water forces and provide a measure of the scour potential at the critical point of impact or downstream of the concrete lined structure.

The slope of the downstream surface of the wall, in case of an overflow spillway, or the radius of the arch wall over which a free jet falls, or the characteristics of the dissipation structure, such as the angle that the exit of the bucket makes with the horizontal, its width and the level at which it is situated on the spillway wall, are important parameters.

5.3.2 Geometry of the downstream area.

The roughness of the surface of the downstream area is a hydraulic parameter important for the estimation of the forces that can develop on the area downstream of the spillway. The roughness of an unlined surface is influenced not only by the rock structure but also by the advancing direction during the excavation and by the blasting method. Roughness of the channel has an effect on the head loss and the velocity of the water flowing downstream of the spillway structures. The turbulent flow around the roughness elements of an excavated rock surface gives rise not only to zones of low and high pressures but also to eddies and force fluctuations. The considerable forces which may occur, may not be able to lift a rock block, but vibration of the block may affect the joints and the stability of the block (Reinius, 1986).

The general friction formula in a conduit shows the correlation between the friction factor, f, and the head loss, h_f(m), as follows:

$$h_f = f * (L / 4 R) * (V^2 / 2 g) \tag{15}$$

where L(m) is the length of the conduit, R(m) is the hydraulic radius, V(m/s) is the mean velocity of the flow, g(m/s^2) is the acceleration of gravity (Reinius, 1970). Manning´s equation for head loss, h_f, is as follows:

$$h_f = (V^2 * L * n^2) / R^{4/3} \tag{16}$$

where n is the Manning coefficient or roughness coefficient (Table 5.10) and is correlated with the friction factor, f, according to the following formula:

$$n = (8 g / f R^{1/3})^{1/2} \tag{17}$$

MANNING'S NUMBER	CHANNEL CHARACTERISTICS
0,010	very smooth-glass, wood
0,011	very smooth concrete or wood
0,012	smooth concrete
0,013	normal concrete
0,014	wood construction
0,015	vitrified clay
0,017-0,020	shot concrete, excellent earth
0,020-0,025	smooth bare earth in good condition
0,025-0,035	earth channels with some vegetation
0,035-0,040	natural streams
0,040-0,050	rough mountain channels or rivers
0,050-0,250	stream with rough beds
0,250-0,700	channel with heavy vegetation

Table 5.10 Manning ´s number according to channel characteristics (Mathewson, 1981).

Fig.5.5 shows the correlation between R, n and K_s(roughness parameter).

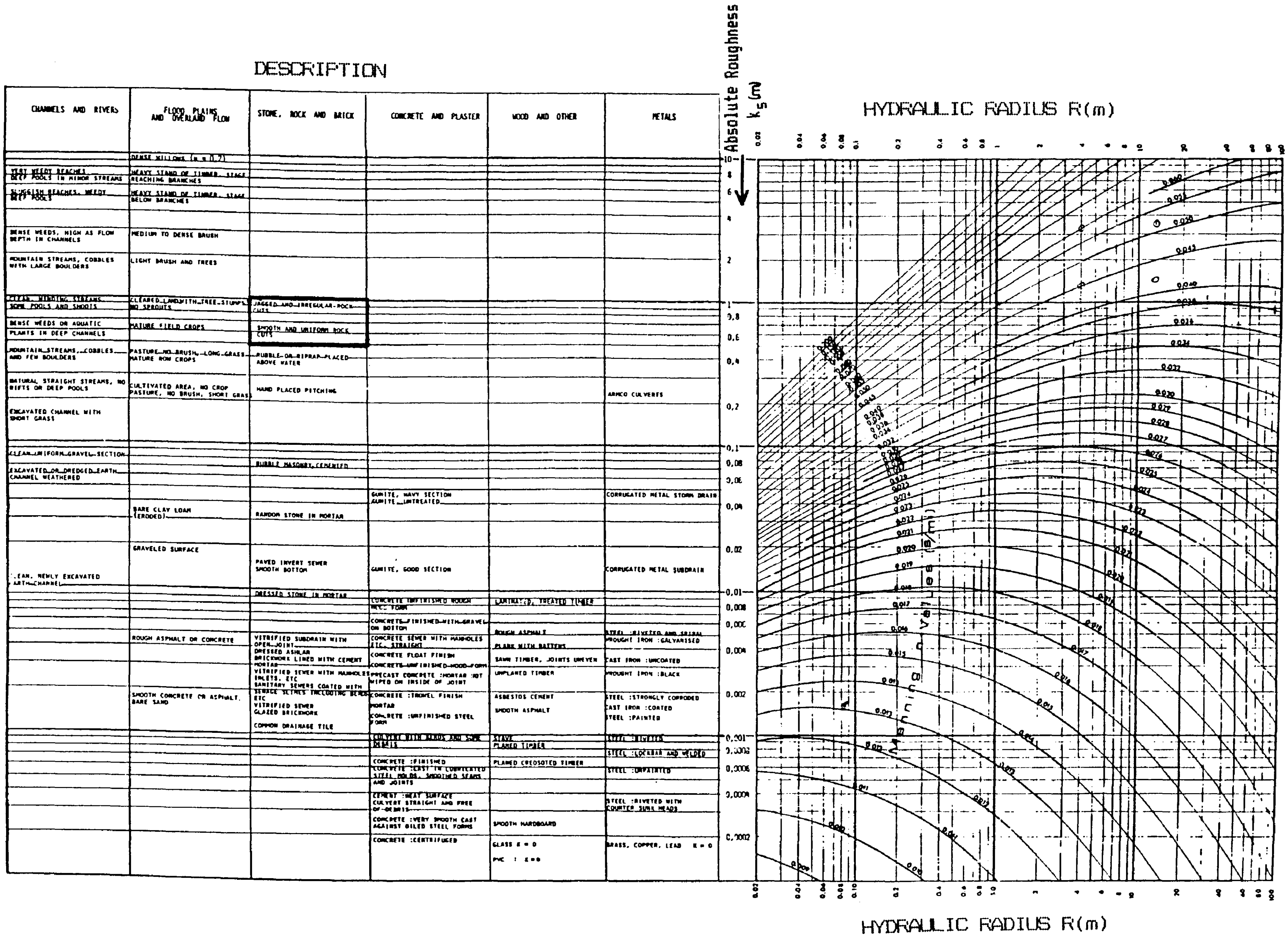


Fig.5.5 Correlation chart between roughness coefficients (k_s and n) and hydraulic radius. Compiled byA.Rooseboom and E.Van Zyl,U.P.,1978.

The Reynold's number, R_e, for flow in pipes is

$$R_e = (V * D) / v \tag{18}$$

where V(m/sec) is the velocity of the flow, D(m) is the diameter of the pipe and v is the kinematic viscosity of the water and is equal to $1,14 \times 10^{-6} m^2/s$ at 15^oC. The following relation between the friction factor, f, the Reynold's number, R_e, and the roughness parameter, k_s(m), has been given (Bar-Shlomo, 1978) for flow in smooth pipes:

$$1 / f^{1/2} = 2 \log[(R_e * f^{1/2}) / 2,51] \tag{19}$$

and for flow in rough pipes:

$$1 / f^{1/2} = 2 \log[(3,71 D) / k_s] \tag{20}$$

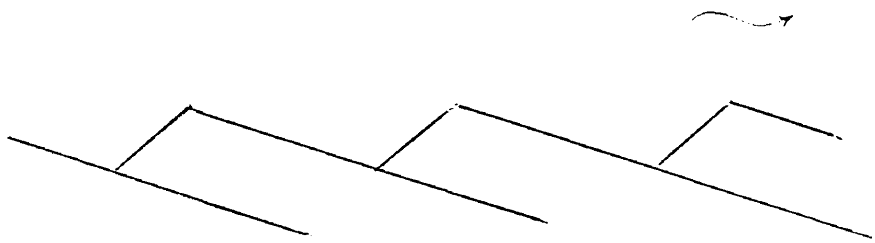
and for the transition region between the two former types, a combination of the equations has been suggested by Colebrook-White (1937) (Bar-Shlomo, 1978), as follows:

$$1 / f^{1/2} = -2 \log[(k_s / 3,71D) + (2,51 / R_e f^{1/2}) \tag{21}$$

Reinius found that the friction factor in open channels does vary approximately as shown in the above equations, except for small differences in the constants, e.g. using 4R in place of D. It is worth noting that flow in unlined excavated surfaces has a high Reynold's number ($\pm 10^6$), which for flow in channels is given by the ratio (V*R)/v. A rough surface shows a higher friction factor, f, than a smooth surface does. The orientation of the discontinuities has a pronounced affect on the frictional resistance to flow. That means that a surface sloped with the dip direction of the major discontinuity against that of the flow has a higher head loss, h_f(m), than a surface excavated in the flow direction (Fig.5.6) (Reinius, 1986).

After the lifting and the removal of rock blocks by scouring, an unlined spillway often has a rough area below the edge of the overflow sill and sometimes further downstream of it. Further spills are able to quarry the rock even more effectively and if they are unchecked, they could destroy the overflow sill and work back to release the entire contents of the reservoir.

a



b

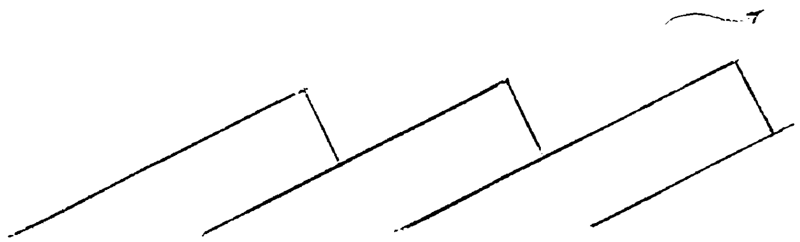


Fig.5.6 Direction of the major discontinuity with that of the flow (a) and against that of the flow (b).

The relevant physical characteristics of an unlined channel are:

- the length, L(m) and width, W(m), of the excavated portion of the channel;
- the slope or gradient of the channel, S, usually expressed in metres per metres, which is the vertical drop in elevation over some horizontal distance;
- the cross sectional area, A(m^2), meaning the width of the channel multiplied by the depth of water flowing;
- the wetted perimeter, P_w(m), that is the measured distance of contact between the water and the channel cross section, measured at right angles to the direction of the streamflow,
- the hydraulic radius, R(m), which is the ratio of the cross sectional area divided by the wetted perimeter.

Any changes of the width, gradient and direction along the channel must be indicated.

In the areas downstream of arch spillway walls, plunge pools might have already developed. The radius, the cross sectional area of the pool and the tailwater depth are important characteristics.

5.3.3 Characteristics of the water jet.

The characteristics of the water jet include the flood frequency (times/year) of the spillway during its operational history, the maximum discharge capacity (m^3/s), the volume (m^3) and the duration (hours) of the actual historic maximum discharge as well as the water height (m) above the spillway crest, the velocity (m/s), the force (N), the pressure (Pa), the power (W) and the energy (J) of the water, the angle (^{o}) with which the water enters the downstream area and the hydraulic attack (volume of discharged water/bottom width of the spillway).

Hydrological studies of catchment areas are complex and often difficult, involving the measurement of intrabasin precipitation, evaporation, infiltration, transpiration, percolation and storage. Flood frequency and magnitude are estimated on the basis of measurements over as long a period of time as is possible for a given basin. High flows at frequent intervals, i.e.once a year, can require that the engineering design of the spillway be sophisticated, particularly with respect to energy dissipators and the construction of a stable downstream channel grade, often at considerable cost.

Hydraulic attack is a factor developed by the U.S. Soil Conservation Service (1973) (Cameron et al.,1988) as V_Q/W where $V_Q(m^3)$ is the total volume of outflow through the spillway and W(m) is the bottom width of the spillway. The term serves to incorporate volume and duration of the flow, and is used to normalize flows at different sites.

Knowing that the energy transmitted to the channel, E(J), is a function of the applied power of flow, P(W), and time of exposure to it, the frequency of operation and the duration of spill, T(s), are important considerations yielding the energy factor,

$$E = P * T. \qquad (22)$$

Power is related to the hydraulic force, F(N), and the velocity, V(m/s), of the water acting on the rocks and/or their derived soils within the channel and to the amount of rock that may be scoured during every flow event. The theoretical flow velocity at any point is determined by the head drop from the reservoir level to the end of the concrete lining and the head drop to the downstream end of the excavation. Laminar and turbulent flow represent the two conditions of fluid movement. The velocity varies with the depth and is at a maximum at the surface. The actual velocity will be less because of friction losses which depend on the roughness and the slope of the discharge path. Two equations for calculating the flow velocity are as follows:

$$V = (2 * g * H)^{1/2} \tag{23}$$

or

$$V = Q / A \tag{24}$$

where $g(m/s^2)$ is the acceleration of gravity, H(m) is the head drop from the reservoir to the unlined bedrock, $Q(m^3/s)$ is the spillway discharge and $A(m^2)$ is the cross sectional area.

The hydraulic force, F(N), with which the water impacts an area, $A(m^2)$, is given by:

$$F = \rho * H * A * g = o * Q * V \tag{25}$$

where H(m) is equal to $V^2/2g$, ρ is the density of the water ($1\,000 kg/m^3$).

The pressure, p(Pa), developed by the water is related to the water velocity,

$$p = \rho * g * H = \rho * V^2 / 2 \tag{26}$$

where H(m) is equal to $V^2/2g$ and ρ is the water density, $1\,000 kg/m^3$. The pressure depends on the sinθ, where θ is the angle that the rock surface makes with the water jet. If the angle is 90^o, the pressure is direct and if the jet is parallel to the surface of the rock, shear stresses are developed. Reinius (1986) indicated that the full dynamic pressure head, calculated in kilo-Newtons per square metre (or kPa), is as follows:

$$p = (\rho * c * V^2) / 2 \tag{27}$$

where c is a pressure coefficient and can be up to 0,67.

The power of the water, P(W), in the unit of time that the water needs to reach the downstream area after leaving the crest of the spillway wall, t(s), that is being applied on the downstream area, is as follows:

$$P = (F * H)/t = (\rho * Q * V * H)/t = \rho * Q * H * g \qquad (28)$$

because g=V/t. The power which is necessary for the initiation of the scour is called the threshold power. It differs for each type of rock and is related to the cohesion and strength of the rock.

The effects of the power with which the water attacks an unlined rock mass are impressive. This is illustrated by the use of water jets for the excavation or quarrying of rock. At Nagawado Dam, in Japan (Fujii, 1970), 20 000m^3 of fault material was sluiced away by a high pressure water jet, at about 2m^3/min, under 100kg/cm^2, through a nozzle of 15-17mm in diameter. The velocity was near 180m/s and the head of the water about 1 000m. The rate of excavation was controlled at 10m^3 per day, depending upon the geological condition. Even though the discharge was low, only 0,03m^3/s, the result on the rock mass was similar to that which occurs downstream of spillways, when the water velocity and the head is much less. The reason is that the power is proportional to the water discharge which is great in flows over spillways, so that the value becomes equal to the power developed by high head jets of small discharge.

5.3.4 Water impact on a jointed rock mass.

One form of failure of both surface and underground excavations is damage to the surface by erosion. Gravity does not induce a sliding component of force on rock blocks resting upon horizontal or near-horizontal discontinuities. Displacement of such blocks can therefore not take place, unless forces due to water pressure or other agencies are sufficient to overcome friction along their bases. This is likely to be critical when running water is allowed to travel along the surface of altered or weathered rocks or poorly cemented sediments or jointed rocks.

The transporting power of a running stream depends on two principal factors; the volume and the velocity of the stream and the specific gravity, shape and size of the fragments handled. Velocities above 20m/s produce such shear stresses that abrasion and cavitation erosion of the surface takes place. The high dynamic pressures created as a result of high velocities and discharge of the water can however increase scouring when they penetrate joints and cracks in the bedrock and succeed in lifting out blocks of rock defined by joints. If the discontinuity surface is initially cemented or rough, a finite value of shear stress will be required to cause sliding when the normal stress is zero. This initial value defines the cohesive strength of the surface.

The condition of incipient movement for an assembly of cohesionless and solid blocks or particles is described in terms of the forces acting on them by the following relation:

$$\tan\phi = F_t \;/\; F_n \qquad (29)$$

where F_t,F_n are the forces parallel and normal to the direction of the shear force and ϕ is the angle of internal friction. Here, F_t and F_n are resultants of the hydrodynamic drag F_D, the uplift force F_L and the submerged weight W. Thus, the previous equation may be rewritten as follows (Graff,1971):

$$\tan\phi = (W\sin a + F_D) \;/\; (W\cos a - F_L) \qquad (30)$$

where a is the inclination of the bed. In addition,

$$F_D = C_D\, k_1\, (d^2\, \rho\, u_b^2 \;/\; 2) \qquad (31)$$
$$F_L = C_L\, k_2\, (d^2\, \rho\, u_b^2 \;/\; 2) \qquad (32)$$

$$W = k_3\, (\rho_s - \rho)\, g\, d^3 \qquad (33)$$

where u_b is the fluid velocity at the bottom of the channel,
C_D,C_L are drag and lift coefficients respectively,
d is rock block or particle diameter,
ρ is water density,
ρ_s is the solid-particle density
and k_1,k_2, k_3 are particle shape factors.
For cohesive materials, the cohesive force F_C is added to F_D.

Due to the forces developed during water discharge on jointed rock masses along and downstream of spillways, rock blocks might move downstream causing erosion. When water impacts on the side of an exposed block formed by a joint plane, the force may be reduced or not, depending on the angle between the direction of the water force and the joint orientation. A simplified approach to the calculation of the developed forces follows:

The calculation of the component (F_1) of the water force, in a direction parallel to the dip direction of the joint, is

$$F_1 = F \cos a_1 \tag{34}$$

where a_1 is the angle between the water flow direction and the joint dip direction and $0^o \leq a_1 \leq 360^o$ (Fig.5.7). For $a_1 > 270^o$ and $a_1 < 90^o$, water is flowing with the joint dip direction, but for $90^o < a_1 < 270^o$, water is flowing against the dip direction of the joint. In addition, for $a_1 = 90^o$ and $a_1 = 270^o$, water is flowing perpendicular to the joint dip direction.

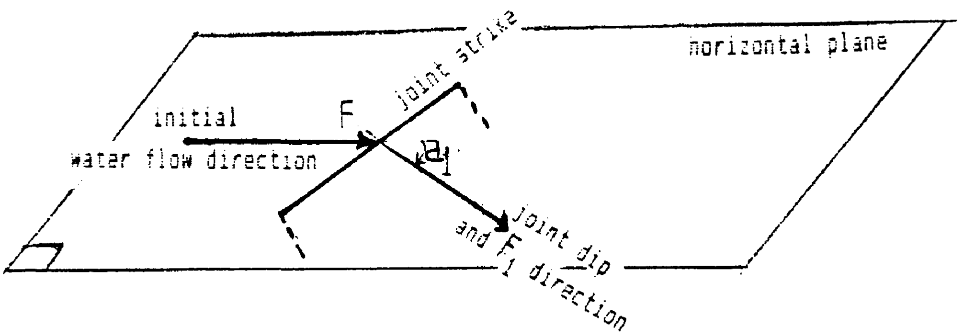


Fig.5.7 View on a horizontal plane.

The component of the water force (F_2) normal to the joint plane along the joint dip direction may be calculated from the following equation

$$F_2 = F_1 \cos a_2 \tag{35}$$

where F_1 is the previously calculated force, positive or negative according to the relation between the water flow direction and joint dip direction, and a_2 is the angle between the plane of the water flow and the normal to the joint plane (Fig.5.8) and which has values of between 270 and 360 degrees.

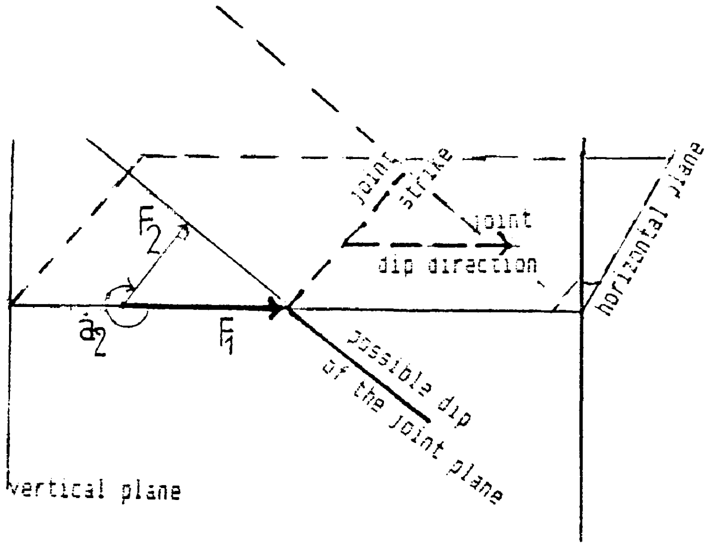


Fig.5.8 View at right angles to joint plane.

From the above calculations, the following observations can be made:

- when water flow is parallel with the joint strike, then the component of the water force will be zero, whatever the dip is,
- when water is flowing at right angles to the joint strike, then the component of the force is equal to the initial water force multiplied by 1 (if the water is flowing with the joint dip direction) or by -1 (if the water is flowing against the joint dip direction). In addition, when the angle between the joint plane dip and the plane containing the water flow direction is zero, the component of the water force will also become zero, whereas the greater the dip angle, the greater the absolute value of the force will be. This value is positive when the water is flowing with the joint dip direction and negative when water is flowing against the joint dip direction,
- when the joint plane is oblique to the water direction, e.g. when the angle between the joint dip direction and water flow direction is between 271^{o} and 359^{o} or between 001^{o} and 089^{o} (water flow is directed with the joint dip direction), and from 091^{o} to 179^{o} or from 181^{o} to 269^{o} (water is flowing against the joint dip direction), the more parallel the joint dip direction and the water flow direction and the more vertical the joint plane, the greater the value of the force will be. The value of the force will be positive when water is flowing with the joint dip direction and negative when water is flowing against the joint dip direction.

If there is a line or a plane, usually of the major discontinuity set, along which erosion due to sliding of the rock block may occur, then the water force normal to a joint plane should be analysed and estimated along this specific direction of the plane. Supposing the potential sliding plane is dipping at a_3 degrees difference from the joint dip direction, the force component F_3 parallel to the failure direction may be calculated as follows:

$$F_3 = F_2 \cos a_3 \qquad (36)$$

where $0^o \leq a_3 \leq 360^o$ (Fig.5.9).
If the potential sliding surface is between 271^o and 359^o or 001^o and 089^o from the joint dip direction, the component of the force is positive, therefore greater than if the potential sliding surface dip direction is against that of the joint dip direction. If the potential sliding direction is at right angles to the joint dip direction then the force will be zero. The force will become greater, the more parallel the dip direction of the joint and the dip direction of the failure plane are.

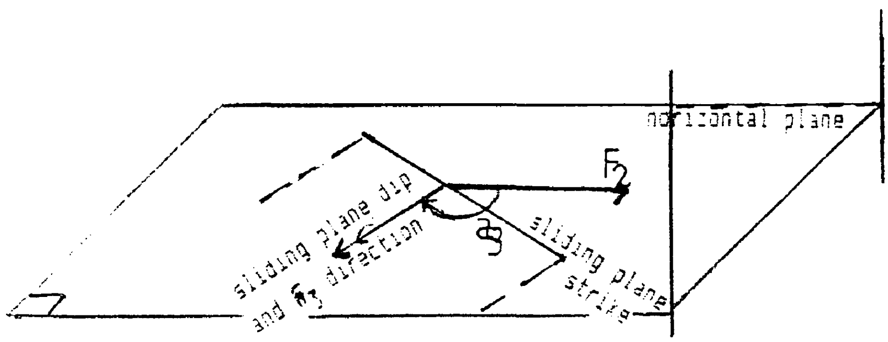


Fig.5.9 View on a plane normal to joint plane.

To calculate the force F_4 parallel to the potential sliding plane and along the direction of sliding the following equation may be used

$$F_4 = F_3 \cos a_4 \qquad (36)$$

where a_4 is the dip angle of the potential sliding surface, having values between 0^o and 90^o (Fig.5.10).
If the potential sliding surface is at right angles to the joint plane, the force is the greatest. If the potential sliding surface is parallel to the joint plane, the force will be zero.

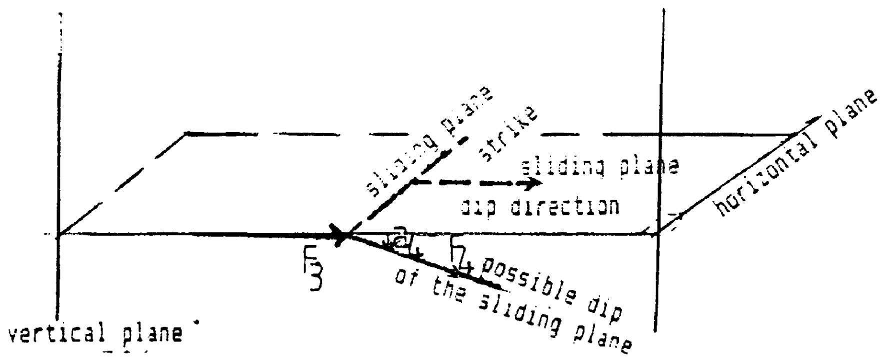


Fig.5.10 View at right angles to sliding plane.

Finally, the force acting on the sides of an exposed block formed by a joint plane may be calculated as follows:

$$F_4 = F \cos a_1 \cos a_2 \cos a_3 \cos a_4 \qquad (37)$$

5.3.5 Conclusions.

Water in contact with a rock mass influences its stability in several ways; it reduces frictional shear strength due to the influence of buoyancy which causes a reduction in the normal load; reduces cohesive strength in weak to medium strength clayey rocks, fault gouge and infill materials; increases seepage forces when these act in the direction of flow; changes the rock or infill chemistry if seeping water causes solution, chemical change or base exchange (Turk et al., 1985).

To initiate the erosion process, the power of the water should reach the limit above which the resistance of the rock mass will be passed. The power of the water is a parameter related to the water discharge, the velocity and the head of the water. The higher their values, the greater the power of the water will be. If the total volume of erosion (m^3) had been available, correlation with the value of power might have given useful results and conclusions. In Table 5.11, it is shown that the longer the spill duration, the greater the extent of erosion will be. This follows from the concept that energy is proportional to the power and the time. Thus, the longer the spill of the water lasts, the greater the energy available for the removal of the rocks will be, hence the larger the total volume eroded will be.

Parameters	Dams visited											
	KAMMANASSIE	KOOS RAUBENHEIMER	DONKERPOORT	HANS STRYDOM	MARICO BOSVELD	HARTEBEESPOORT	WAGENDRIFT	CRAIGIE BURN	ROODEPLAAT	GOEDERTROUW	VYGEBOOM	GAMKAPOORT
maximum spillway capacity (m^3/s)	2 830	935		1 716	1 440	2 322	1 300	340	970	7 000	2 617	4 531
length of spillway crest (m)	183		42,5	192	130	130	120	121,9	143	160	183	100,6
width of the chute (m)						38,6	80	75			30,4	55
length of the flip bucket (m)										160	41,5	16,5
vertical drop from spillway crest (m)		0,0	5,3	1	3,59	4,6	35	35,3	50,4	10,5	42,3	37,5
vertical drop from the flip bucket exit (m)										7,0	2,7	2,7
erosion depth (m)	1,50	0,60	0,30	0,30(7,00)	10,0	1,50	1,50	0,80	0,3(1,9)	3,00	0,25	2,0
flood frequency (times/year)	1/66	1/18		1/9	1/2		23/24	19/29	17/19	1/7	12/12	6/17
maximum daily discharge (m^3/s)		62,6		78,6			359,2	169	207	589,5	70,9	274
peak outflow (m^3/s)						825	687,1	366,5	971		129	499
volume of outflow (m^3)	84,3			257,4	69,0	103,0	88,5	28,4	125,5	73,9	224,5	57,5
duration (hours)	4 392	216		7 032	4 368	98	140	110	8 568	95,5	2 640	144
power of the water flow ($\times 10^6$ W)				0,09	0,15	13,17	60,25	24,82	48,55	14,89	0,62	2,93

Table 5.11 Hydraulic parameters of dams visited in the R.S.A.

Roughness and gradient of the downstream area are important to the erosion process. Large values of the head drop of water (greater than 50m), in addition to a gradient of the discharge path greater than 10^{o} increase the possibility of erosion, while areas with less than 5^{o} gradient have a low scour potential (Fig.5.11) (Woodward, 1984). In particular any locally steep section of the discharge path of highly erodible material, i.e.fractured, weathered or weak rock mass, could be the site for significant local scour.

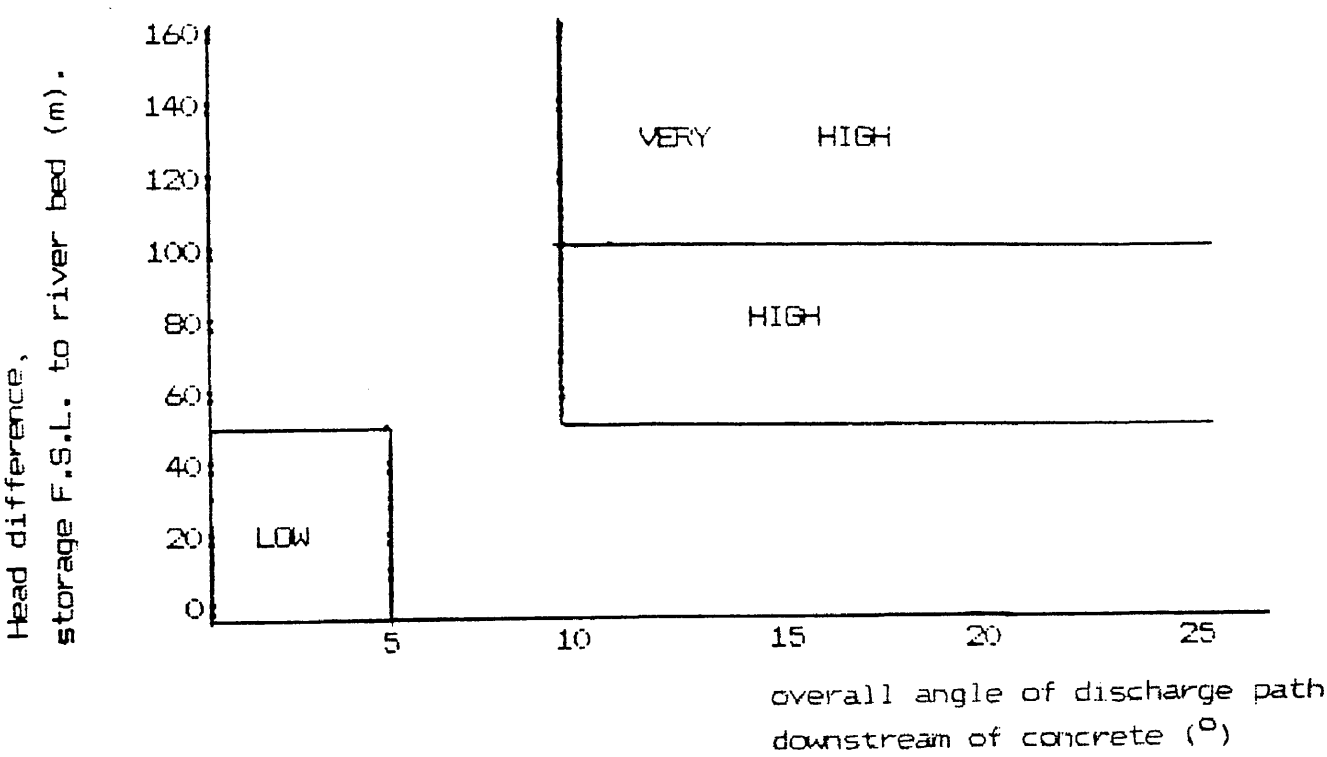


Fig.5.11 Potential of scour according to the vertical drop from F.S.L. to downstream bedrock level and to the slope of discharge path (Woodward, 1984).

Supposing that a rock block formed by joints could slide along a surface or a line of intersection parallel to a horizontal water flow direction, it could be concluded that

- no water pressure will occur on a joint set "striking parallel with the water flow direction", or a joint set "striking at right angles to the water flow direction but dipping at zero degrees" or a joint set "striking obliquely to the water flow direction but dipping at zero degrees". Nevertheless, the last two types of joint sets may serve as planes along which shearing and sliding will occur,
- the component of the water force, along the direction normal to the joint plane will have a maximum value when the joint is striking perpendicular to the water flow direction and is dipping vertically,
- the component of the water force becomes greater and positive, the smaller the angle between the joint dip direction and the water flow direction and the more vertical the joint dip angle is, when the water is flowing with the joint dip direction,
- the component takes negative values when the water flow is directed against the joint dip direction; the smaller the angle between the water flow and the joint dip direction, and the greater the joint dip angle is, the greater the absolute values of the force will be,
- positive and greater values of the component of the water force result when a joint plane and the major discontinuity set which acts as a potential sliding surface, are dipping with the water flow and when a joint plane is dipping against the water flow but the potential sliding direction is along with the flow. Accordingly, when the potential sliding direction is dipping upstream, regardless of the joint dip direction, the force will be negative. The potential for failure of the rock mass is therefore greater if the potential plane or line of sliding is dipping downstream.

The determination of the pressure distribution in a jointed rock mass requires a thorough knowledge of the geology, an understanding of the fundamental laws which govern the flows of the fluids in jointed rock masses, a knowledge of the methods of measuring and estimating the permeability of a jointed rock mass and a familiarity with the analytical techniques available to calculate pressure distributions.

CHAPTER 6 EROSION IN UNLINED SPILLWAYS OF DAMS IN THE R.S.A.

6.1 Introduction.

Through back analyses of spillways where erosion has occurred, an attempt is being made to characterise the rock mass in terms of erodibility.

The investigation of cases in the Republic of South Africa included visits to more than 13 dams. The purpose of the visits was to provide field data on the geology of the downstream area which suffers from the erosion due to water discharge.

The design of a spillway depends on the topographical and geological conditions of the surrounding area and the dam structure. The variety of spillway types encountered, required the grouping of structures according to the spillway operation. Even so, spillways of the same group have their own characteristics that make every case unique.

Geological factors such as stratigraphy and lithology influence the individual behaviour during water discharge. The geotechnical properties of every rock type, in general, but also in the particular area, dictate the response to the water forces and furthermore to the erosion process. The main engineering geological property upon which field surveys have been conducted is jointing of the rock mass. A classification of the rock mass to provide an indication of the rock mass quality, is envisaged.

The hydrological history of the spillway discharge is necessary to determine any correlation between the extent of erosion with the maximum outflow discharge, the volume and the duration of the discharge, the energy and the velocity of the water flow, the frequency of operation and other hydraulic factors.

6.2 Overflow spillways.

6.2.1 Kammanassie Dam.

The dam was completed in 1923 for irrigation purposes. It is situated on Kammanassie River, to the east of the town of Oudtshoorn, in the Cape Province (Plate 1).

It is a gravity dam, 41m high above lowest foundation level and 389m long. The gross capacity of the reservoir is 32 $900 \times 10^3 m^3$ and reservoir area is 3 $486 \times 10^3 m^2$.

The dam has two spillway structures, a service spillway and an emergency one. The maximum discharge capacity of the spillways is 2 $830m^3/s$. The service spillway is a side channel which is located to the right - looking downstream - of the main dam wall, discharging water into an unlined channel (Plate 2).

The bedrock of the spillway channel is strong but well jointed greywacke of the Ceres Formation of the Bokkeveld Group. Greywacke is a type of sandstone with fine-grained matrix comprising between 15 per cent and 50 per cent of the rock, and of 25 per cent feldspar. Its properties are quite similar to those of sandstone, with compressive strength between 40MPa and 200MPa and dry unit between 1 $900kg/m^3$ and 2 $600kg/m^3$, depending upon the composition, the cementing and the degree of weathering of the rock. They are dark grey-green rocks.

Preliminary investigation by Du Toit (1920) at the dam site showed that the hard and potholed greywacke with clayey seams is resting upon a soft black slate layer of several centimetres thick, and which included some quartz veins, pyrites and cavities. A small fault occurs at the site. The slates exposed at the bottom of the cut-off trench are black, quite fresh, well-bedded but jointed. Along the left flank, the rock is solid near the surface, whereas along the edge of the hillside, it exhibits extensive jointing to a depth of 9m. On the right flank, beds of rock are remarkably fresh and hard with thin slatey layers.

The rock is weathered and more than six main joint sets exist. The orientations of the joints are slightly different upstream and downstream of the bend in the concrete wall. Water flow is

Plate 1 Kammanassie Dam (Dept of Water Affairs).

Plate 2 Erosion of the rock mass in the unlined spillway channel (Van Schalwkyk).

following the strike of the joints or the line of intersection between joints. There is sliding along some unfavourably orientated joint planes and wedge failure also occurs (Fig.6.1).

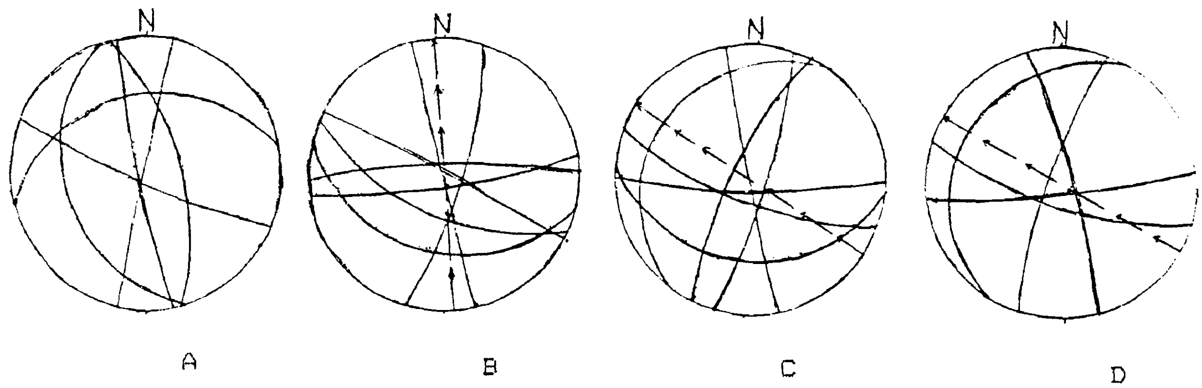


Fig.6.1 Stereographic projection of the discontinuity planes (--) and the water flow direction (-- --).
A-Projection of discontinuities on the right flank of the unlined channel.
B-Projection of discontinuities and water flow along the part of the channel, immediately downstream of the spillway.
C-Projection of discontinuities and water flow near the turn of the left concrete wall of the channel.
D-Projection of discontinuities and water flow downstream of the concrete road, crossing the unlined channel.

The direction of the joints on the right side wall of the channel is similar to those of the bedrock in the channel floor. A large number of sub-horizontal joints are open and contain sandy filling material with thickness of about 200mm. It seems that shearing has taken place along the sub-horizontal direction which caused the highly jointed nature of the rock mass.

Along the upstream part of the channel (upstream of the bend), the rock mass is well jointed, containing six main joint sets as well as discontinuous undulating random joints. The RQD is about 5 per cent. The joints are closely spaced (approximately 300mm, but sometimes less than 100mm). The sub-vertical joint sets are usually tight or have a separation of about 1mm and display stained surfaces. The sub-horizontal joints and those joints that dip at 20^{o}-30^{o}, are open and filled with crushed, loose, sandy material. The joint walls are rough to slightly rough and undulating. Some joints are moist.

Further downstream, near the bend in the side wall, erosion to a maximum depth of about 1,5m has developed. The condition of the jointing - RQD, spacing, separation, alteration, and roughness - remains the same but the direction of the water flow changes abruptly from place to place.

Downstream of the concrete roadway, the channel steepens and the prevailing joint set is the one with the smallest dip (about 14^{o}). Erosion reaches about 1m depth in certain areas. All the joints are tight and closely spaced, except one joint set that has a separation of more than 5mm and a spacing of approximately 300mm. Thin veins of quartz occur along some of the joints and are orientated perpendicular to the subhorizontal set.

Testing the rock mass for hardness, on the right side and along the channel floor, using the Schmidt Hammer, a mean value of 49 was recorded. The rebound numbers were correlated with the unconfined compressive strength and the modulus of elasticity. According to the various equations, the following range of values was calculated:

- for compressive strength, σ_c(MPa), 40-225MPa, and
- for modulus of elasticity, E_t(GPa), 32-85GPa.

Monthly records of the outflow discharge are available which show the spillway discharge since October 1922 till September 1981. The spillway did not overflow every year, but only 11 times during the above period. The maximum overflow volume of $84,3x10^6m^3$ occurred in 1954, when the duration of overflow was 6 months. During August 1954 alone, a water volume of $65,20x10^6m^3$ was discharged. The frequency of the spillway operation, according to the hydrological data, is about 1 in every 5 years (Fig.6.2).

Erosion occurs mainly near the bend in the left side wall of the channel and downstream of the concrete path further downstream (Plate 3). At present, it does not endanger the spillway. Downstream of this concrete path, the slope of the channel is steeper and the left wall changes from concrete to earth. When water is flowing along the upper part of the spillway channel, it impacts mainly on joint surfaces that dip between 30^{o} and 90^{o} and strike almost parallel to the water flow direction, but also on some joints that strike perpendicular, but dip against the flow

KAMMANASSIE DAM

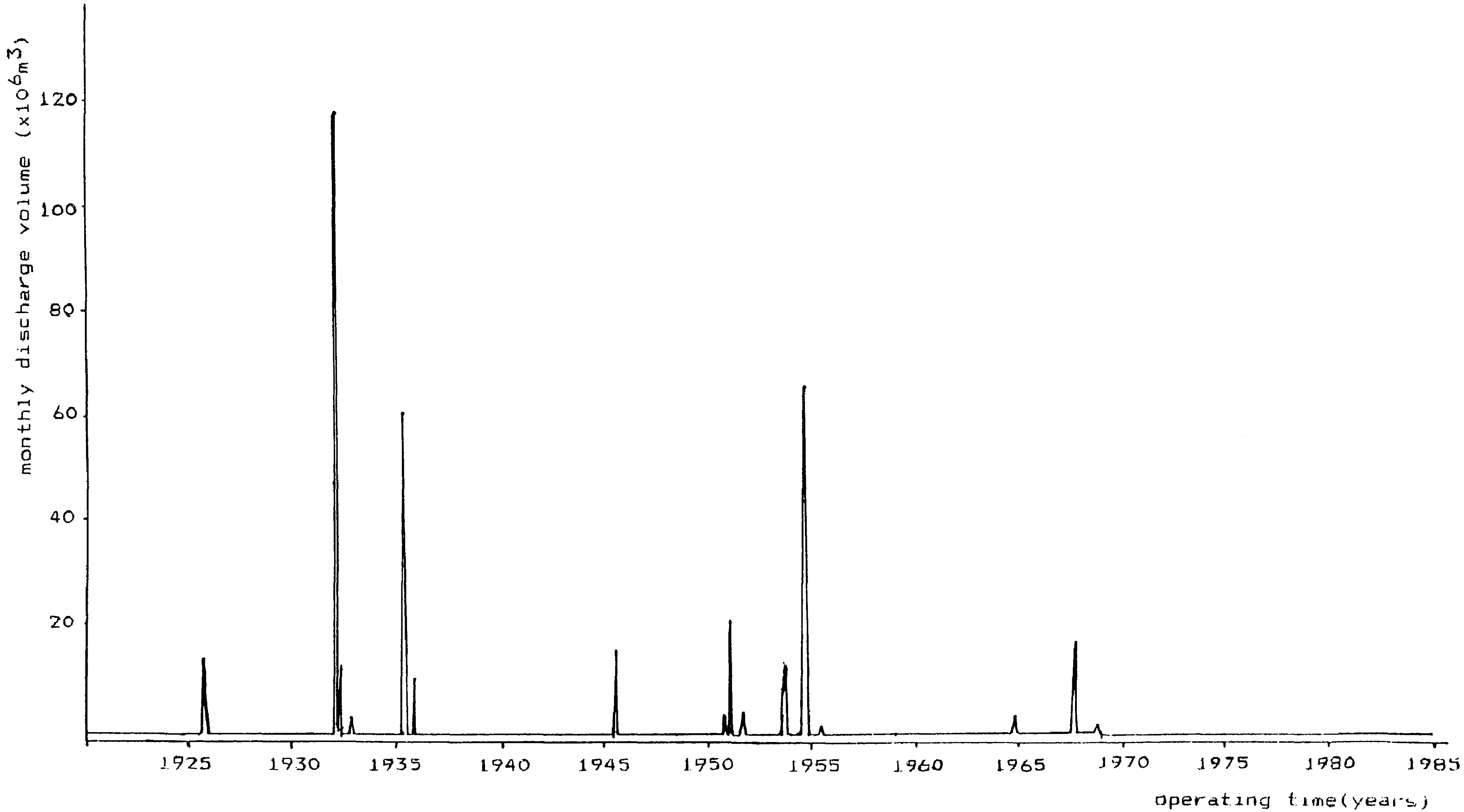


Fig.6.2 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

direction. Only one joint set strikes normal to the water flow direction and dips downstream, almost vertically. Downstream of the bend, water impacts on the right side-wall of the channel and is then forced to change direction from azimuth 356^{o} to 301^{o}. Then water flows along joints, some of which strike parallel to the flow direction and are almost vertical, dipping towards the left concrete wall, and others that strike diagonally to the water flow, creating wedges with one joint set that strikes perpendicular to the flow direction and dips with it at a small angle of 14^{o} to 15^{o}. Therefore, downstream of the bend, conditions become unfavourable and erosion is of a greater extent. Joints, that strike parallel but dipping at opposite directions and microfold structures show that tectonic stresses have acted on the rock mass, lessening its strength and creating shear and stress relief joints.

Plate 3 Erosion in the area downstream of the concrete road crossing the spillway channel.

6.2.2 Koos Raubenheimer Dam.

It is an earthfill dam, situated on Klein Le Roux River, close to the town of Oudtshoorn, in the Cape Province. It was completed in 1971. It has been constructed for water supply and irrigation of the surrounding area.

The main dam wall is 49m high above lowest foundation and 247m long. The gross capacity of the reservoir is 9 $260 \times 10^3 m^3$ and the reservoir area is $533 \times 10^3 m^2$. It has two spillways. One is a shaft spillway, where the overflow water drops vertically through a funnel and shaft and is conducted downstream more or less horizontally in a concrete pipe. The other spillway is an emergency one, situated on the right side of the embankment. It is an uncontrolled spillway, discharging water in an unlined excavated channel (Plate 4). The slope of the channel is initially zero, but steepens further downstream. The maximum spillway capacity of both spillways is $935 m^3/s$.

The bedrock in the unlined channel is highly to moderately weathered, grey sandstone, with interbedded zones of clayey material, intersecting the spillway channel. The thickness of the clayey beds varies from 500mm to 10mm, and they strike across the channel and dip between 55 and 80 degrees in a 226^o direction. They occur on the flanks and in the bottom of the channel (Plate 5). It is estimated that the sandstones have a dry unit weight of between 1 900 and 2 $600 kg/m^3$ and a uniaxial compressive strength of between 40MPa and 120MPa.

The rock is very well jointed and contains closely spaced ($\pm$10mm) open joints with up to 5mm separation. Five main joint sets were observed. Soft filling material, occasionally clayey, exists in places where clayey zones are close to the joints. The orientation of the joints on the right side and on the floor of the channel are shown in Fig.6.3. Hardness of the sandstone tested with the Schmidt Hammer gave a mean rebound number of 40 which indicates

- an unconfined compressive strength, σ_c(MPa) of between 20 and 170MPa and
- Modulus of Elasticity, E_t(GPa), of 25-70GPa.

Hydrological data are deficient. A measuring weir located several kilometres from the dam does not give information on discharge through the emergency spillway channel alone, but the combined

Plate 4 Koos Raubenheimer Dam - unlined side channel spillway.

Plate 5 Extensively jointed rock mass with interbedded zones of clayey material on the flanks and in the floor of the channel.

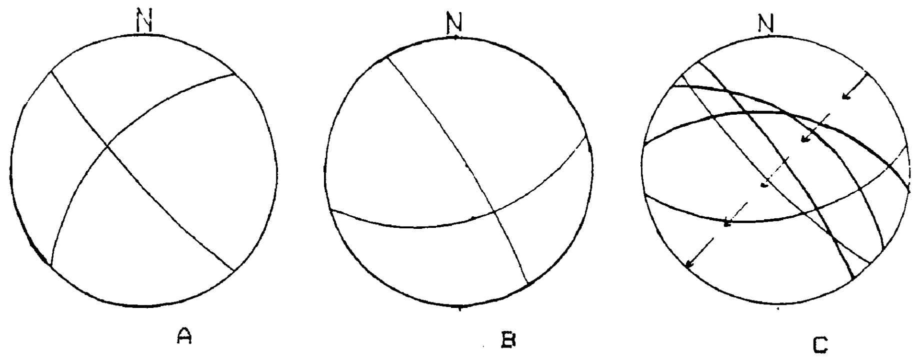


Fig.6.3 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).
A-Projection of the clayey seams direction.
B-Projection of discontinuities on the flanks.
C-Projection of discontinuities and water flow in the floor of the channel.

discharge with the service spillway. The records obtained since October 1974, exhibit a maximum discharge of 62,6m^3/s on 9th of May 1977, for a period of about nine days. There is a possibility that a certain quantity of water discharged through the channel. Till September 1989, on eleven occasions - in May 1975 (41,2m^3/s), in October 1976 (11,1m^3/s), in November 1976 (19,2m^3/s), in May 1981 (52,6m^3/s), in August 1981 (19,6m^3/s), in October 1981 (24,3m^3/s), in April 1982 (10,5m^3/s), in October 1983 (15,4m^3/s), in January 1985 (10,2m^3/s) and in December 1985 (14,0m^3/s) - the maximum flow rate surpassed 10,0m^3/s, but discharge through the emergency spillway is unlikely (Fig.6.4). The erosion in places has reached 0,60m depth, in particular where the slope of the channel is steeper.

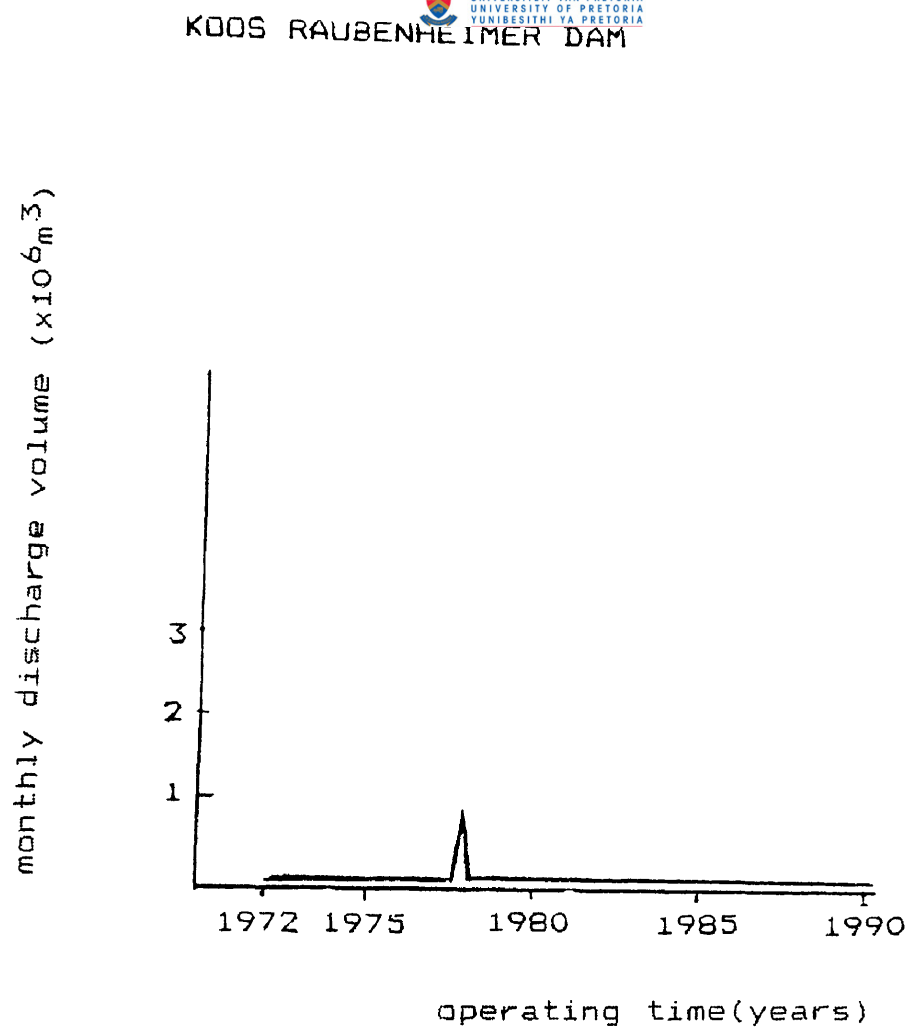


Fig.6.4 Frequency of water discarge according to hydrological information obtained from the Dept of Water Affairs.

6.2.3 Donkerpoort Dam.

Donkerpoort dam was constructed in 1959. It is situated on Klein Nyl River, near the town of Nylstroom, in the Transvaal.

It consists of a concrete arch wall, an embankment structure some kilometres on the right side of the arch and a side channel spillway on the left side - looking downstream - of the embankment (Plate 6).

The spillway is an uncontrolled, concrete overflow structure with an unlined channel. The channel width varies between 6,25m and 12,5m and is 57,75m long. The vertical drop from the spillway crest to the toe of the wall is of 5,3m.

The rocks that dominate t[illegible]d Donkerpoort dam, belong to the Nylstroom Subgroup of the Waterberg Group, and consist of Alma greywacke, Swaershoek sandstone and Wilgerivier sandstone. In particular, the rock mass consists of reddish brown, medium to coarse grained sandstone, pebbly in places with intercalated flows of trachytic lava, tuffaceous greywacke, mudrocks and conglomerate. For foundation conditions of various engineering constructions, these rocks are in general favourable. Unconfined compressive strength tests on slightly weathered silty sandstone give values ranging between 70MPa and 130MPa. It can be assumed that the coarser grained sandstone, grit and conglomerate will have strengths in the range of 50MPa to 100MPa and the dry unit weight is approximately 2 500kg/m^3 (Brink, 1981).

The rock in the unlined channel is weathered greywacke/sandstone. The mean value of the Schmidt Hammer Rebound Number to test hardness of the rock is 48, and indicates

- an unconfined compressive strength, σ_c, between 80 and 200MPa and
- a Modulus of Elasticity, E_t(GPa), of between 60 and 80GPa.

Three main joint sets and numerous random, non continuous joints occur (Plate 7). The joints are closely spaced and often open to 5mm. The joint surfaces are slightly rough. There are about 24 joints per m^3.

The lack of any hydrological data is a serious disadvantage for the study. Erosion is estimated at about 0,30m depth. The slope of the channel is nearly flat and that is a positive factor to minimize erosion. The three main joint sets strike obliquely to the water flow direction, one of them is dipping downstream at 7°, striking parallel with one of the other sets and perpendicular to the third one (Fig.6.5), so that when sliding occurs, it will be along this sub-horizontal joint.

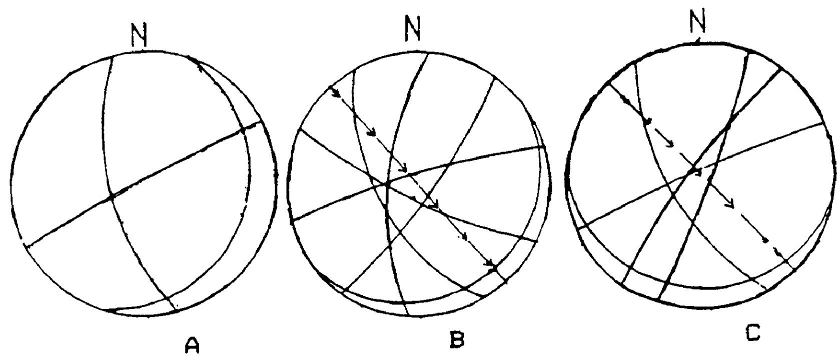


Fig.6.5 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).
A-Projection of discontinuities on the left flank of the unlined channel.
B-Projection of the discontinuities and water flow in the first position of joint survey along the unlined channel.
C-Projection of the discontinuities and water flow in the second position further downstream, along the channel.

Plate 6 Donkerpoort Dam (Dept of Water Affairs).

Plate 7 Removable rock blocks formed by sligthly continuous, undulating joints.

6.2.4 Hans Strydom Dam.

The construction of Hans Strydom Dam was completed in 1980. It is located in the Mogol River, between the towns of Ellisras and Vaalwater. The purpose for the construction of the dam is irrigation and water supply.

The height of the rockfill dam wall above lowest foundation is 57m and the length of its crest is 525m. The gross capacity of the reservoir is 153 800x10^3m^3 and the reservoir area is 8 387x10^3m^2. The dam incorporates a free standing intake tower, 66m high, a 240m long diversion tunnel of 7m diameter, constructed under the shallow cover through the left flank and an uncontrolled spillway, 210m wide, cut with a maximum depth of 40m on the right flank (Plate 8) and involved the removal of some 1,4x10^6m^3 of rock for use in the dam construction (rockfill material). The vertical drop from the spillway crest to the floor of the excavated channel is less than 1m. The spillway capacity is 1 716m^3/s while the Probable Maximum Flood is 8 000m^3/s.

Plate 8 Hans Strydom Dam - side-channel spillway (Dept of Water Affairs).

In the area surrounding the dam, the dominant rock type is sandstone of Sandriviersberg Formation of the Kransberg Subgroup, Waterberg Group. The rocks of the Waterberg Group have been subjected to several periods of faulting, the most dominant trends being NW-SE and SW-NE. The rocks are also folded along an axis trending WSW-ENE. Lithologically, the formation consists of yellowish brown coarse-grained sandstone, locally gritty, sometimes reddish, with ferruginous laminae and cross bedding planes. Intercalations of sericitized siltstone and shale indicate an upstream dip of 0^{o}-15^{o}. Considering the engineering characteristics, foundation conditions are in general very favourable. Unconfined compressive strength on slightly weathered silty sandstone give values ranging between 70 and 130MPa. The coarser grained sandstone grit and conglomerates will have strengths in the range of 50 to 100MPa. Dry density of the sandstones is approximately 2 000kg/m^3 (Brink,1981). The permeability of the moderately and highly weathered rock at this site is high but the fresh rock seems very watertight (Thomas, 1972).

Field surveys were conducted in several localities within the spillway channel. The rocks on the right and left side walls are thinly bedded sandstones. On the right wall, close to the spillway wall, there is indication of faulting and a local small anticlinical structure. Some of the faults seem to continue on the left flank. Most of the joints along the fault zones are closely spaced and orientated perpendicular to the bedding planes. Measurements using the Schmidt Hammer result in a value of 35 for hardness. The RQD is about 35 per cent and there are three main joint sets and many random joints. The separation of the joint walls is between 5mm and 10mm often with soft, clay-free filling material. The joint surfaces are rough and undulating.

The bedrock of the channel is also well jointed. Random undulating joints form different shaped and sized blocks, exposed in the erosion process. The rebound number according to the Schmidt Hammer test gives a mean value of 52,5 and is related to the unconfined compressive strength and modulus of elasticity as follows:

- U.C.S., σ_c(MPa), 50-170MPa, and
- Modulus of Elasticity, E_t(GPa), 38-70GPa.

There are three or four main joints, and the RQD is varying between 5 per cent and 55 per cent (Fig.6.6). The joint spacing is less than 300mm and the separation of the joint walls is between 1mm and 10mm. The filling material has usually been washed out, but in some bedding joints soft material from weathered siltstone or shale occurs as infilling.

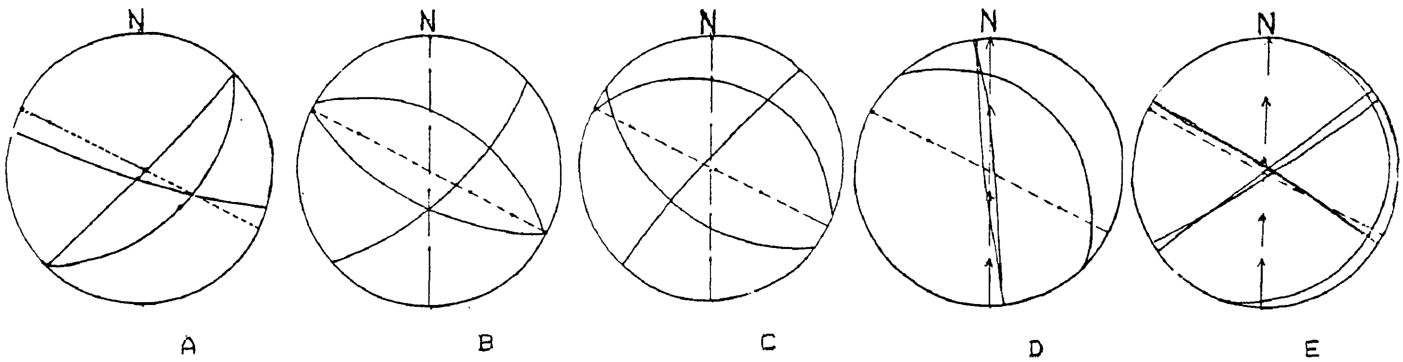


Fig.6.6 Stereographic projection of the discontinuity planes (——), the water flow direction (— —►) and the deep scoured gorge (- - -).
A-Projection of the discontinuities on the right wall of the spillway channel.
B-Projection of the discontinuities and the water flow on the downstream bedrock, close to the spillway wall.
C-Projection of the discontinuities and the water flow some metres further downstream of the previous position.
D-Projection of the discontinuities and the water flow close to the scoured gorge.
E-Projection of the discontinuities and the water flow in the area downstream of the scoured gorge.

Since the completion of the dam (1980), till October 1989, only once, in 1981, water overflowed the concrete spillway wall, according to the daily average flow-rate and the monthly reservoir records (Fig.6.7). The maximum daily discharge was 78,65m^3/s on 7 February 1981. The discharge lasted from the 10th of January 1981 till the 29th of October of the same year. February had the maximum monthly average discharge volume and also the maximum daily values. The peak outflow was not large for the design and amounted to 4,58 per cent of the maximum discharge capacity of the spillway. The duration of the outflow, however, had been for a period of 10 months and the total discharged volume was 257,41x10^6m^3.

HANS STRYDOM DAM

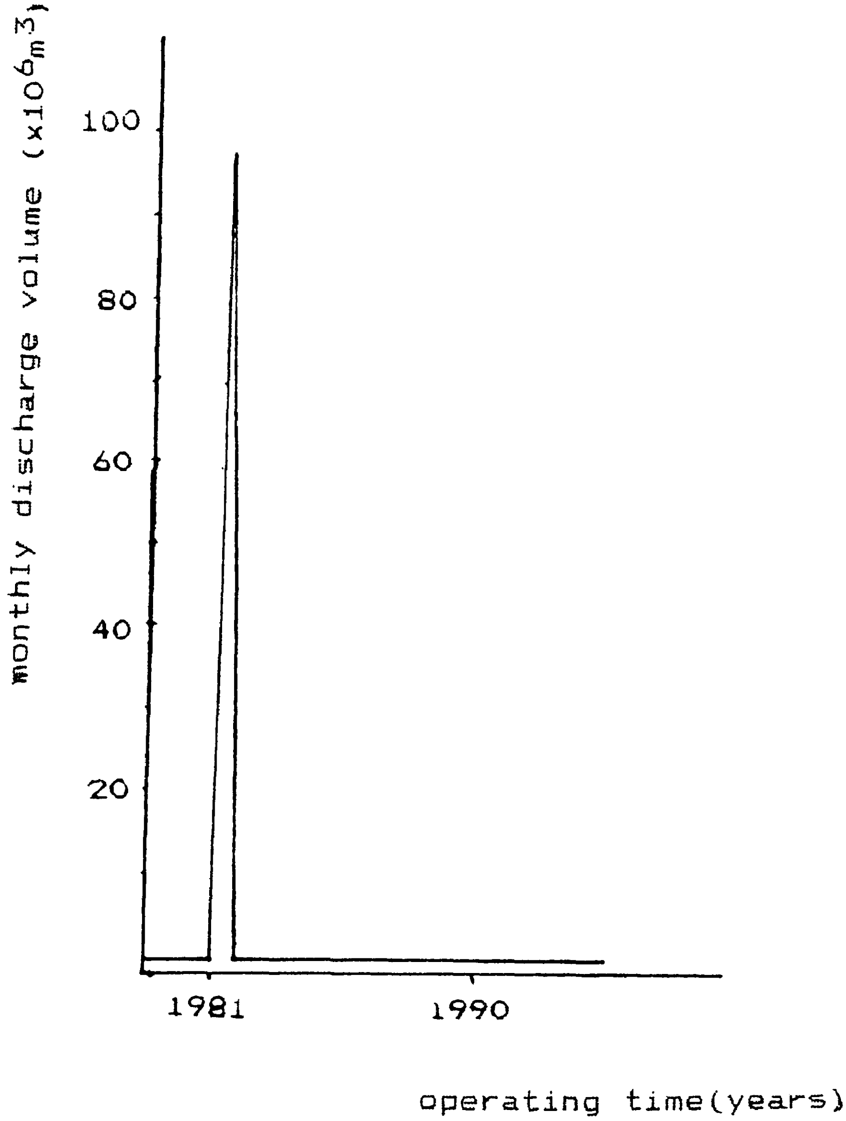


Fig.6.7 Frequency of water discharge according to hydrological information obtained from the Dept. of Water Affairs.

After this flood event, erosion was evident along the channel downstream of the spillway. The gradient changes slightly along the channel, from 2^o-3^o in the area immediate downstream to about 6^o farther where it becomes narrower. At about two-fifths of the channel's length, a large erosion gully appears almost in the middle of the channel. The 2m wide fault zone filled with brecciated material that runs diagonally (SE to NW) across the spillway, 250m from the concrete spillway wall, combined with the pronouned jointing, became the initial cause of erosion (Plate 9). It is narrow with steep sides in the beginning, becoming wider as it approaches the left side. Its direction is about 297^o

and its maximum depth is 7m. The colour of the rock is lighter brown and grey, because of the higher degree of weathering of the highly jointed fault zone along which erosion took place.

From Fig.6.6, it can be seen that joints, striking NW-SE and resulting from down-drag along the fault occur. There are joint sets striking parallel to the direction of the fault and obliquely to the direction of the water flow. Two sets occur close to the fault zone, one striking parallel with the water flow direction and the other, a sub-horizontal joint with almost similar strike. Sliding because of water pressure must have taken place along lines of intersection between beds, dipping almost parallel to the water flow, and the joints striking almost parallel to the water flow direction. Joints caused during folding and/or faulting, occur on the right side of the channel as well. The open bedding planes caused the damage when the water flow washed out the infilling material of weathered sandstone, reducing cohesion and forming loose rock blocks due to the presence of the vertical joints (Plate 10). It was indicated by Maeyens (1976) during research before the completion of the dam that the presence of subvertical joints with cohesion near zero and faults was causing instability problems at the quarry site which was to be the future spillway channel.

The faulting, presence of open, closely spaced joint sets and the long period of water flowing along vertical and sub-horizontal discontinuities, are some of the factors which gave rise to the erosion process. Faults in the channel introduce non-uniform conditions, causing concentrated movement of the water mass flowing along the channel and differential behaviour of the rock mass. Further erosion can eventually result in damage to the downstream side of the dam and the outlet work, as water is channelled along the eroded trench against the left wall of the spillway during overflow conditions. The rock on this flank is susceptible to further erosion and eventual breakthrough. Backfill downstream of the spillway wall has also been eroded, leaving foundation rock exposed. For remediation purposes, a number of concrete walls constructed across the eroded gorge (founding on solid rock) and backfilling of the compartments with the largest possible rock blocks, grouting of the voids with cement and protection of the immediate downstream area against undercutting, were proposed by Barnard (1986).

Plate 9 Scoured gorge in the spillway channel, closely related to the faults occurring on the right wall (Van Schalkwyk).

Plate 10 Closely spaced, open joints forming removable rock blocks (Van Schalkwyk).

6.2.5 Marico Bosveld Dam.

The dam was completed in 1933. It is located on the Groot Marico River, 30km east of the town of Zeerust.

It consists of an embankment with a 1 524m long crest. It is 34m high above lowest foundation, and has two spillway structures (Plate 11). The storage capacity of the dam is $27,9x10^6m^3$, while the catchment area is 1 $219km^2$. The spillway consists of two different structures, a rack-type spillway on the right side of the embankment which is 71m long and 3,32m high, and an arch wall, 13,25m high above the river bed level, further to the right, serving as an emergency spillway. The maximum capacity of the spillway is 1 $243m^3/s$. The outflow water is discharged on a water cushion, downstream of the service spillway and is then allowed to flow in an unlined channel. Both spillways have been constructed after a catastrophic flood, in 1936. Previously, the spillway consisted of a straight concrete wall, 130m long and 3,59m high above the bedrock level. The maximum capacity was 1 $440m^3/s$.

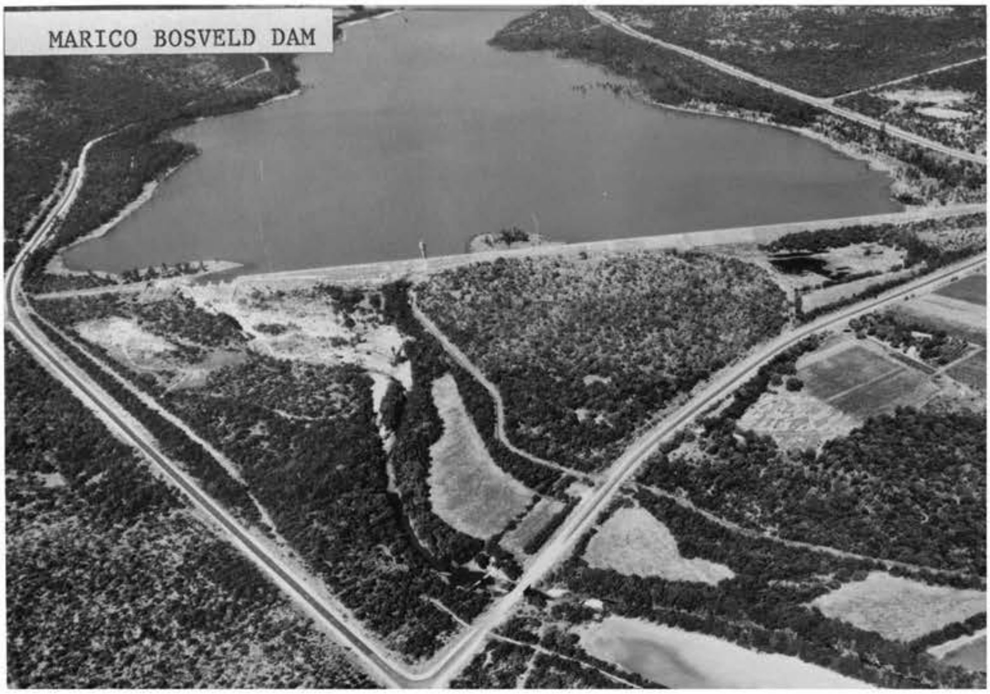


Plate 11 Marico Bosveld Dam (Dept of Water Affairs).

The lithostratigraphy of the area comprises rocks of the Pretoria Group of Transvaal Sequence. The rock at the dam-site is a whitish to yellowish medium grained quartzite, gently dipping to the North (downstream). The quartzite is the result of high degree of metamorphism. The metamorphism coupled with the highly developed system of vertical and horizontal joints has been conducive to much disintegration, leading to the rather loose, incoherent nature of the rock (Frommurze, 1930). The quartzite is found to be cut by a narrow sheet of intrusive diabase striking E-W and dipping at low angle to the North. This band is roughly parallel to the fissures exposed on the right flank (Frommurze, 1931). On the right flank a deep trench has been eroded along a completely weathered diabase dyke, 12,0-18,0m wide. A large pond of leakage water occurs in the trench.

Previous investigations indicated that leakage occurs on both flanks and in the river section at the dam's toe, possibly through the embankment and not the foundation. In the river section, water emerges from the rock surrounding the tunnel outlet, creating a large pool of water with material in suspension. Quartzites that dominate the area downstream of the spillway and the flanks, are generally considered favourable rocks for engineering purposes. The dry unit weight of quartzite is approximately 2.700kg/m^3 and the compressive strength and modulus of elasticity, derived from a mean Schmidt hardness of 31,8, is as follows

- compressive strength, σ_c(MPa), 30-130MPa and
- Modulus of Elasticity, E_t(MPa), 40-56GPa.

In the unlined channel, rock blocks of conglomerate, possibly deposited by water action, as well as quartzitic blocks, occur. The rock is very weathered and jointed; more than four main joint sets exist. The joint walls are slightly rough, undulating, altered and free from filling material. Multiple shear zones and many open fractures have been noticed (Plate 12). The RQD varies between 0 per cent, farther downstream, and 35 per cent, close to the spillway wall. Medium flow is observed between some joint wall contacts.

Hydrological data includes daily average flow-rate records from January 1978 till December 1988 and monthly average outflow volume records from October 1935 till September 1988. The total volume of the spillway discharge since February 1936 until the dam breached (Plate 13) in November 1936, was $69{,}081 \times 10^6 m^3$.

From the available plans of the dam, it is estimated that the maximum depth of erosion during the period of the damage reached 10m, whereas the minimum depth was 3m. Even though quartzites are considered as a very resistant rock type, the condition of jointing and weathering alter its strength and its resistance to erosion. Sub-horizontal joints dipping downstream, vertical joints parallel to the water flow direction, the separation of the joint walls that permits the penetration of the water and reduces cohesion, the long period during which the water overflowed, represent unfavourable conditions for resistance to the erosion process.

After the damage had been repaired, the spillway has overflowed many times (Fig.6.8). In 1976 and in 1978 the total discharge volume was similar to the one causing the failure.

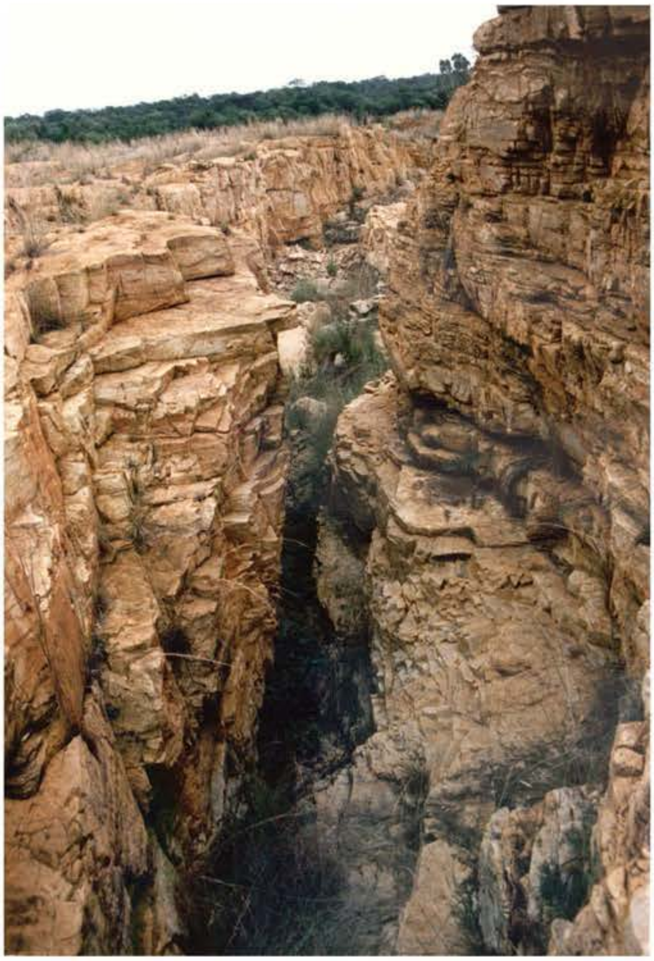


Plate 12 Jointed rock forming the sides of the scoured gorge.

Plate 13 Old spillway structure which failed in 1936 after water discharge (Van Schalkwyk).

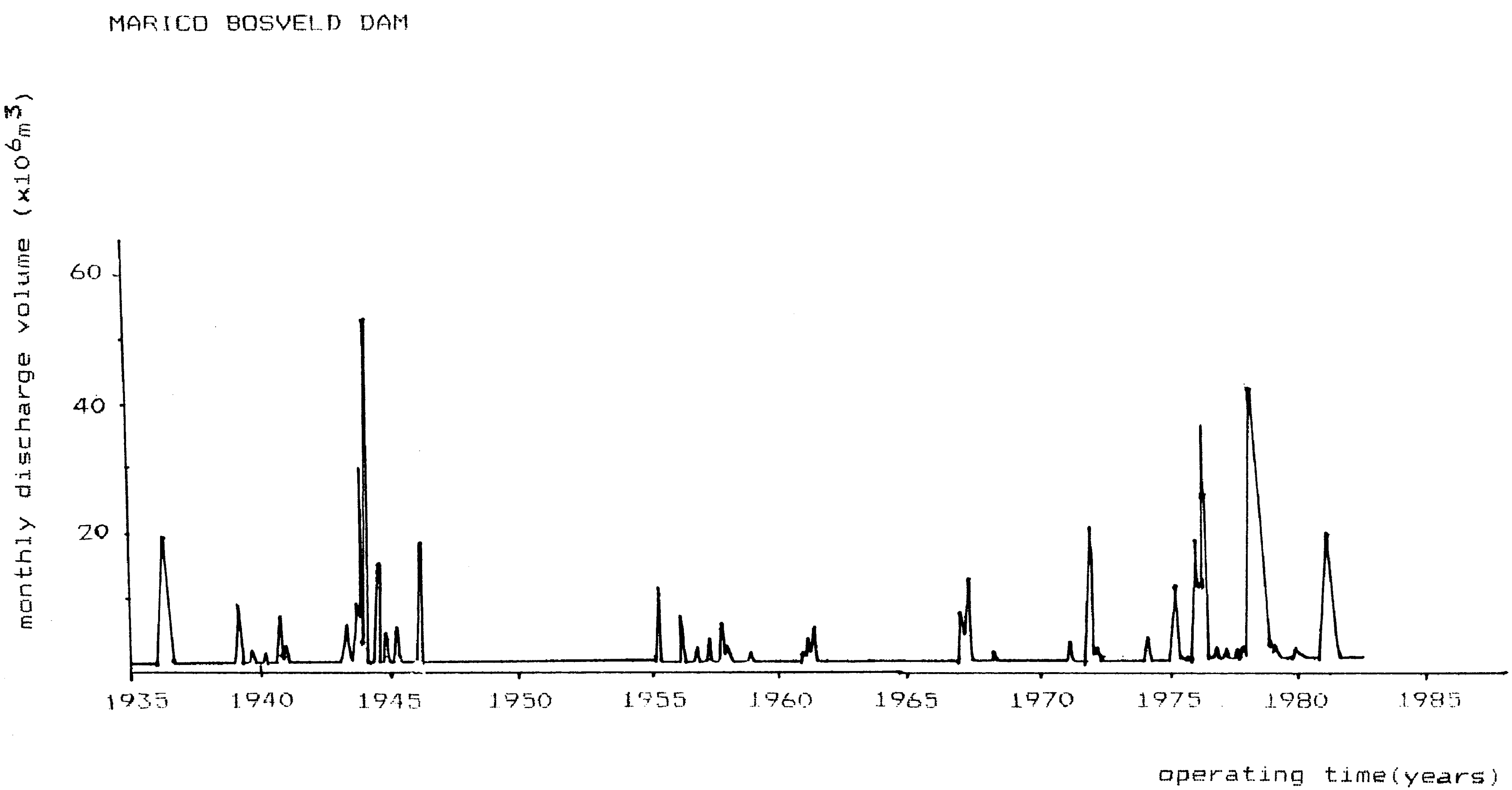


Fig.6.8 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

6.2.6 Bell Park Dam.

Bell Park Dam is a small dam, in Natal, located on the Sterkspruit River. It has been constructed for irrigation purposes of the surrounding farms.

The original uncontrolled side spillway comprised a concrete slab with baffles to protect the underlying weathered dolerite.

In the rock mass, five joint sets can be observed, some parallel and others perpendicular to the water flow direction, forming cubic rock blocks of different size. The joints are open and the joint surfaces are slightly rough and undulating. Wherever joint filling occurs, it is soft silty material. It is usually the vertically dipping joints that are open. Most of the joints, as well as their lines of intersection, dip against the water flow direction, reflecting the water backwards (Fig.6.9).

Water overflowed the spillway twice during the dam´s history. The first flood of 1987, started on the 29th of September and ended on the 3rd of October. The soil covering the downstream area had been eroded and large protruding rock boulders remained. A second flood occurred in March 1989. Protruding boulders in the unlined area, reflected water back underneath the concrete slab. Erosion of the soil below the slab resulted in dramatic settlement and complete failure of the concrete slabs (Plate 15).

Reconstruction of the spillway was done with a concrete lining of the downstream area and by filling the gaps between huge doleritic blocks with concrete (Plate 14). Wire mesh gabions were placed along the sides of the spillway.

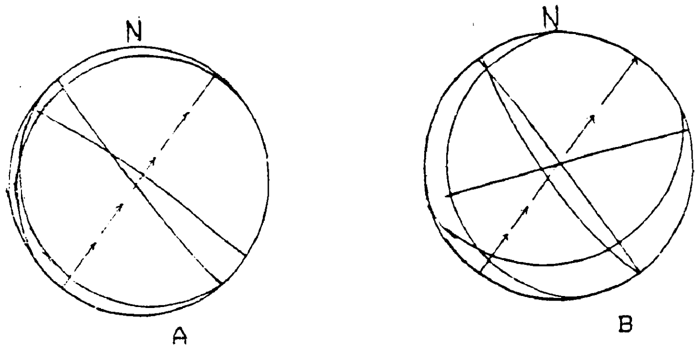


Fig.6.9 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).
A-Projection of the discontinuities and water flow close to the downstream end of the spillway slab.
B-Projection of the discontinuities and water flow further downstream of the first position.

Plate 14 Bell Park Dam - Spillway after repair work (Van Schalkwyk).

Plate 15 Damage to the spillway slab.

6.2.7 Hartebeespoort Dam.

The dam was completed in 1925. It is located on the Crocodile River, 37km west of Pretoria.

The dam is an arch structure with a side channel spillway on its left flank. In 1971, the spillway was fitted with radial gates and a partially concrete lined chute. The maximum spillway discharge capacity is 2 322m^3/s (Plate 16).

The foundation rock belongs to Pretoria Group of the Transvaal Sequence and in particular it consists of Magaliesberg Quartzites with a northerly dip of about 30^o. Ripple marks on the surface of the bedding planes are observed, indicating different periods of deposition and action of waves in the shallow marine or beach environments. The different grain sizes of the quartzites are caused by recrystallization. The latter may give rise to localised development of coarser and/or oriented material. The various joint sets own their development to the metamorphism which the rocks have undergone. Quartzite is considered a strong rock type. Its compressive strength could reach values higher than 280MPa. Quartzites do not weather easily but they are prone to weathering along the joints and bedding planes.

Photographs showing flow conditions during a flood event confirm what is obvious from a field visit of the area downstream of the chute; the larger part of discharging water follows two naturally eroded channels on the right, and the smaller part of the flow is being directed towards the main river through a third channel on the left side of the area. The erosion has exposed large areas of bedding planes with ripple marks on them. The measurements in the field have been conducted in three positions, according to the three different channels that the overflowing water follows. The area immediately downstream of the concrete channel is almost flat but the slope changes farther downstream to a gradient of 30-40^o. The area is uneven because of previous movement of rock blocks. Steps, ridges and exposed joint surfaces occur (Plate 17). The joint walls are almost smooth and planar and the joint intersections are sharp. The conditions change farther away from the end of the channel and close to the point where the gradient changes, the joint walls become more rough and undulating.

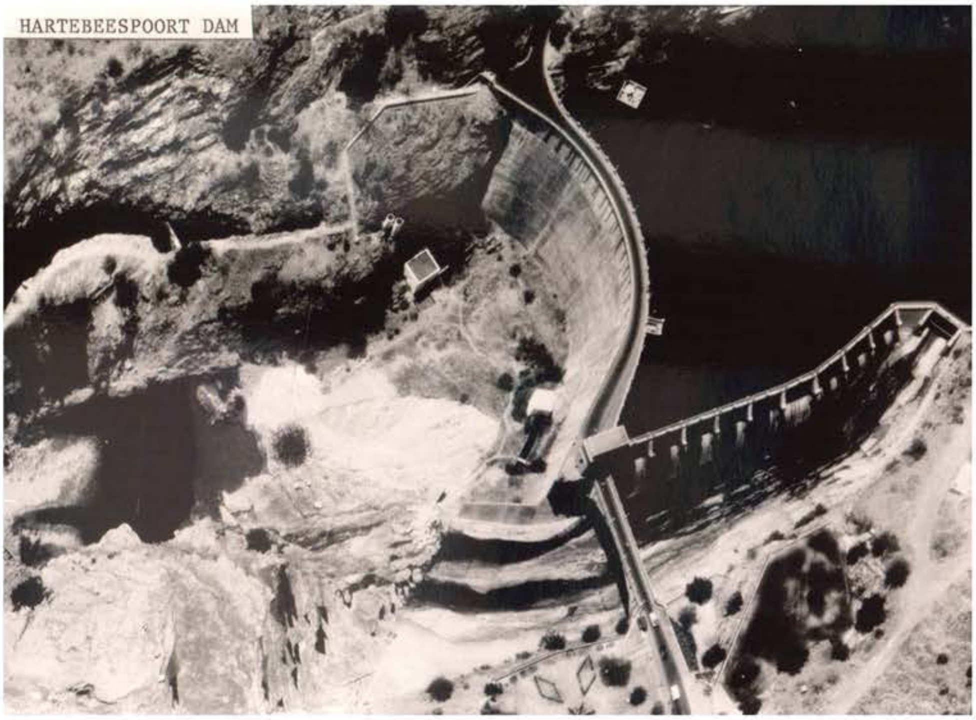


Plate 16 Hartebeespoort Dam (Dept of Water Affairs).

Plate 17 Steps and ridges in the spillway channel due to jointing

The first position, near the first channel on the right side, is the most eroded area. The RQD is less than 5 per cent and the number of joint sets is six. Joints are continuous, with spacing mostly about 80mm - seldom 400mm - and separation between the joint walls of 1-5mm. Where gouge exists, it is sandy material, sometimes with plant roots. Thin iron oxide stains the surfaces and indicates water circulation through the joint planes. Near to the second channel where the grain size of the rock mass becomes larger because of recrystallization, there are four major joint sets. The joints are closely spaced and tight, except one joint set, where separation of up to 200mm was observed. The filling material, where it has not been washed out, consists of sand. In the third position, close to the side, there are three main joint sets with many tight, curved and discontinuous random joints. The joints are mainly tight and the spacing varies between 3mm and 250mm (Fig.6.10). The hardness measured with the Schmidt Hammer Test varies from position to position, but average value of 40 corresponds with

- an U.C.S., σ_c(MPa), of 50-180MPa, and
- a Modulus of Elasticity, E_t(GPa), of 57-73GPa.

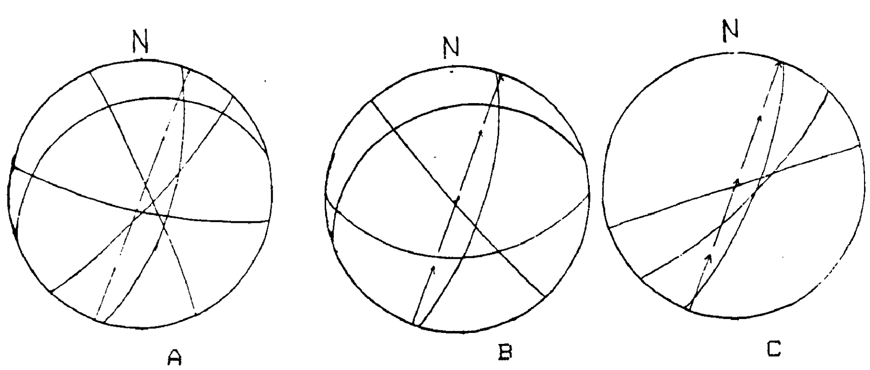


Fig.6.10 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).

A-Projection of discontinuities and water flow along the first channel.

B-Projection of discontinuities and water flow along the second channel, before the gradient changes.

C-Projection of discontinuities and water flow along the third channel, near the flank of the spillway.

Since its completion, the dam must has overflowed many times. The outflow data, which is available, is for the year 1978 and showed that outflow started on the 25th of January and lasted for about 4 days with a peak of 825m^3/s on the 28th of January. The total volume of the flood event was 103x10^6m^3 (Fig.6.11).

The maximum depth of the erosion has been estimated at about 1,50m. Remnants of concrete lining on the rock immediately downstream of the end of the concrete channel, as well as old constructions have been observed. Rock blocks have been moved into new positions by the water and others are to be moved during forthcoming floods. The rock mass is well jointed and the close spacing results in small blocks that are easily transported. The contact between the joint walls is generally tight but should it become wider the erosion process will be favoured. Ripple marks reduce strength, but increase roughness which resists movement or sliding. The channel along which water flows, limits the areas affected by erosion.

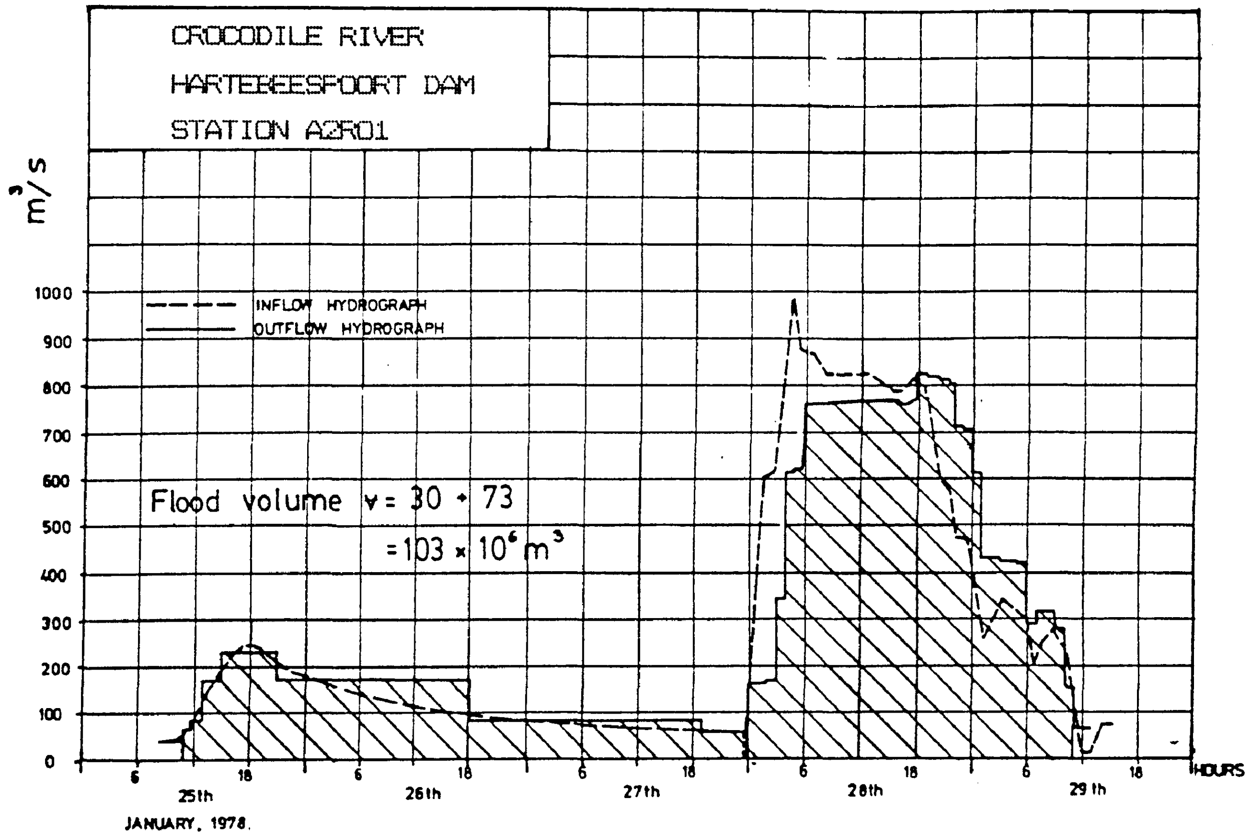


Fig.6.11 Hydrograph of the flood in Hartebeespoort Dam in January 1978 (Dept of Water Affairs).

6.3 Free-falling jets.

6.3.1 Wagendrift dam.

The dam was completed in 1963. It is situated on the Bushmans River, near the town of Estcourt, in the Natal. The purpose for its construction is irrigation of the area.

It is a multiple arch butress structure with height above lowest foundation of 40m and crest length of 281m (Plate 18). The gross capacity of the reservoir is 59 900x$10^3$$m^3$ whereas the reservoir area is 5 054x$10^3$$m^2$. The orientation of the dam structure is about N170^o so that water falls in the direction of N080^o. The two central arches form the uncontrolled free-fall spillway. The crest is 35m high above the lowest foundation and the maximum discharge capacity is 1 300m^3/s.

Plate 18 Wagendrift Dam (Dept of Water Affairs).

The spillway discharges the overflow water on rocks of the Estcourt Formation of the Beaufort Group of the Karoo Sequence which had been intruded by dolerites. No major faults occur within this area. The Estcourt Formation consists of westwards dipping, laminated carbonaceous shale, plus subordinate cross bedded, often coarse-grained, occasionally pebbly sandstone and a few thin coal seams. The shale downstream of Wagendrift spillway is not more than 1m thick and looks more like siltstone because it feels gritty and silt occupies more than 1/3 of the mass. The density as determined from the literature should vary between 2 500kg/m^3 and 2 600kg/m^3 and compressive strength between 40MPa and 160MPa. The mean value of hardness as measured by the Schmidt Hammer Test is 44,7 and its correlation with unconfined compressive strength and modulus of elasticity gives the following values:

- unconfined compressive strength, σ_c(MPa), betweeen 60-200MPa, and
- modulus of elasticity, E_t(GPa), between 53-78GPa.

Intrusions of the dolerite affect shales (like every sedimentary rock), causing highly jointed zones and developing deep weathering along these contacts. The intrusions of dolerite are usually horizontal but others are gently and evenly inclined in discordant relationship to the host sediments. Formed during the terminal phase of basalt solidification, fresh dolerite is a hard rock, suitable for engineering purposes. Chemical decomposition in the area causes changes of primary minerals to secondary minerals and particularly to montmorillonite. Downstream of the spillway, dolerite on the right flank is slightly weathered and slightly fractured, but on the left downstream area, where the bedrock consists only of dolerite, the rock mass is more weathered and very well jointed. The compressive strength of dolerites varies from 100MPa to 200MPa and its dry unit is about 2 900kg/m^3. Measurements for hardness, using the Schmidt Hammer Test, show the average value of 58,5 and the correlation with the compressive strength and the modulus of elasticity is as follows:

- unconfined compressive strength, σ_c(MPa), between 130-330MPa, and
- modulus of elasticity, E_t(GPa), between 94-116GPa.

In the right spillway area, dolerite underlies the shale (Plate 19). The sedimentary rock, about 80cm thick, is very well jointed with open joints, spaced between 100mm and 500mm and smooth to slightly rough surfaces. There is a soft silty filling material of weathered shale. On the doleritic mass, four joint sets occur. They are all tight (with one exception of 50mm), widely spaced, with rough walls and hard, brittle filling material. In the left area, only dolerite exists, in various degrees of weathering, but definitely more weathered than the dolerite of the right area (Plate 20). The RQD is between 30 per cent and 35 per cent. The joints have a spacing of 100-250mm and separation of 2-8mm. The joint walls are continuous but rough and undulating (Fig.6.12).

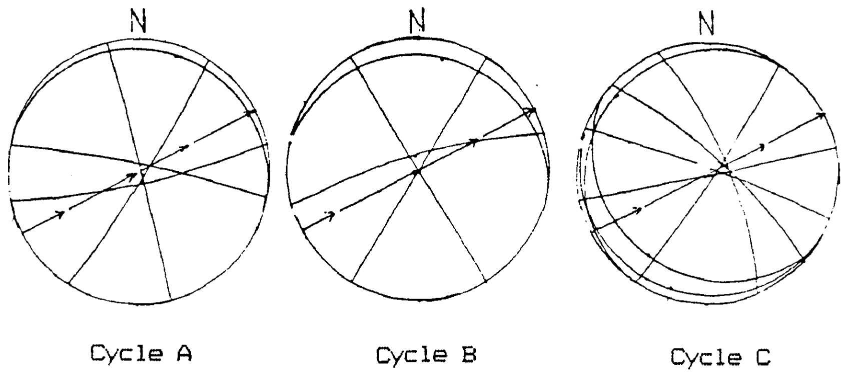


Fig.6.12 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).
A-Projection of discontinuities of the shale and water flow in the right spillway area.
B-Projection of discontinuities of the dolerite and water flow in the right spillway area.
C-Projection of discontinuities of the dolerite and water flow direction in the left spillway area.

Since its construction, water has overflown many times, almost every year and of significant volume and discharge (Fig.6.13). In 1975 a large flood occurred. The spillway had been overflowing for almost 9 months, from the 21st of November 1975 till the 1st of August 1976. The maximum daily average discharge was 152,35m^3/s. Scouring of the sedimentary rocks resulted in a steady deterioration of the rock mass coherence after construction. This situation was only recognized after 15 years of operation. Regular surveys of the apron were then initiated in order to monitor and possibly determine the rate of erosion. The 1978 survey indicated scour appeared at 2-4m from the dam's foundation. During the preceding 15 years, about 1m of rock had

Plate 19 Jointed shale overlying dolerite rock mass downstream of the right spillway arch.

Plate 20 Jointed dolerite downstream of the left downstream area (Van Schalkwyk).

been plucked from both aprons. In 1987, the maximum flood in the history of the dam ($359{,}20m^3/s$ - daily average discharge) happened, when water passing over the wall reached two metres height. The flood lasted from the 27th of September till the 1st of October (140 hours), had a peak outflow of $687{,}1m^3/s$ on the 29th of September. The outflow volume was $88{,}5x10^6m^3$ (Fig.6.14). This event scoured the bedrock in the right area to within 2 metres of the wall, removing about $150m^3$ of rock.

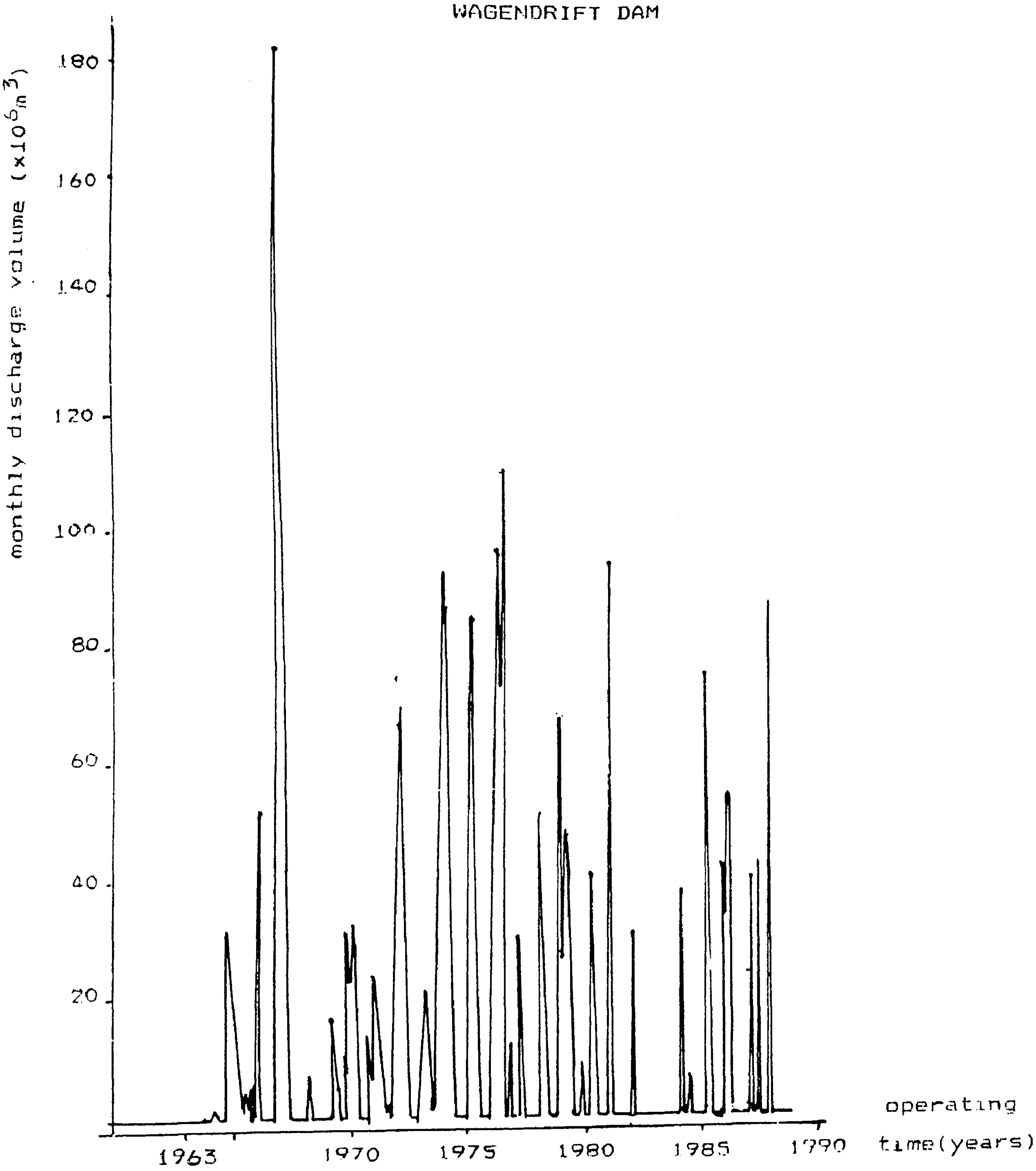


Fig.6.13 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

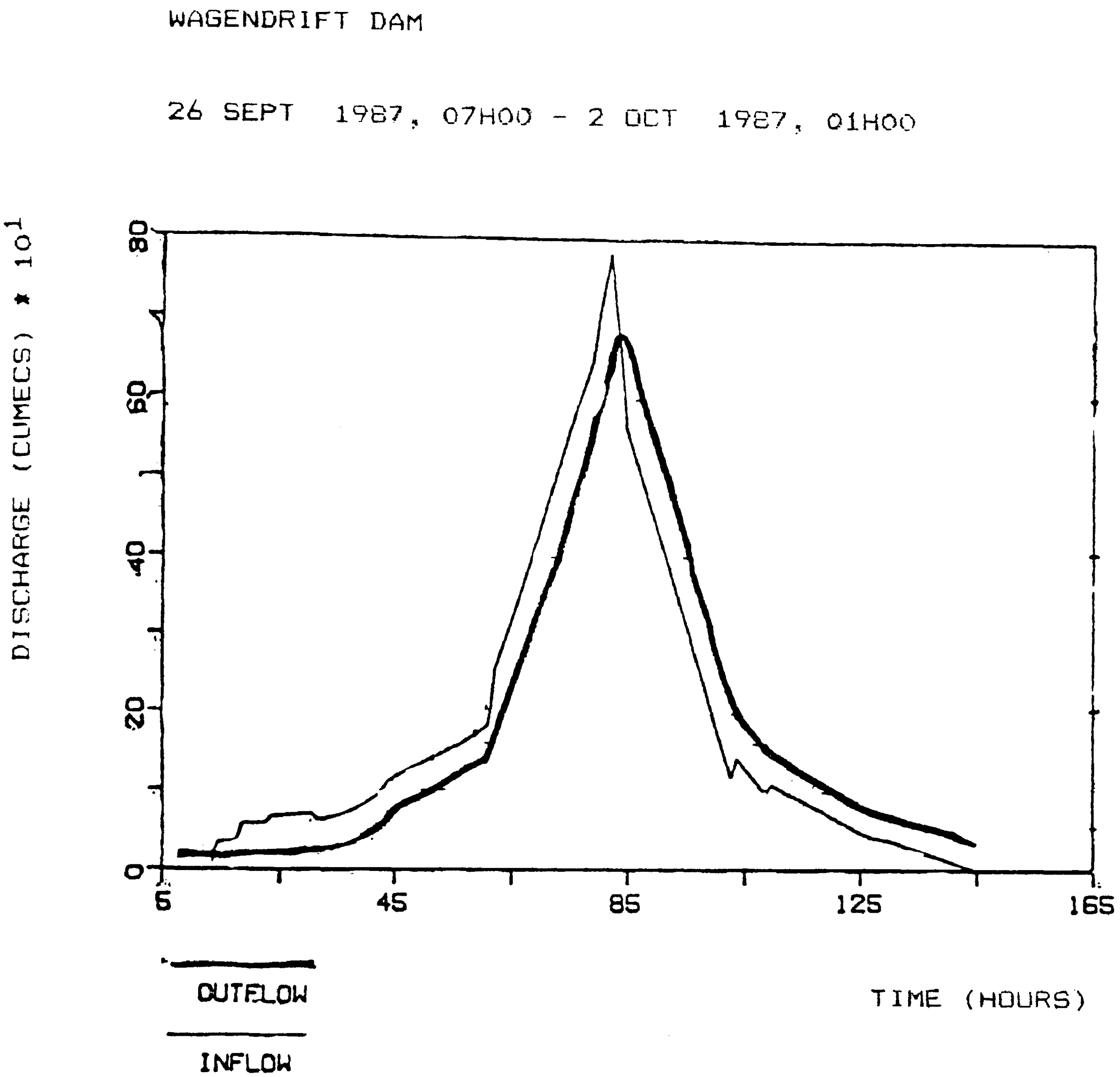


Fig.6.14 Hydrograph of the flood in 1987 (Dept of Water Affairs).

As temporary protection measure, gunite was applied to the eroded area below the right hand spillway. This performed well during subsequent floods. More permanent measures in the form of thick reinforced concrete slab with rock bolts, on both sides of the spillway, were constructed during 1989.

The discharge during the maximum event reached the highest value of 687,1m^3/s whereas the maximum discharge capacity of the spillway is 1 300m^3/s. The non-uniform lithology of the bedrock caused backward scour of the most erodible rock type, the sedimentary rock on which erosion had started, and that endangered the foundations of the wall. The dip direction of the joints that dominate on the shale is with the water flow direction. Four joint sets strike obliquely to the water direction and one set is perpendicular and dips at 90°. The directions of the joint sets of the sedimentary rock on the right apron are similar to the joint directions of the underlying dolerites. However, the separation of the joints is tighter, the spacing is greater and the filling material between the rough joint walls is harder. Sliding of the shale blocks took place along subhorizontal planes and failure along lines of intersection also occurred. In the left area, dolerite is moderately jointed, some of the joints are discontinuous and most of them are undulating. More of the joints on the dolerite of this apron are open than on the dolerite of the right area. Erosion is estimated at about 1,50m on the right apron and mainly scoured is the sedimentary rock, while on the left side scour is about 1m deep.

6.3.2 Craigie Burn Dam.

The dam was completed in 1963. It is located on the Mnyanvubu River, near Greytown, in Natal. The purpose of its construction is irrigation of the surrounding area.

It is an arch dam with embankments on both sides (Plate 21). The concrete crest is 200,5m long and the embankments are 259m long. The height of the arch wall above lowest foundation is 38m. The gross capacity of the reservoir is 25 100x$10^3$$m^3$ and the reservoir area is 2 206x$10^3$$m^2$. The uncontrolled spillway is situated on the arch wall, it is 35,3m high above lowest foundation level and its crest is 121,9m long. The maximum discharge capacity of the spillway is 340m^3/s.

The river at the site cuts through a prominent N-W trending dolerite intrusion of post-Karoo age. The general geology of the area consists of topsoil, weathered dolerite and fresh dolerite. The topsoil is 0,3-0,6m deep while the weathered dolerite occurs to depths 1,5-2,1m and in some cases to 7,5m (Van Wyk, 1960). On the right flank, the doleritic intrusion disappears under horizontally disposed Upper Ecca Shales. Owing to their mode

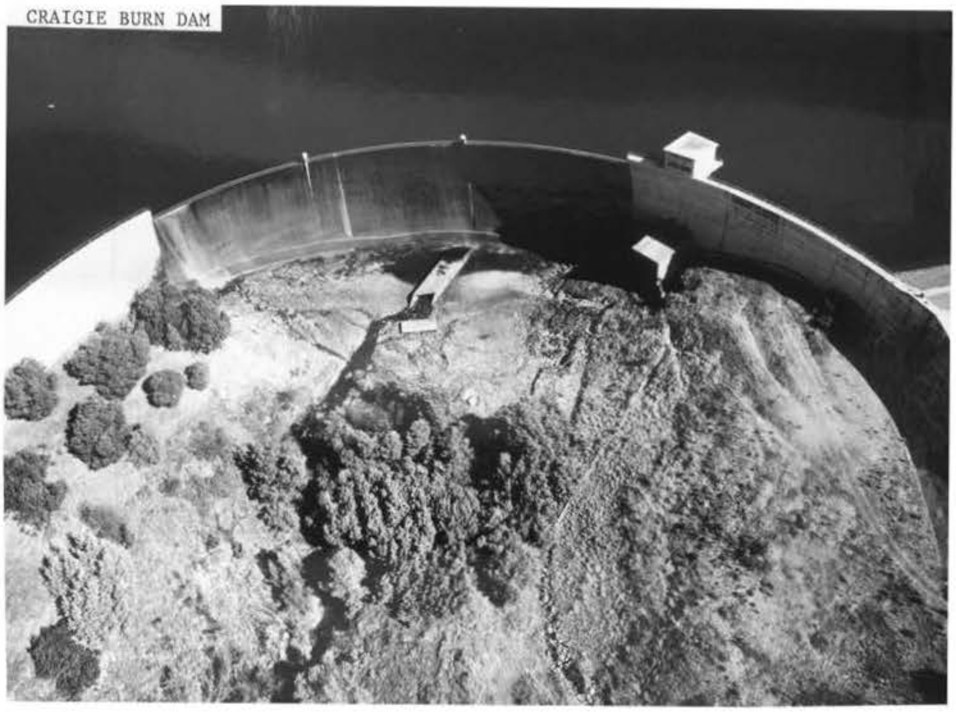


Plate 21 Craigie Burn Dam (Dept of Water Affairs).

Since the completion of the dam, water has been overflowing for several times, almost once a year until 1979 and then in 1985, 1986 and 1987 (Fig.6.16). The maximum flood event occurred in September 1987, when the maximum peak outflow was of $366,5m^3/s$. The outflow lasted 110 hours and the total volume was $28,4x10^6m^3$ (Fig.6.17).

Erosion downstream of the wall is of minor extent - maximum depth of 0,80m - and for the time being, it does not endanger the structure (Plates 22, 23). Water enters the area from different directions, the average is N090°, and falls into the central pool. Joints intersect each other, forming lines of intersection, but sliding mostly occurs along the subhorizontal joints. The water flows obliquely to the strike of most of the joint sets. There is one set, perpendicular to the water flow direction and dipping vertically against it and almost perpendicular to the strike of the subhorizontal joint set, forming unstable conditions,in addition to the vertical joints, that strike closely to the direction of the flow and permit the entrance of the water. Many of the dolerite blocks formed by the joints could be moved during next outflows, and that will lead to increased erosion.

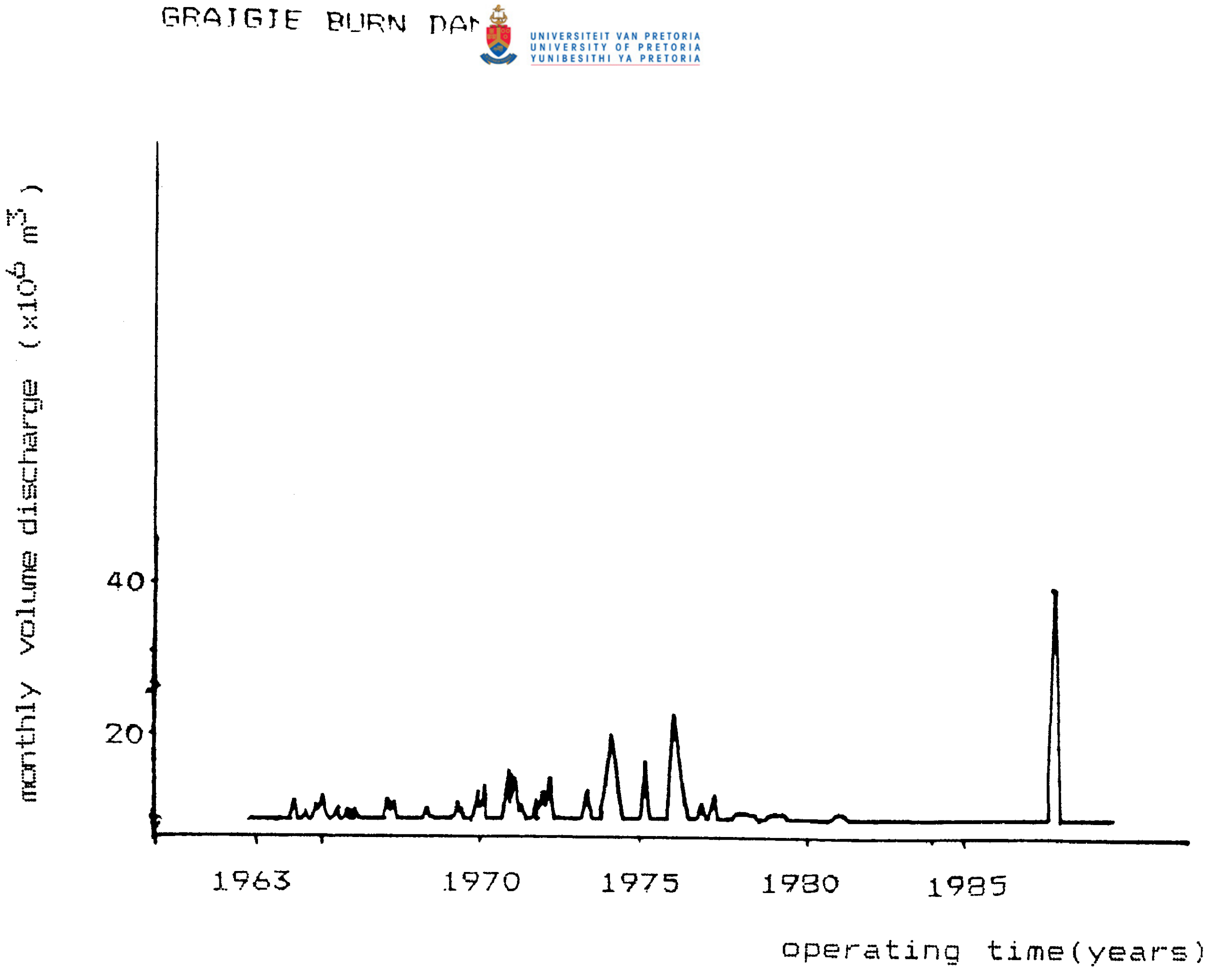


Fig.6.16 Frequency of water discharge according to hydrological information from the Dept of Water Affairs.

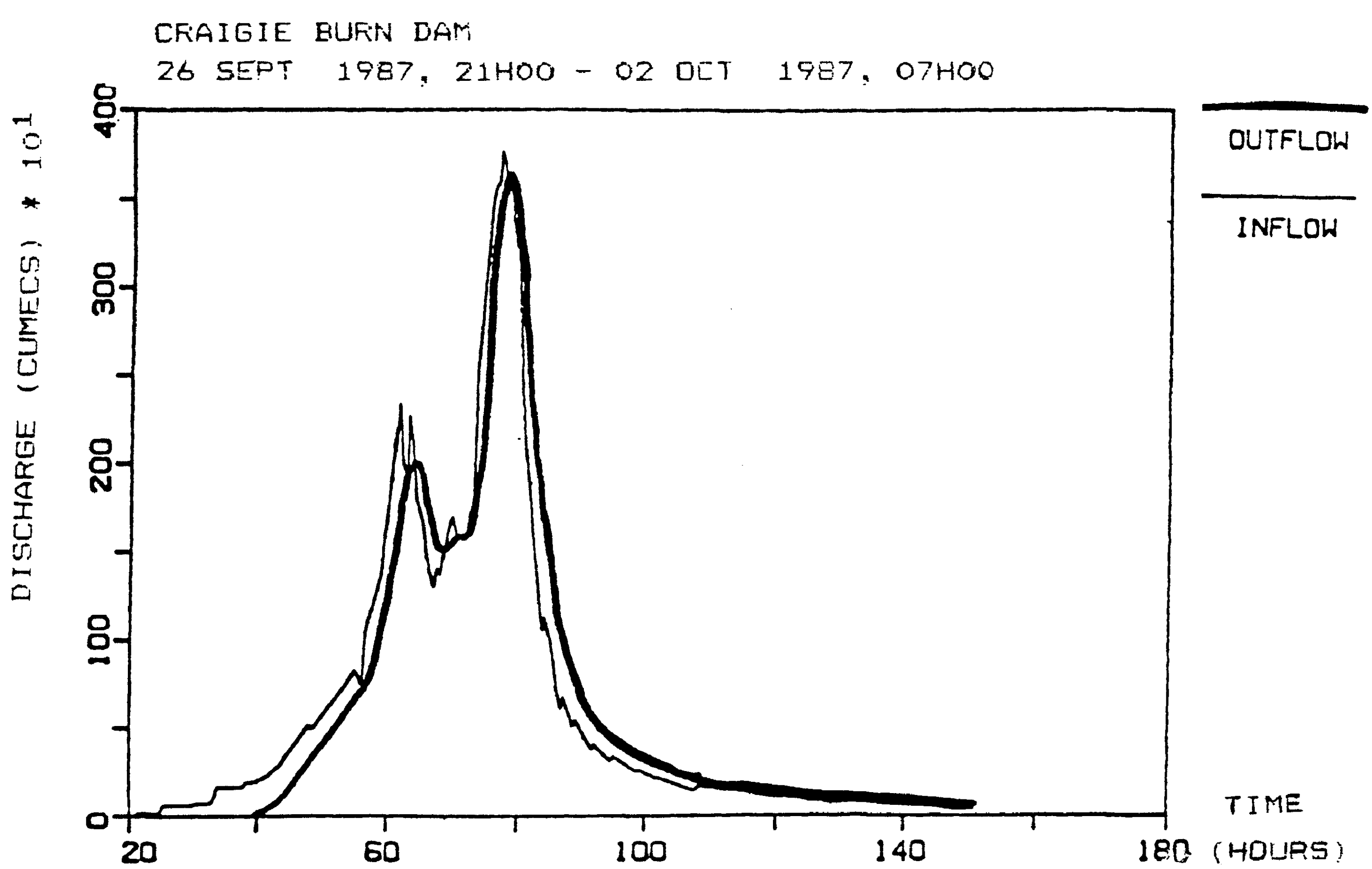


Fig.6.17 Hydrograph of the flood in 1987 (Dept of Water Affairs).

Plate 22 Removal of the topsoil downstream of the arch in the central and right flank areas (Van Schalkwyk).

Plate 23 Depth of erosion from the original ground depth, on the left downstream area of the arch.

6.3.3 Roodeplaat Dam.

The dam was completed in 1959. It is situated on the Pienaars River, near Pretoria. The purpose of its construction is irrigation.

The dam is an arch structure, with concrete gravity sections along the tops of both flanks (Plate 24). It has a 351m long crest which is 54,1m high above lowest foundation. The gross capacity of the reservoir is 42 100x$10^3$$m^3$, whereas the reservoir area is 4 035x$10^3$$m^2$. The free fall spillway is located on the main dam wall, 50,4m high above the lowest foundation and 143m long. The maximum spillway capacity is 970m^3/s. There is no concrete apron provided. The spillway had to be shifted to the right of the dam wall to avoid a closely jointed zone of felsite, which is highly weathered to great depth below the surface on the left flank. A pothole which had underlied the central block of the wall, contained about 1 000m^3 of alluvium and covered a surface of 150 m^2, was filled with concrete and it is now overgrown by grass (Schulze-Hulbe et al., 1988).

Plate 24 Roodeplaat Dam (Dept of Water Affairs).

The site of the dam is near the northern end of the fairly narrow valley, where the Pienaars River has cut through a succession of sedimentary, volcanic and intrusive rocks. The wall is underlain by slightly weathered to unweathered, medium jointed felsite, belonging to the Pienaars River Complex. The centre of the complex, 5km north of the dam site, is an old volcanic crater consisting of an outer zone of trachyte and felsite and a central zone composed of volcanic tuff and tuffaceous shale. Circular diabase dykes surround most of the crater. Where the dam is located, felsite has been intruded by a syenite porphyry dyke, although the two different types can not easily be distinguished and they are of similar engineering properties (Schulze-Hulbe et al., 1988). The felsites are dense, compact rocks of a brownish to purplish red colour. They predominantly consist of quartz and feldspar microcrystalline matrix in which there are moderately large ctystals of red feldspar (Kent, 1952). The maximum unit weight should be around 2 700kg/m^3. The rock is hard; tested by the Schmidt Hammer Test gives a mean value of 58,25. Correlating hardness with unconfined compressive strength and modulus of elasticity, the following values are obtained:
- U.C.S., σ_c(MPa), between 130-310MPa, and
- modulus of elasticity, E_t(GPa), between 83-110GPa.

The rocks are heavily jointed by continuous, as well as many discontinuous, undulating, rough, usually some milimetres open and closely spaced joints (Fig.6.18). The rock mass is intersected by 20-30 joints/m^3. Stress relief joints developed along the downstream valley, on both flanks, strike perpendicular to the dam wall axis and dip towards the river along both flanks. This results in a dome-shaped appearance of the exposed rock surfaces (Plate 25). The felsites display in places a flow banding which dips downstream. The rock mass is generally slightly weathered, but intensively weathered along the discontinuity planes. Weathering takes place around a rock block to form spheroidal corestones (Plate 26). The joint planes are in nearly all cases stained with a brownish or reddish brown colour which proves that water has seeped through them. Along a discontinuity plane, striking perpendicular to the dam wall on the left side of the plunge pool, a zone of moderately weathered rock of 500mm thickness occurs. Pyrite sheets, veinlets of sulphides or fluorite, fluospar and quartz veins, even zeolite veins are often present (Kent, 1953).

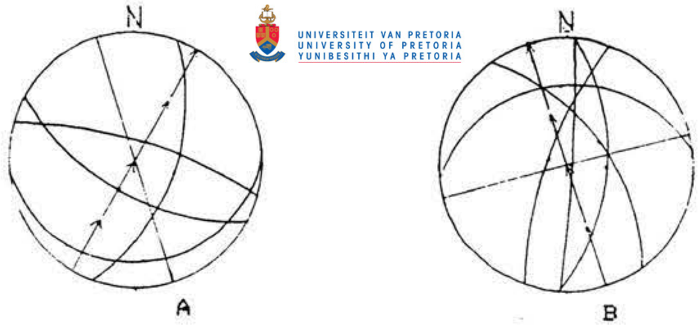


Fig.6.18 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).
A-Projection of discontinuities of the rock mass and water flow on the left side.
B-Projection of discontinuities of the rock mass and water flow on the right side.

Daily average records, for the period October 1970 - September 1989, show that water overflowed the spillway almost every year, except during 1985 and 1986 (Fig.6.19). The outflow was usually small; only in 1975 and 1978, the daily average value exceeded $100m^3/s$. The maximum outflow of $971m^3/s$ which is the maximum spillway capacity occurred on 27 January 1978. The water had been overflowing since 31 November 1977 till 21 November 1978, for almost one year, discharging a total volume of $125,5x10^6m^3$.

High flood frequency, peak outflow equal to the maximum discharge capacity and high duration of the maximum flood event are the hydraulic parameters which assist in the erosion process downstream of the spillway. Downstream of the concrete filled pothole, a pool of 1,50-2,3m depth has been eroded and many loose rock blocks have been washed off the flanks, since the first overflow event occurred.

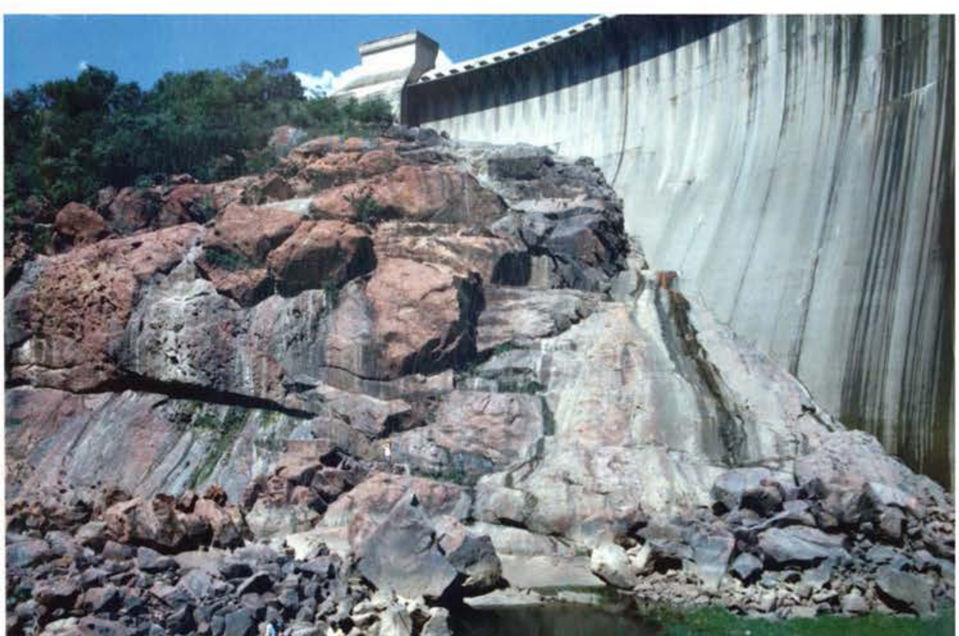


Plate 25 Rock blocks of various sizes formed by many joint sets - Stress relief joints dipping at right angles to water flow direction (Van Schalkwyk).

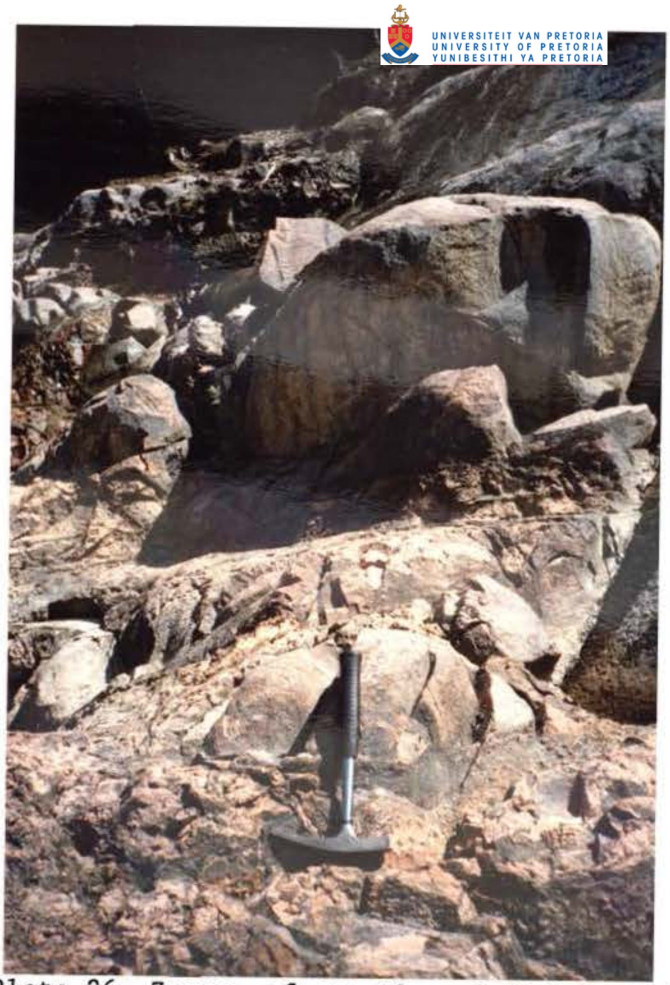


Plate 26 Zones of weathered felsite including unweathered jointed rock on the left side downstream of the arch.

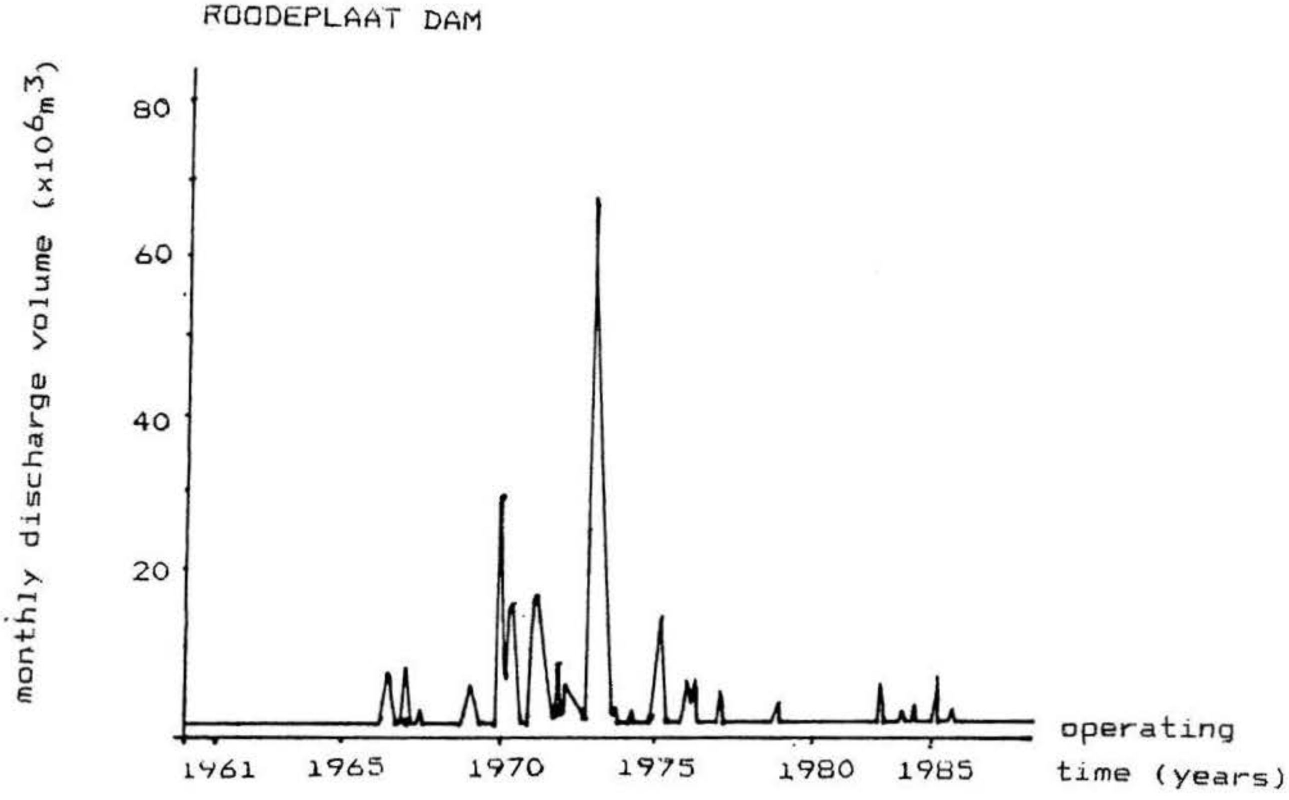


Fig.6.19 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

6.4 Spillways with flip buckets.

6.4.1 Goedertrouw Dam.

The dam was completed in 1982 and is situated on the Mhaltuze River, north of Eshowe, in Natal Province. The dam has been constructed for irrigation and water supply of the surrounding area.

It is an earth-fill structure, 88m high above the lowest foundation and with crest length of 660m. The gross capacity of the reservoir is 321 000x10^3m^3 and the reservoir area is 12 000x10^3m^2. It has a spillway structure located 230m from the right flank of the main dam wall. Because of the pressure of erodible material in the downstream area, the spillway incorporates a flip bucket with an exit of 33^o with the horizontal on the downstream surface, a short apron and a cut-off wall to dissipate the energy of the falling water and to avoid the effect of headward erosion undercutting the structure (Plate 27). The spillway is 160m long and 10,5m high above the initial bedrock level. The maximum spillway capacity is 7 000m^3/s.

The spillway discharges water on weathered tillite of the Dwyka Formation of the Karoo Sequence. When unweathered, the tillite is greenish- to bluish-grey rock, consisting of pebbles and boulders of various rocks set in a matriz of fine grained argillaceous material. Boulders of varying rock types caught up in the original glacial moraine forming the tillite are predominantly granitic in origin (Price et al., 1978). The physical properties of the rock primarily depend on those of the clayey matrix. This material is hard and fairly strong in-situ but tends to become soft when it is wet and it is susceptable to air slaking and disintegration when it is exposed to the atmosphere (Van Schalkwyk, 1969). In northern Natal, tillite rests on a complex pre-Karoo topography of high relief, with deep valleys and elongate ridges. About 2km upstream of the dam-site, the Table Mountain Series on which the Dwyka Tillite has been unconformably deposited, outcrops. Below the dam-site, the country flattens out considerably, in passing from tillite into higher members of the Karoo Sequence.

The results of seismic velocity and unconfined compressive strength concerning each different degree of weathering, and results of other tests on Dwyka tillite at Goedertrouw and Oppermansdrift dams are listed in Table 6.1. Point Load Strength Tests on tillite showed that there is a large strength difference between fresh or slightly weathered rock and moderately to highly weathered rock. The moderately weathered tillite which underlies the spillway section has an unixial strength ranging from 40 to 50MPa (Maeyens, 1975).

	Seismic velocity (m/s)	Un.Compressive strength (MPa)	Porosity (%)	Density (kgr/m^3)
W1	4.960-5.512	122-298	range	range
W2	3.858-4.960	80-130	0,0021-0,0076	2.508-2.690
W3	1.654-3.858	10- 40		
W4	551-1.654	5- 22		
W5	<551	<1		

Table 6.1 Engineering properties of tillite at Goedertrouw Dam and Oppermansdrift Dam (Brink, 1983).

As weathering has started from the joints, it appears worst in highly jointed areas. Stained joints persist to great depth indicating a deep circulation of groundwater. The form which weathering takes place, seems to be in relation with the joint spacing, so that in case of widely spaced joints, spheroidal weathering occurs similar to that encountered in regularly jointed igneous rocks. When weathering has not progressed very far, very large in-situ boulders of fresh tillite are left surrounded by highly weathered and closely jointed tillite zones (George, 1977). Definitions of the different degrees of weathering of tillite are given in Table 6.3. A high degree of jointing (RQD being about 27 per cent), the rock mass into roughly cubical blocks which, upon weathering, changes into rounded corestones. There are three main joint sets and many other sets intersect them at random. Primarily structural joints are the major continuous near-vertical and sub-horizontal joints which indicate the previous stresses of geological origin to which the tillite was subjected. Weathering proceeded along some incipient lines of weakness and progressively reduced the joint spacing, causing irregular secondary jointing. Two of the main joint sets are dipping vertical and striking at right angles whereas the third set is almost horizontal. The joints are continuous with smooth and planar surfaces which become undulating along the subhorizontal joints and the moderate weathered surfaces. The spacing is between 60mm and 200mm for the vertical joints and 600mm - rarely becomes 100mm - for the subhorizontal set. The joint walls are open up to 30mm. Filling material, if it is not washed out, may be sandy, soft weathered rock in some joints (Fig.6.20). The Schmidt Hammer Test gave a mean value of 55,6, which corresponds with the unconfined compressive strength and modulus of elasticity as follows:

- U.C.S., σ_c(MPa), of 100-270MPa, and
- modulus of Elasticity, E_t(GPa), of 64-100GPa.

GRADE OF WEATHERING	COLOUR	DESCRIPTION OF THE ROCK
W1. Unweathered	blue	completely fresh material; very hard rock
W2. Slightly weathered	brown	similar to blue material, but stained brown on discontinuity planes; hard rock
W3. Medium weathered	brown	the material is stained completely brown; soft to hard rock
W4. Highly weathered	yellow, pink or white	very soft to soft rock, retaining original structure
W5. Completely weathered	dark brown to black	gravelly residual soil with little or no inherited fabric; may be soft, firm or stiff, usually a loose gravel

Table 6.2 Weathering classes of tillite (Brink, 1983).

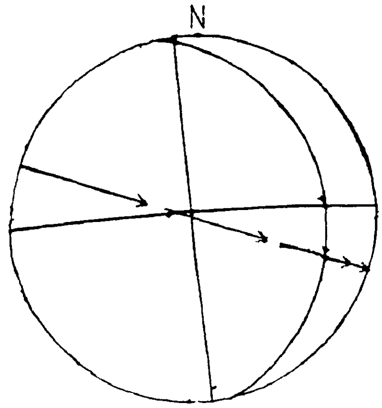


Fig.6.20 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).

From the monthly discharge records, it can be seen that the dam overflowed only once (Fig.6.21), in September 1987. According to the hydrograph of this flood event, the outflow started on 28 September. It lasted 95,5 hours, and had a peak outflow of 589,5m^3/s, 5,5 hours after it started. During the flood, 73,9x10^6m^3 of water was discharged (Fig.6.22).

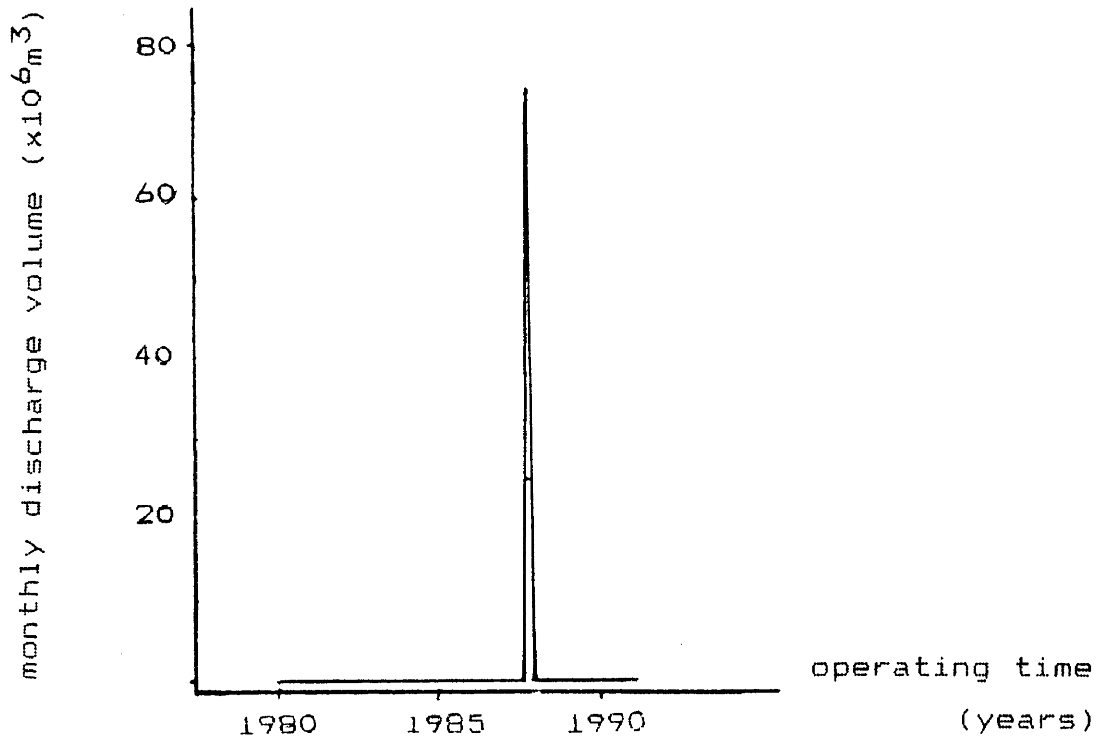


Fig.6.21 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

26 SEPT 1987, 06H00 - 01 OCT 1987, 23H00

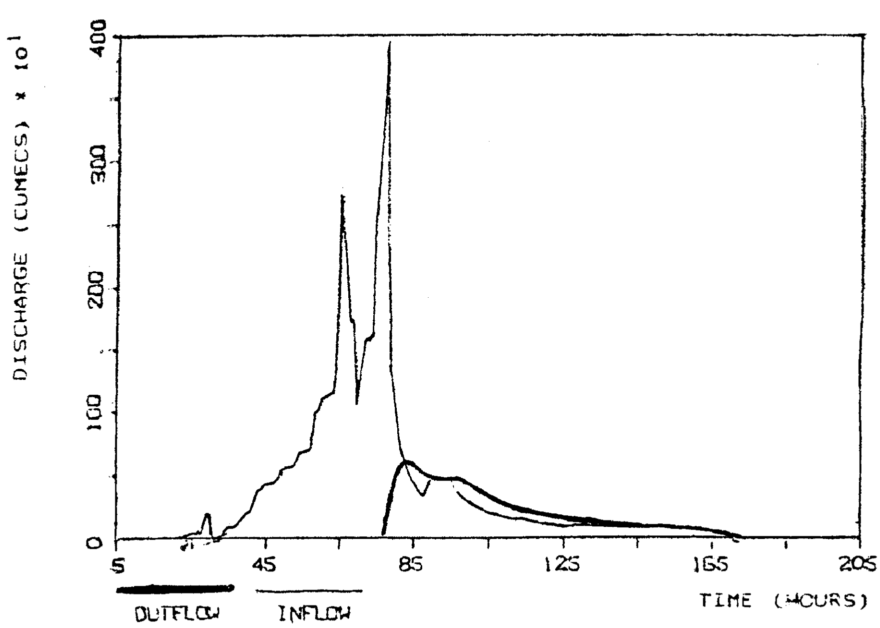


Fig.6.22 Hydrograph of the flood in 1987 (Dept of Water Affairs).

The flood event was less than the 12 per cent of the maximum spillway capacity. Scour of the downstream bedrock to a depth of approximately 3m, it has been estimated (Plate 28). Removal of many blocks, some of them of large size and change a smooth, inclined area of brown sandy topsoil (George et al., 1981) to an uneven, rough surface with loose boulders of various sizes and closely spaced jointing. The close spacing and the open joint walls reduce the cohesiveness of the rock mass and increased the probability of movement. The intense jointing is a negative factor and the direction of the joints according to the direction of the water flow direction is not favourable. Two of the joints are striking perpendicular to the water flow direction, one almost at 90 degrees dip angle and the other at about 20°, dipping with the water flow direction. In addition, the third vertical joint set is nearly parallel to the water flow, so that cubical rock blocks unfavourable for stability can be formed. The downstream slope of the bedrock is approximately 15 degrees, and sliding along the subhorizontal joint set is possible.

Plate 27 Goedertrou Dam - Side-channel spillway with flip bucket, apron and cut-off wall (Van Schalkwyk).

Plate 28 Extensive erosion of the jointed tillite downstream of the spillway (Van Schalkwyk).

6.4.2 Vygeboom Dam.

Vygeboom Dam - previously known as Kafferskraal - was completed in 1971. It is situated on the Komati River, near Badplaas in the Eastern Transvaal. The purpose of the construction of the dam is for irrigation and water supply to the surrounding area.

It is an earth-fill structure with a crest 1 220m long and 48m high above lowest foundation. The gross capacity of the reservoir is 78 $800 \times 10^3 m^3$ and the reservoir area is 6 $690 \times 10^3 m^2$. The spillway structure consists of a concrete chute on the left side of the embankment (Plate 29). The chute is 189m long and consists of different sections according to the slope. At its end, a flip bucket is included, at 45^o to the chute direction which forms a 45^o angle with the main dam wall axis. Water leaves the flip bucket at an angle of 40^o from the horizontal. The lip of the flip bucket is 2,7m higher than the downstream bedrock surface, whereas the height of the spillway crest is

42,3m, above the lowest foundation. The width of the chute at its exit is 41,55m. The maximum spillway capacity is 2 617m^3/s.

The bedrock downstream of the spillway consists of granites and granitic gneisses of the Basement Complex. Though usually referred to broadly as "granites", they in fact constitute a complex suite, ranging in mineralogical composition from true granites, through granodiorites to quartz-diorites and tonalites, and even include more basic rocks. Gneissic banding and veins of coarse-grained pegmatite and fine-grained aplite are also common. They are the oldest rocks in South Africa. In the humid regions of the Eastern Transvaal, the granites may be decomposed into residual soils of great depth. However, the degree of weathering of the granites downstream of the spillway of Vygeboom Dam is not of high order. Granites and gneisses are rock types of high strength, but the spacing and the quality of the joints and fractures control the rock mass properties. The rock mass downstream contains five joint sets. The joint walls are slightly rough and undulating (Plate 30). Some joints are discontinuous. The joint spacing is usually more than 300mm and the contacts are tight. The RQD is approximately 50 per cent (Fig.6.23). Tests with the Schmidt Hammer gave a mean value of 56,25 and the correlation with unconfined compressive strength and modulus of elasticity is as follows:
- U.C.S., σ_a(MPa), 110-280MPa, and
- modulus of Elasticity, E_t(GPa), 72-102GPa.

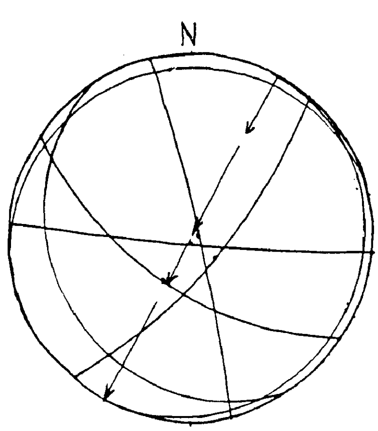


Fig.6.23 Stereographic projection of the discontinuity planes (——) and the water flow direction (— —).

Since October 1971, monthly and daily records of the outflow are available. The peak outflow of 129m^3/s occurred on 8 February

1974, representing about 5 per cent of the spillway capacity. During this discharge, outflow of more than $10m^3/s$ lasted for 110 days and the total discharged volume was $224,551x10^6m^3$ ($54,070x10^6m^3$ in December, $66,243x10^6m^3$ in January, $69,382x10^6m^3$ in February, $34,856x10^6m^3$ in March). Discharge over the spillway or/and through the outlet has occurred every day, throughout its operation years (Fig.6.24).

In spite of the presence of more than three joint sets in the rock mass, they do not induce poor rock quality and the erosion has reached only 0,25m depth. The joints are undulating and not always continuous, so that there are not many removable rock blocks. Also, the joint walls are mostly tightly closed and rough, and thus capable of resisting movement.

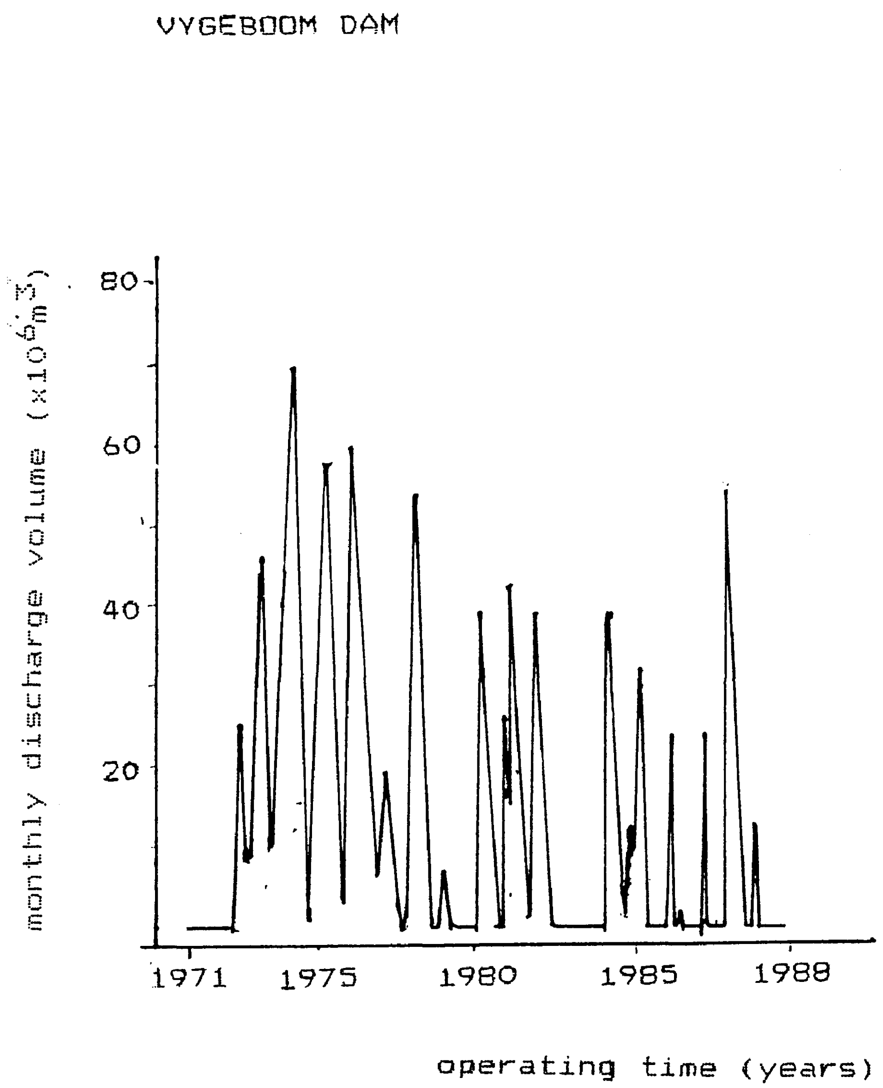


Fig.6.24 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

Plate 29 Vygeboom Dam - Partially lined, side-channel spillway with flip bucket.

Plate 30 Granitic rock mass with undulating, rough joint planes.

6.4.3 Gamkapoort Dam.

Gamkapoort dam was completed in 1969. It is situated on the Gamka River, near the town of Prince Albert, in the Cape Province. The dam serves irrigation purposes.

The structure consists of a gravity wall, of 231m total length and of 43m height above lowest foundation. The gross capacity of the reservoir is 54 300x$10^3$$m^3$ and the reservoir area is 6 250x$10^3$$m^2$. It includes a partially controlled, gated spillway, located on the main dam wall and a double flip-bucket structure on the left side of the wall. The overflowing water falls partially on a concrete apron and partially on rock (Plate 31). The length of the spillway is 108,9m and the drop from the full supply level of the reservoir area to the river bed level is 37,5m. The maximum capacity of the spillway is 4 531m^3/s.

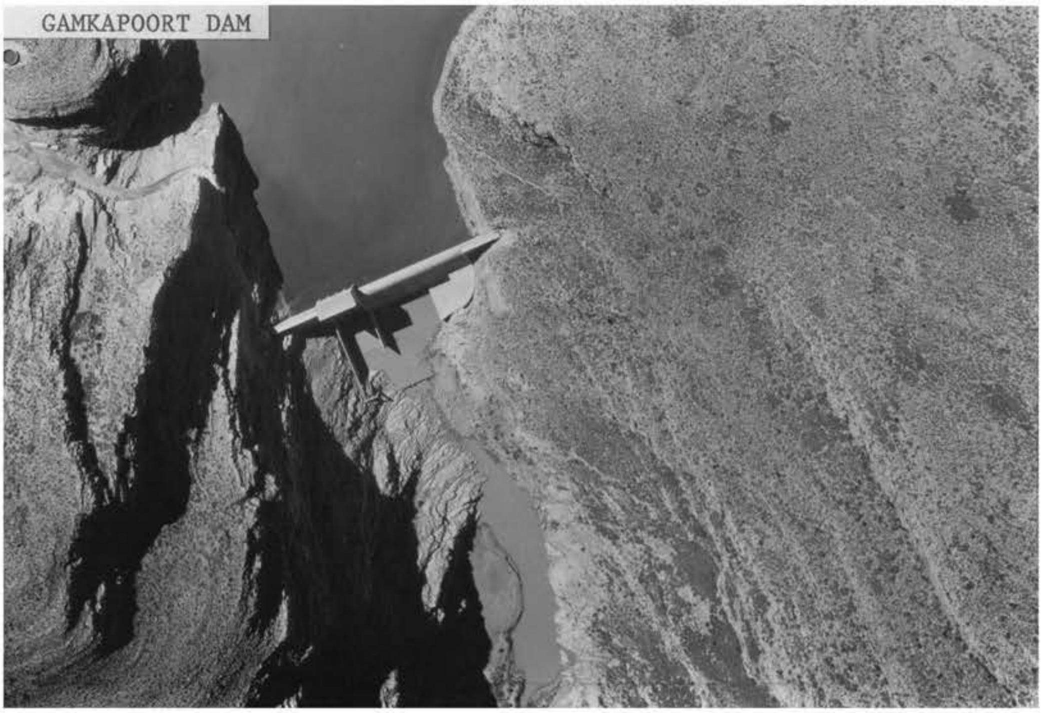


Plate 31 Gamkapoort Dam (Dept of Water Affairs).

The dam is founded on quartzitic sandstones of the Table Mountain Group of the Cape Supergroup, on the northern limb of a large anticline. The average dip of the strata is upstream. Cross bedding is well developed. The rock mass is thickly bedded and has a light brown colour, whereas the weathered surfaces have a reddish brown colour. Thin bands of clayey or micaceous, shaly sandstone occur between the layers of the quartzitic sandstones. When it is fresh, this material is hard, but when it is weathered and wet, it becomes soft and easily eroded (Steenstra, 1961). Crushing and brecciation have occurred along sub-horizontal fracture planes that may follow the bedding for some distance. The fracture planes originated during regional folding of the strata. Brecciation of the rocks took place due to differential movement between the thickly bedded quartzitic layers and softer, intensely, clayey sandstone bands. They constitute structurally weak and probably permeable zones.

Compressive strength of the fresh, hard quartzitic sandstones is generally very high, - 250MPa - and compares very well with that of Witwatersrand Quartzites. Tensile and shear strengths are governed by the discontinuities and the nature and frequency of interbedded soft material, breccia or phyllite layers. Measurements of hardness, using the Schmidt Hammer Test, exhibit an average of 58,5, which correlates with unconfined compressive strength and modulus of elasticity, as follows:

- U.C.S., σ_c(MPa), between 110-285MPa, and
- modulus of Elasticity, E_t(GPa), between 76-104GPa.

The dry density is about 2 600kg/m^3 and the porosity is between 1 per cent and 4per cent.

Intense folding of the strata, in the general area, has led to a variety of joint sets. Where the joints are closely spaced and intersect thinly bedded sandstone or intensely cross-bedded strata, the rocks are relatively weak and break up into small slabs. Boreholes drilled before the construction showed large water losses due to interconnected, open joints that persist over considerable distances. The rocks are only slightly weathered near the surface but due to numerous open joints and bedding planes, narrow or partly weathered seams may extend downwards to various depths within perfectly fresh rocks. The RQD ranges from 0 per cent, close to the lowest flip bucket wall, to 20 per cent, farther to the left. The spacing of the joints is less than 300mm and becomes about 50mm close to the concrete wall, while the wall separation is between 1mm and 5mm. The brecciated material between some quartzitic beds is about 60mm thick and appears approximately every 250mm (Plate 32). The surface of the joints is slightly rough, with the exception of the weathered subhorizontal joints where it becomes smooth (Fig.6.25).

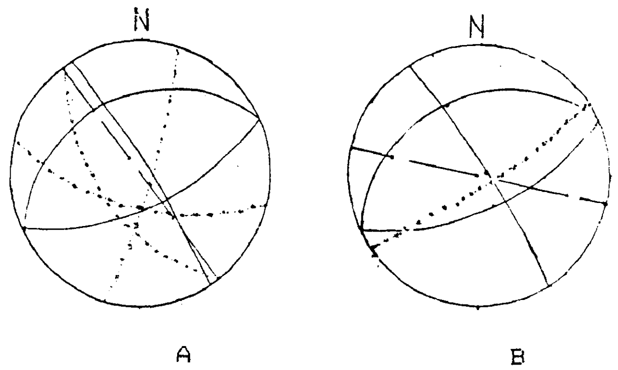


Fig.6.25 Stereographic projection of the major (——) and the secondary (...) discontinuity planes, and the water flow direction (— —).
A-Projection of discontinuities and water flow downstream of the bottom flip bucket.
B-Projection of discontinuities and water flow further to the left of the A position.

From data available for the years 1970-1989 (excluding the period 1979-1982), it can be seen that the dam overflowed six times (Fig.6.26). Two major floods occurred in February, 1974 and in February, 1976. The maximum event in the dam's history, is the 1976 flood. The peak outflow, on 5 February 1976, reached $499m^3/s$. The flood lasted six days, and the total discharged volume was $57,5x10^6m^3$.

Erosion on the left side, downstream the flip bucket, is estimated at about 2,00m depth (Plate 33). The rock mass there is very well jointed with three major sets and many random joints. The loss of strength of the rock mass is caused from weathering and the softer material between the bedding layers. The joint orientation downstream of the wall is unfavourable for stability, according to the water flow direction. Sliding occurs along a subhorizontal joint set, and along intersections between main and secondary joint sets.

Plate 32 Brecciated rock between quartzitic beds (Van Schalkwyk).

Plate 33 Erosion of the quartzites downstream of the bottom flip bucket (Van Schalkwyk).

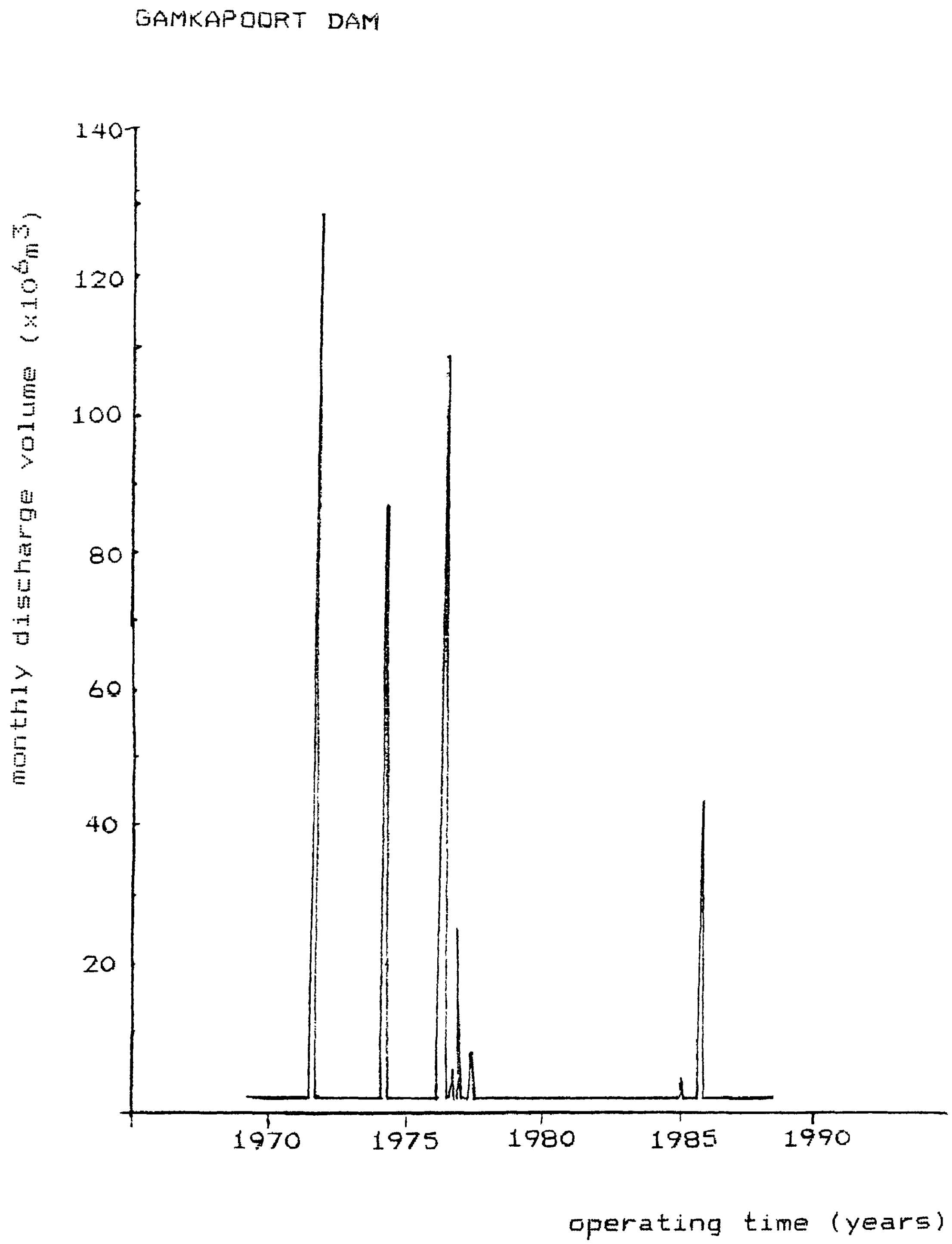


Fig.6.26 Frequency of water discharge according to hydrological information obtained from the Dept of Water Affairs.

6.5 Conclusions.

Some conclusions on the effects of the discontinuities and other rock mass properties on the erosion at the visited sites could be made.

In Table 6.4, the various rock types and the extent of erosion is shown. Igneous rocks downstream of free-falling jets have experienced greater depth of erosion than igneous rocks downstream of spillways provided with a flip bucket. Sandstone exhibits a great variation of erosion depth, whereas tillite was considerably eroded even though a flip bucket and a cut-off wall had been provided. Regardless the type of the spillway structure, igneous rocks show less erosion compared with the sandstones and shales. The effect of rock type on erodibility cannot be generalised because the information do not include a great number and a great variety of rock types.

Rocks with low values of strength exhibit greater erosion depth, mainly downstream of the free-falling jets and spillways with flip buckets (Table 6.3). The reason for this could be that the greater the strength of the rock, the greater its resistance to fracturing and consequently, to erosion.

Comparing the extent of erosion and the properties of the discontinuities (Table 6.4), it can be deduced that rock in unlined spillway channels with fairly favourable orientation of joints, exhibit greater resistance than the rock masses with unfavourably orientated joints. The joints in the rock masses downstream of free-falling jets were all unfavourable to erosion resistance. Rough and undulating joint surfaces are more resistant to movement, so that they are related to a better response of the rock mass during water discharge than the smooth and planar surfaces are. The greater the joint separation and the smaller the joint spacing of the rock, the more easily erosion will occur.

NAME OF DAM	STRUCTURE	ROCK TYPE	EROSION (m)	DRY DENSITY kgr/m^3	HARDNESS (mean average)	UN.COMPR.STRENGTH (MPa)
Kammanassie	overflow	greywacke	1,50	1 900-2 600	49	40-225
Koos Raubenheimer	overflow	sandstone	0,60	1 900-2 600	40	20-170
Donkerpoort	overflow	sandstone/greywacke	0,30	2 500	48	80-200
Hans Strydom	overflow	sandstone	0,30(7,00)	2 000	52,5	50-170
Marico Bosveld	overflow	quartzites	3,00(10,0)	2 700	31,8	30-130
Bell Park	overflow/ partially lined	dolerite	1,50	2 900		
Hartebeespoort	gated side channel partially lined chute	quartzites	1,50	2 700	40,5	50-180
Wagendrift	free-fall	shale	1,50	2 500-2 600	44,7	60-198
		dolerite	1,00	2 900	58,5	130-330
Craigie Burn	free-fall	dolerite	0,80	2 900-3 000		>100
Roodeplaat	free-fall	felsite	0,30 (1,5-2,3)	2 770	58,25	130-310
Goedertrouw	flip bucket	tillite	3,00	2 500-2 700	55,6	100-270
Vygeboom	flip bucket	granites/granitic gneisses	0,25	2 700	56,25	110-280
Gamkapoort	flip bucket	quartzitic sandstone	2,00	2 600	58,5	110-285

Table 6.3 Geological properties of the rock mass downstream of the spillways, visited in the R.S.A.

NAME OF THE DAM	RQD (%)	No OF JOINT SETS	JOINT SPACING (average value) (mm)	JOINT SEPARATION (average value) (mm)	ROUGHNESS AND WAVINESS OF THE JOINTS	JOINT ORIENTATION	EROSION (m)
Kammanassie	5	7	<300	1-5	slightly rough to rough, undulating	unfavourable	1,50
Koos Raubenheimer	10	5	50-300	<5	slightly rough, planar	fair	0,60
Donkerpoort	35	6	100-400	1-5	slightly rough, undulating	fair	0,30
Hans Strydom	5-35	5	50-300	1-10	slightly rough, planar	unfavourable	0,30(7,00)
Marico Bosveld	0-35	4	50-300	1-5	slightly rough, planar	unfavourable	3,00(10,0)
Bell Park		5	<300	5<	slightly rough, undulating	fair to unfavourable	1,50
Hartebeespoort	5-10	6	<800	<15	slightly rough to rough, planar	unfavourable	1,50
Wagendrift	35	5	100-500	2-34	smooth to slightly rough, planar	unfavourable	1,50
	30-55	5	200-900	<30	rough, undulating	unfavourable	1,00
Craigie Burn	45	4	150-600	1-10	slightly rough, planar	unfavourable	0,80
Roodeplaat	30	5	<300	<10	slightly rough to rough, undulating	unfavourable	0,30 (1,5-2,3)
Goedertrouw	27	3	60-600	<30	smooth to slightly rough, planar	unfavourable	3,00
Vygeboom	55	5	40-300	<3	rough, undulating	fair	0,25
Gamkapoort	0-20	3	<200	1-5	slightly rough to rough, undulating	fair to unfavourable	2,00

Table 6.4 Properties of the discontinuities of the rock masses in the unlined spillways, visited in the R.S.A.

CHAPTER 7 CLASSIFICATION OF THE ROCK MASSES IN UNLINED SPILLWAYS OF DAMS

7.1 Introduction.

Surface excavations in civil and mining engineering works range from comparatively small foundations trenches to road cuttings, terraces, quarries, open pits and strip mines of large extent. By means of classification systems, the general characteristics of the rock mass may be evaluated. This is used for estimating the method of excavation and construction, the support requirements and the cost of the works.

The requirements for a classification system, can be summarized as follows:

- simplicity of form, clarity of terminology and flexibility in application,
- incorporation of the most significant geological parameters, all of which must be measurable in the field and/or the laboratory,
- provision of a rating to distinguish the relative importance of the chosen classification parameters,
- ability to provide quantitative data for engineering design of specific structures to be built in rock and for the stability and resistance to failure of the rock mass during the operating time.

7.2 Literature review of the classification systems.

Classifications of rock divised by geologists usually have a genetic basis. Unfortunately, such classifications may provide little information relating to the engineering behaviour of the rocks concerned.

Terzaghi (1946) was one of the first workers to attempt an engineering classification of rock in situ for predicting tunnel support requirements. His classification was descriptive terminology and concentrates on jointing, bedding, weathering and poor quality squeezing or swelling rock.

John (1962) considered the compressive strength of rock material, the jointing and the degree of alteration in his classification. Most of the following classification systems have elaborated upon his ideas.

A major advance in the development of engineering classifications of rock was made by Coates (1964). He considered the unconfined compressive strength, the pre-failure deformation characteristics (indicating whether creep could be expected in the material at stress levels less than those required to cause failure), and the failure characteristics of the intact rock influencing the factor of safety used in design, the precautions to be taken during construction, homogenity and isotropy of the formation, and the competency of the rock mass.

Deere and Miller (1966) based their classification of intact rock on the unconfined compressive strength and the modulus of elasticity.

Wickham et al.(1972) introduced the concept of rock structure rating (RSR), which refers to the quality of rock structure in relation to ground support in tunnelling. The RSR system rates the relative effect on ground support of three parameters - geological structure, joint pattern and direction of drive and groundwater/joint condition.

The classification of rock masses by Bieniawski (1973) incorporates the RQD, the unconfined compressive strength, the spacing, separation and continuity of the joints, the groundwater conditions and the orientation of the joints with respect to a particular structure. Bieniawski grouped each of the chosen rock mass parameters into five classes, showing that the higher the total rating, the better the rock mass quality (Table 7.1). This classification system, CSIR-Geomechanics Classification System has been successfully applied in civil engineering and in mining for problems related to tunnels, caverns, slopes, foundations, mine chambers. Later, Bieniawski et al.(1976) presented a rapid site appraisal for dam foundations based on the Geomechanics Classification System.

CLASSIFICATION PARAMETERS AND THEIR RATINGS

PARAMETERS		RANGES OF VALUES				
Strength of intact rock material	Point load strength index	> 8 MPa	4 - 8 MPa	2 - 4 MPa	1 - 2 MPa	For this low range - uniaxial compressive test is preferred
	Uniaxial compressive strength	> 200 MPa	100 - 200 MPa	50 - 100 MPa	25 - 50 MPa	10-25 MPa; 3-10 MPa; 1-3 MPa
Rating		15	12	7	4	2; 1; 0
Drill core quality RQD		90% - 100%	75% - 90%	50% - 75%	25% - 50%	< 25%
Rating		20	17	13	8	3
Spacing of joints		> 3 m	1 - 3 m	0.3 - 1 m	50 - 300 mm	< 50 mm
Rating		30	25	20	10	5
Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Hard joint wall rock	Slightly rough surfaces Separation < 1 mm Soft joint wall rock	Slickensided surfaces or Gouge < 5 mm thick or Joints open 1 - 5 mm Continuous joints	Soft gouge > 5 mm thick or Joints open > 5 mm Continuous joints
Rating		25	20	12	6	0
Ground water	Inflow per 10 m tunnel length	None		< 25 litres/min	25 - 125 litres/min	> 125 litres/min
		OR		OR	OR	OR
	Ratio $\frac{\text{joint water pressure}}{\text{major principal stress}}$	0		0.0 - 0.2	0.2 - 0.5	> 0.5
		OR		OR	OR	OR
	General conditions	Completely dry		Moist only (interstitial water)	Water under moderate pressure	Severe water problems
Rating		10		7	4	0
Strike and dip orientations of joints		Very favourable	Favourable	Fair	Unfavourable	Very unfavourable
Ratings	Tunnels	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	-60
Rating		100 — 81	80 — 61	60 — 41	40 — 21	< 20
Class No		I	II	III	IV	V
Description		Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

Table 7.1 CSIR - Geomechanics classification system (Bieniawski, 1973).

Aufmuth (1973) indicated a site engineering index for rock, to provide a common language which will allow the engineering and lithologic descriptions of a rock mass to be presented in a brief, clear and relevant manner. The result of the index tests was two lists, one of the parameters used as a basis for various classifications and and the other of properties which could be of additional use for the determination of the index.

Barton et al.(1974) defined a rock mass quality index (Q) in terms of the following six parameters (Table 7.2):

1. The RQD or an equivalent system of joint density.
2. The number of joint sets (J_n) which is an important indication of the degree of freedom of a block within the rock mass. The RQD and the number of joint sets provide a crude measure of relative block size (RQD/J_n).

CLASSIFICATION OF INDIVIDUAL PARAMETERS

Description	Value
ROCK QUALITY DESIGNATION	RQD
Very poor	0-25
Poor	25-30
Fair	50-75
Good	75-90
Excellent	90-100
JOINT SET NUMBER	Ja
Massive, no or few joints	0,5-1,0
One joint set	2
One joint set plus random	3
Two joint sets	4
Two joint sets plus random	6
Three joint sets	9
Three joint sets plus random	12
Four or more joint sets, random, heavily jointed, "sugar cube", etc	15
Crushed rock, earthlike	20
JOINT ROUGHNESS NUMBER	Jr
a. rock wall contact and	
b. rock wall contact before 10cms shear	
Discontinuous joints	4
Rough or irregular, undulating	3
Smooth, undulating	2
Slickensided, undulating	1,5
Rough or irregular, planar	1,5
Smooth, planar	1,0
Slickensided, planar	0,5
c. no wall contact when sheared	
Zone containing clay minerals thick enough to prevent rock wall contact	1,0
Sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1,0
JOINT ALTERATION NUMBER	Ja
a. rock wall contact	
Tightly healed, hard, non-softening, impermeable filling	0,75
Unaltered joint walls, surface staining only	1,0
Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2,0
Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3,0
Softening or low friction clay mineral coatings i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc, and small quantities of swelling clays(discontinuous coatings)	4,0
b. rock wall contact before 10cms shear	
Sandy particles,clay-free disintegrated rock	4,0
Strongly over-consolidated, non-softening clay mineral fillings (continuous,<5mm thick)	6,0
Medium or low over-consolidated, softening, clay mineral filling(continuous,<5mm thick)	8,0
Swelling clay fillings, i.e. montmorillonite (continuous, <5mm thick). Values of Ja depend on percent of swelling clay-size particles and access to water	8,0-12,0
c. rock wall contact when sheared	
Zones or bands of disintegrated rock	6,0
Zones or bands of crushed rock and clay	8,0
Clay mineral filling	8,0-12,0
Zones or bands of silty or sandy clay, small clay fraction, (non-softening)	5,0
Thick, continuous zones of clay	10,0-13,0
Bands of clay	13,0-20,0
JOINT WATER REDUCTION FACTOR	Jw
Dry excavations or minor inflow,i.e.<5lt/min	1,0
Medium inflow or pressure, occasional outwash of joint fillings	0,66
Large inflow or high pressure in competent rock with unfilled joints	0,5
Large inflow or high pressure considerable outwash of fillings	0,33
Exceptionally high inflow or pressure at blasting, decaying with time	0,2-0,1
Exceptionally high inflow or pressure continuing without decay	0,1-0,05
STRESS REDUCTION FACTOR	SRF
a. weakness zones intersecting excavation,which may cause loosening of rock mass when tunnel is excavated	
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10,0
Single weakness zones containing clay, or chemically disintegrated rock (excavation depth <50m)	5,0
Single weakness zones containing clay, or chemically disintegrated rock (excavation depth >50m)	2,5
Multiple shear zones in competent rock (clay free) loose surrounding rock (any depth)	7,5
Single shear zones in competent rock (clay free) (depth of excavation <50m)	5,0
Single shear zones in competent rock (clay free) (depth of excavation >50m)	2,5
Loose open joints, heavily jointed or "sugar cube" (any depth)	5,0
b. competent rock, rock stress problems	
Low stress, near surface	2,5
Medium stress	1,0
High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	0,5-2,0
Mild rock burst (massive rock)	5-10
Heavy rock burst (massive rock)	10-20
c. squeezing rock,plastic flow of incompetent rock under the influence of high rock pressure	
Mild squeezing rock pressure	5-10
Heavy squeezing rock pressure	10-20
d. swelling rock,chemical swelling activity depending upon presence of water	
Mild swelling rock pressure	5-10
Heavy swelling rock pressure	10-20

Table 7.2 Parameters used in NGI Tunnelling Quality Index (Barton et al.,1974).

3. The roughness of the most unfavourable joint set (J_r). The joint roughness and the number of joint sets determine the dilatancy of the rock mass.

4. The degree of the alteration or infilling of the most unfavourable joint set (J_a). The roughness and the degree of alteration of the joint walls or the filling material provide an approximation of the shear strength or the rock mass (J_r/J_a).

5. The degree of water seepage or the joint water reduction factor (J_w).

6. The stress reduction factor (SRF). Squeezing and swelling are taken account of in the stress reduction factor. The active stress is defined as J_w/SRF.

The rock mass quality index is derived from the following equation

$$Q = (RQD/J_n)\ (J_r/J_a)\ (J_w/SRF).$$

This classification system, otherwise the N.G.I. Tunnelling Quality Index, includes sufficient information to provide a realistic assessment of the factors influencing the stability of an underground operation.

Weaver (1975) proposed a rippability rating chart (Table 7.3) which is based upon the CSIR system but uses the parameters including rock type, seismic wave velocity, rock hardness, rock weathering, joint spacing, joint continuity, and joint orientation (favourable or not).

RIPPABILITY RATING CHART

Rock class	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock
Seismic Velocity(m/s)	>2 150	2 150-1 850	1 850-1 500	1 500-1 200	1 200-450
Rating	26	24	20	12	5
Rock hardness	Extremely hard rock	Very hard rock	Hard rock	Soft rock	Very soft rock
Rating	10	5	2	1	0
Rock weathering	Unweathered	Slightly weathered	Weathered	Highly weathered	Completely weathered
Rating	9	7	5	3	1
Joint spacing(mm)	>3 000	3 000-1 000	1 000-300	300-50	<50
Rating	30	25	20	10	5
Joint continuity	Non continuous	Slightly continuous	Continuous-no gouge	Continuous-some gouge	Continuous-with gouge
Rating	5	5	3	0	0
Joint gouge	No separation	Slight separation	Separation<1mm	Gouge-<5mm	Gouge->5mm
Rating	5	5	4	3	1
Strike and dip orientation	Very unfavourable	Unfavourable	Slightly unfavourable	Favourable	Very favourable
Rating	15	13	10	5	3
Total rating	100-90	90-70	70-50	50-25	<25
Rippability assessment	Blasting	Extremely hard ripping and blasting	Very hard ripping	Hard ripping	Easy ripping

*Strike and dip orientation are revised for rippability assessment.

Table 7.3 A classification system for rippability assessment (Weaver, 1975).

A classification system by Laubscher (1977) also involved class rating according to the influence of weathering, field and induced stresses, changes in stress and the influence of strike and dip orientations.

I.A.E.G. (1979) proposed a system grouping rock masses in classes according to dry density, porosity, deformability, permeability and sonic velocity.

Based upon the NGI system, Kirsten (1982) defined four characteristic parameters, each one including some of the geological or geotechnical properties of the rock mass - mass strength number (Ms), block size number based on the ratio between the RQD and the number of different joint sets (RQD/Jn), relative ground structure number (Js), and the joint strength number expressed by the ratio between the joint roughness number and the joint alteration number (Jr/Ja) - and determined an excavatability index (N) as follows

$$N = Ms\ (RQD/Jn)\ Js\ (Jr/Ja)$$

According to this index, he proposed a classification system for excavation in natural materials (Table 7.4).

For tunnelling purposes, Price (1990), introduced a primary classification of the rock mass according to the rock type - introducing seven rock groups -, and furthermore to the strength or durability, the discontinuity number, the discontinuity orientation, the discontinuity spacing, the discontinuity filling, the discontinuity alteration, opening, roughness, continuity, water condition, cementing and consistency.

The classification systems of Weaver (1975) and Kirsten (1982) classify the rock mass into different classes according to the rock quality and they determine the excavatability or the rippability of the rock mass. For rock masses downstream of a spillway, the resistance to erosion opposes the rippability process; the more favourable to ripping a rock mass is, the more unfavourable it would be to resist the erosion forces. The parameters used are similar but the ratings of the properties differ from one system to the other. Weaver´s chart does not include the fracture frequency and the roughness. The use of seismic velocity in Weaver´s system is aimed at providing an indication of rock mass conditions (especially the effect of joints) in depth or beneath overburden (Van Schalkwyk, 1989). Both Weaver´s and Kirsten´s systems have a parameter that describes the effect of joint orientations during the rippability process.

MASS STRENGTH NUMBER FOR ROCKS (Ms)

Hardness	Identification in profile	Unconfined compressive strength (MPa)	Mass strength number (M_s)
Very soft rock	Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife. It is too hard to cut a triaxial sample by hand	1,7	0.87
		1,7- 3,3	1,86
Soft rock	Can just be scraped and peeled with a knife, indentations 1 mm to 3 mm show in the specimen with firm (moderate) blows of the pick point	3.3- 6.6	3.95
		6.6- 13.2	8.39
Hard rock	Cannot be scraped or peeled with a knife, hand-held specimen can be broken with hammer end of a geological pick with a single firm (moderate) blow	13.2- 26.4	17.70
Very hard rock	Hand-held specimen breaks with hammer end of pick under more than one blow	26.4- 53.0	35.0
		53.0-106.0	70.0
Extremely hard rock (very, very hard rock)	Specimen requires many blows with geological pick to break through intact material	106.0-212.0	140.0
		212.0	280.0

RELATIVE GROUND STRUCTURE NUMBER (Js)

Dip direction of closer spaced joint set (degrees)	Dip angle of closer spaced joint set (degrees)	Ratio of joint spacing, r			
		1:1	1:2	1:4	1:8
180/0	90	1.00	1.00	1.00	1,00
0	85	0.72	0.67	0.62	0.56
0	80	0.63	0.57	0.50	0.45
0	70	0.52	0.45	0.41	0.38
0	60	0,49	0.44	0.41	0.37
0	50	0,49	0.46	0.43	0.40
0	40	0.53	0.49	0.46	0.44
0	30	0.63	0.59	0.55	0.53
0	20	0.84	0.77	0.71	0.68
0	10	1.22	1.10	0.99	0.93
0	5	1.33	1.20	1.09	1.03
0/180	0	1.00	1,00	1,00	1.00
180	5	0.72	0.81	0.86	0.90
180	10	0.63	0.70	0.76	0.81
180	20	0.52	0.57	0.63	0.67
180	30	0.49	0.53	0.57	0.59
180	40	0.49	0.52	0.54	0.56
180	50	0.53	0.56	0.58	0.60
180	60	0.63	0.67	0.71	0.73
180	70	0.84	0.91	0.97	1.01
180	80	1.22	1.32	1.40	1,46
180	85	1,33	1.39	1.45	1,50
180/0	90	1,00	1,00	1.00	1,00

JOINT ALTERATION NUMBER (Ja)

Description of gouge	Joint alteration number (J_a) for joint separation (mm)		
	<1.0[1]	1.0-5.0[2]	>5.0[3]
Tightly healed, hard, non-softening impermeable filling	0,75	—	—
Unaltered joint walls, surface staining only	1,0	—	—
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	4.0	6.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Non-softening strongly over-consolidated clay mineral filling, with or without crushed rock	3.0*[4]	6.0*	10.0*
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0	13.0
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0*	8.0*	13.0*
Shattered or micro-shattered (swelling) clay gouge with or without crushed rock	5.0	10.0	18.0

JOINT COUNT NUMBER (Jc)

Number of joints per cubic metre (J_c)	Ground quality designation (RQD)	Number of joints per cubic metre (J_c)	Ground quality designation (RQD)
33	5	18	55
32	10	17	60
30	15	15	65
29	20	14	70
27	25	12	75
26	30	11	80
24	35	9	85
23	40	8	90
21	45	6	95
20	50	5	100

JOINT SET NUMBER (Jn)

Number of joint sets	Joint set number (J_n)
Intact, no or few joint/fissures	1.00
One joint/fissure set	1.22
One joint/fissure set plus random	1.50
Two joint/fissure sets	1.83
Two joint/fissure sets plus random	2.24
Three joint/fissure sets	2.73
Three joint/fissure sets plus random	3.34
Four joint/fissure sets	4.09
Multiple joint/fissure sets	5.00

JOINT ROUGHNESS NUMBER (Jr)

Joint separation	Condition of joint	Joint roughness number (J_r)
Joints/fissures tight or closing during excavation	Discontinuous joint/fissures	4.0
	Rough or irregular, undulating	3.0
	Smooth undulating	2.0
	Slickensided undulating	1.5
	Rough or irregular planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation	1.0
	Shattered or micro-shattered clays	1.0

DEFINITION OF EIGHT POINT EXCAVATION CLASSIFICATION SYSTEM

Material type	Class	Excavation class boundaries	Description of excavatability
Soil Detritus	1	Less than 0.01	Hard spade
	2	0.01-0.0999	Hand pick and spade
	3	0.1-0.999	Power tools
Rock	4	1.0-9.99	Easy ripping
	5	10.0-99.9	Hard ripping
	6	100.0-999	Very hard ripping
	7	1 000.0-9 999	Extremely hard ripping/blasting
	8	Larger than 10 000	Blasting

Table 7.4 Parameters used to determine excavatability of rocks (Kirsten, 1982).

7.3 New ideas for evaluation of the ratings of the parameters used in the classification systems.

Existing rock mass classification systems include most of the parameters of importance to erodibility, but none provides a satisfactory correlation with the observed erosion damage (Table 7.5). The existing systems would possibly be applicable to erodibility, if a satisfactory classification for joint orientation can be obtained.

Firstly, a classification of the rock masses of the visited sites was attempted without including the orientation of the joints. The results are shown on Table 7.5.

Since the application of Weaver's parameter is rather vague and that of Kirsten's can obviously not be applied to erodibility, a new parameter termed "possibility of movement" has been proposed (Van Schalkwyk, 1989). The new parameter is defined by the number of degrees of freedom of movement of a rock block out of the rock mass and the directions in which movement can take place. Degrees of freedom are defined as the 4 horizontal and the 2 vertical mutually perpendicular directions, along which a block can theoretically move. A loose block on a horizontal plane e.g. has 5 degrees of freedom. The direction of movement is viewed in 2 dimensions along a vertical section parallel to the general flow direction (x) of the water. The upstream direction is presented by $\underline{x}$, and the vertical up- and downward by $\underline{z}$ and z respectively. The classification of stability according to possibility of movement is shown in Table 7.6.

MOVABILITY	DEGREES OF FREEDOM	DIRECTIONS
Very stable	<2	any
Stable	2	not x, or z
Slightly unstable	3	not x, or z
Unstable	3	any
Very unstable	>3	any

Table 7.6 Classification of stability according to possibility of movement (Van Schalkwyk, 1989).

DAMS	TOTAL RATINGS FOR THE CLASSIFICATIONS								EROSION DEPTH (m)
	no joint orientations considered				joint orientations considered				
	NGI	CSIR	WEAVER	KIRSTEN	CSIR	WEAVER	KIRSTEN		
Kammanassie	0,16/0,3	38	54/57	140/280	13	57/60	67,2/134,4	56/112	1,5
Koos Raubenheimer	0,06	36	54	70	11	57	33,6	28	0,6
Donkerpoort	0,93	57	59	1 960	50	69	1 822	1 568	0,3
Hans Strydom	0,15	38	54	392,9	23	59	255,3	235,7	0,3(7,0)
Marico Bosveld	0,6	27	56	102,6	12	61	66,6	554,5	3,0(10,0)
Bell Park	0,5	35	54	525	10	57	194,2	210	1,5
Hartebeespoort	0,05	48	64	105	23	67	38,8	42	1,5
Wagendrift (shale)	0,038	37	54	163,3	12	57	60,4	65,3	1,5
(doler)	0,3	70	71	1 260	45	74	466,2	504	1,0
Craigie Burn	0,45	32	52	192,5	7	55	71,2	77	0,8
Roodeplaat	0,75	65	59	419,1	50	64	272,4	251,4	0,3
Goedertrouw	0,15	43	50	207,6	28	55	134,9	124,5	3,0
Vygeboom	3,66	63	62	13 832	38	65	5 117,8	5 532,8	0,25
Gamkapoort	0,12/0,2	44	54	117/471	29	59	76,5/330 4	70,6/282,8	2,0

Table 7.5 Classification of the rock masses in the visited sites by NGI, CSIR, Weaver and Kirsten systems.

An alternative way for determining the effect of the joint orientations is to consider if during water impact, a rock block is subjected to the full force of water, a part of it or nothing at all, along a direction normal to a joint plane forming the sides of the block and along a major discontinuity plane on which sliding will occur (Fig.5.7, 5.8, 5.9, 5.10). It must be pointed out that in case of more than three major and continuous joint sets, the conditions are very unfavourable, regardless of the orientations of the joint sets.

Combining the degrees of freedom and the results of the calculations of the water force, a strong correlation between them is obvious, and the conclusions are as follows:
- if the rock mass has only one joint set, then it will be very stable against the erosion process (<2 degrees of freedom),
- when two joint sets occur, it will be generally stable, unless the two sets are intersecting to form a line of intersection parallel to the water flow direction, or if one of the two joint sets dips downstream (2 degrees of freedom, excluding x or z direction),
- when there are three joint sets, the conditions will be slightly unstable, unless the intersection of two joint sets is directed with the water flow direction and a third joint plane is orientated parallel to the water flow direction, or where one of the three joint sets dips downstream (3 degrees of freedom, excluding x or z direction),
- when there are three joint sets and one or all of them or their lines of intersection dip downstream, the conditions will be unstable (3 degrees of freedom),
- when the rock mass is dominated by more than three main joint sets then it is very unstable (>3 degrees of freedom).

The previous conclusions mean that the classification of the orientation of the joints as favourable or not could be determined by careful observations in the field, analysis and stereographic projection of the joint survey data.

The next problem requiring solution is the rating for each class of stability of the rock blocks. Each classification system has a different approach to the subject (Table 7.7). CSIR-Geomechanics Classification System has three different joint orientation ratings, depending upon the structure under consideration (tunnels, foundations or slopes). Kirsten uses the relative ground structure number based on the ratio between the spacing of the most closely spaced set and the most widely spaced set, on the dip direction of the closer spaced joint set relative to the direction of ripping and on the dip angle of the closer spaced joint set in the vertical plane containing the direction of ripping.

STRIKE AND DIP ORIENTATIONS OF JOINTS	OR MOVABILITY OF THE ROCK MASS	RATINGS BY				
		BIENIAWSKI			WEAVER	KIRSTEN
		tunn.	found.	slop.		
very favourable	very stable	0	0	0	15	1,50
favourable	stable	-2	-2	-5	13	↑
fair	slightly unstable	-5	-7	-25	10	
unfavourable	unstable	-10	-15	-50	5	↓
very unfavourable	very unstable	-12	-25	-60	3	0,37

Table 7.7 Rating adjustment for joint orientations for each classification system.

For the classification of the visited dams, Weaver's ratings were used as they are. For Bieniawski's classification, the ratings for foundation purposes were chosen as more appropriate. For Kirsten's classification an adjustment was made (Table 7.8). Two different sets of values for Kirsten's relative ground structure number were tested. One set of values was similar to the ratings used by Van Schalkwyk (1989) and the other set was derived by calculating the mean value of the five groups into which the range of values for relative ground number could be divided. Regardless of which set was used, it was found that the result of the classification did not differ.

MOVABILITY OF THE ROCK MASS	RATINGS FOR KIRSTEN'S CLASSIFICATION		
Very stable	1,36	or	1,40
Stable	1,14		1,00
Slightly unstable	0,92		0,80
Unstable	0,70		0,60
Very unstable	0,48		0,40

Table 7.8 Ratings for relative ground structure number of Kirsten's classification, acquired by dividing the existing values in five groups and calculating the mean value for each one (first column) or using the same as Van Schalkwyk (1989) (second column).

The classifications, using the "possibility of movement" of the rock mass due to the joint orientation are shown on Table 8.5.

7.4 Results of the modified classification systems.

The correlation of erodibility classification with observed erosion for different spillway types is as follows:

7.4.1 Overflow spillway with unlined channel.

Among the overflow structures (Tables 7.9a,b), Donkerpoort Dam is indicated by all the classification systems as having the best of the rock masses, which also agrees with the observed erosion. Even though the properties of the rock such as RQD, joint conditions, water conditions, weathering, strength are not of the highest ratings, when the joint orientation is considered, the total rating is high.

The rock mass downstream of Marico Bosveld Dam is classified as being a poor rock mass by the CSIR system, which agrees with the approximately 10m deep erosion and the damage of the spillway but differs from the results of the other classification systems. It should be noted that the ratings are applicable to the rock mass which withstood the erosion and not of the most erodible material.

At Koos Raubenheimer Dam, the lowest rock mass rating has been found. Nevertheless, the observed erosion is not significant because water discharged into the channel only once during the spillway´s operating time and it was a low outflow of short duration. The rock mass downstream of the partially lined channel of Hartebeespoort Dam is classified as very good by the Weaver and CSIR systems, which is different from the results of the classifications by Kirsten and NGI.

In general, Weaver´s system results in higher ratings than the other systems and classifies all the rock masses as very hard ripping rocks. Kirsten´s classification is more related to the observed extent of erosion.

KAMMANASSIE DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	10	strength	12	Ms	140	s.velocity	26
Jn	15	RQD	3	RQD	5	hardness	10
Jr	2,0	j.spacing	10	Jn	5,0	weathering	5
Ja	2,0/1,0	j.condition	6	Js	0,48-0,40	j.spacing	10
Jw	1,0	water	7	Jr	2,0	j.continuity	0/3
SRF	5,0	orientation	-25	Ja	2,0/1,0	j.gouge	3
						orientation	3
total rating	0,16/0,3		13		76,2/134,4-56/112		57/60

KOOS RAUBENHEIMER DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	10	strength	7	Ms	70	s.velocity	26
Jn	15	RQD	3	RQD	10	hardness	10
Jr	1,5	j.spacing	10	Jn	5,0	weathering	5
Ja	3,0	j.condition	6	Js	0,48-0,40	j.spacing	10
Jw	1,0	water	10	Jr	1,5	j.continuity	0
SRF	5,0	orientation	-25	Ja	3,0	j.gouge	3
						orientation	3
total rating	0,06		11		33,6-28		57

DONKERPOORT DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	35	strength	12	Ms	140	s.velocity	26
Jn	15	RQD	8	RQD	35	hardness	10
Jr	2,0	j.spacing	10	Jn	5,0	weathering	7
Ja	1,0	j.condition	20	Js	0,92-0,8	j.spacing	10
Jw	1,0	water	7	Jr	2,0	j.continuity	3
SRF	5,0	orientation	-7	Ja	1,0	j.gouge	3
						orientation	10
total rating	0,93		50		1 803-1 568		69

HANS STRYDOM DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	25	strength	12	Ms	140	s.velocity	26
Jn	12	RQD	3	RQD	25	hardness	10
Jr	1,5	j.spacing	10	Jn	3,34	weathering	5
Ja	4,0	j.condition	6	Js	0,70-0,60	j.spacing	10
Jw	1,0	water	7	Jr	1,5	j.continuity	0
SRF	5,0	orientation	-15	Ja	4,0	j.gouge	3
						orientation	5
total rating	0,15		23		275,0-235,7		59

Table 7.9.a

MARICO BOSVELD DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	18	strength	7	Ms	70	s.velocity	26
Jn	15	RQD	3	RQD	18	hardness	10
Jr	3,0	j.spacing	10	Jn	4,09	weathering	7
Ja	1,0	j.condition	0	Js	0,70-0,60	j.spacing	10
Jw	1,0	water	7	Jr	3,0	j.continuity	0
SRF	5,0	orientation	-15	Ja	1,0	j.gouge	3
						orientation	5
total rating	0,6		12		646,9-554,5		61

BELL PARK DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	50	strength	7	Ms	70	s.velocity	26
Jn	15	RQD	8	RQD	50	hardness	10
Jr	3,0	j.spacing	0	Jn	5,0	weathering	5
Ja	4,0	j.condition	10	Js	0,48-0,40	j.spacing	20
Jw	1,0	water	10	Jr	3,0	j.continuity	0
SRF	5,0	orientation	-25	Ja	4,0	j.gouge	3
						orientation	3
total rating	0,5		10		252-210		57

HARTEBEESPOORT DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	10	strength	12	Ms	140	s.velocity	26
Jn	15	RQD	3	RQD	10	hardness	10
Jr	1,5	j.spacing	20	Jn	5,0	weathering	5
Ja	4,0	j.condition	6	Js	0,48-0,40	j.spacing	20
Jw	1,0	water	7	Jr	1,5	j.continuity	0
SRF	5,0	orientation	-25	Ja	4,0	j.gouge	3
						orientation	3
total rating	0,05		23		50,4-42		67

Table 7.9.b

Table 7.9a,b Classification of the rock masses downstream of spillways with water overflowing along the surface of the spillway wall.

7.4.2 Free-fall spillways.

All the classification systems except the one by Weaver, indicated that Roodeplaat Dam has the best downstream rock mass. In Weaver's system, the dolerite downstream of Wagendrift Dam is indicated as of better quality than the felsite in Roodeplaat Dam. Dolerite of Wagendrift Dam and felsite of Roodeplaat Dam are shown as better rock masses than the shale of Wagendrift Dam and dolerite of Craigie Burn Dam by CSIR, Weaver and Kirsen classifications. The NGI system classifies the rock masses of the visited dams in accordance with the observed erosion.

The results from the classifications (Table 7.10) differ from the observed erosion because of the lack of consideration of the hydraulic parameters which are more important to determine the response the rock mass during water discharge after the erosion process has been initiated. The geological characteristics and the orientation of the joints are important during the first stages of the scour development. After jointing and fracturing have separated the rock mass into removable blocks, discharge of the water and its characteristics become more important.

7.4.3 Spillways with flip buckets.

The rock masses downstream of the three visited spillways were classified as very hard ripping but poor rocks (Table 7.11). The granites downstream of Vygeboom Dam exhibit the least erosion and are indicated as the best rocks. At Goedertrouw Dam, although the spillway is provided with a flip bucket, the erosion extent is great and according to the classifications, the rock mass is the poorest.

WAGENDRIFT DAM (shale)

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	35	strength	12	Ms	140	s.velocity	26
Jn	15	RQD	8	RQD	35	hardness	10
Jr	1,0	j.spacing	10	Jn	5,0	weathering	5
Ja	6,0	j.condition	0	Js	0,48-0,40	j.spacing	10
Jw	1,0	water	7	Jr	1,0	j.continuity	0
SRF	10,0	orientation	-25	Ja	6,0	j.gouge	3
						orientation	3
total rating	0,038		12		78,4-65,3		57

WAGENDRIFT DAM (dolerite)

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	30	strength	15	Ms	280	s.velocity	26
Jn	15	RQD	8	RQD	30	hardness	10
Jr	3,0	j.spacing	20	Jn	5,0	weathering	7
Ja	4,0	j.condition	20	Js	0,48-0,40	j.spacing	20
Jw	1,0	water	7	Jr	3,0	j.continuity	5
SRF	5,0	orientation	-25	Ja	4,0	j.gouge	3
						orientation	3
total rating	0,3		45		604,8-504		474

CRAIGIE BURN DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	45	strength	7	Ms	70	s.velocity	26
Jn	15	RQD	8	RQD	45	hardness	10
Jr	1,5	j.spacing	10	Jn	4,09	weathering	5
Ja	2,0	j.condition	0	Js	0,48-0,40	j.spacing	10
Jw	1,0	water	7	Jr	1,0	j.continuity	0
SRF	5,0	orientation	-25	Ja	4,0	j.gouge	1
						orientation	3
total rating	0,45		7		92,4-77		55

ROODEPLAAT DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	30	strength	15	Ms	280	s.velocity	26
Jn	12	RQD	8	RQD	30	hardness	10
Jr	3	j.spacing	10	Jn	3,34	weathering	7
Ja	2	j.condition	25	Js	0,70-0,60	j.spacing	10
Jw	1,0	water	7	Jr	1,0	j.continuity	5
SRF	5,0	orientation	-15	Ja	6,0	j.gouge	1
						orientation	5
total rating	0,75		50		293,4-251,4		64

Table 7.10 Classification of the rock masses downstream of spillways with free falling jets.

GOEDERTROUW DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	27	strength	15	Ms	210	s.velocity	26
Jn	9	RQD	8	RQD	27	hardness	10
Jr	1,0	j.spacing	10	Jn	2,73	weathering	3
Ja	4,0	j.condition	0	Js	0,70-0,60	j.spacing	10
Jw	1,0	water	10	Jr	1,0	j.continuity	0
SRF	6,0	orientation	-15	Ja	10,0	j.gouge	1
						orientation	5
total rating	0,15		28		145,3-124,6		55

VYGEBOOM DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	55	strength	15	Ms	210	s.velocity	26
Jn	12	RQD	8	RQD	55	hardness	10
Jr	3,0	j.spacing	10	Jn	3,34	weathering	7
Ja	0,75	j.condition	20	Js	0,48-0,40	j.spacing	10
Jw	1,0	water	10	Jr	3,0	j.continuity	5
SRF	5,0	orientation	-25	Ja	0,75	j.gouge	4
						orientation	3
total rating	3,66		38		6 639-5 532		65

GAMKAPOORT DAM

NGI		CSIR-GCS		KIRSTEN		WEAVER	
properties	rating	properties	rating	properties	rating	properties	rating
RQD	10/20	strength	15	Ms	210	s.velocity	26
Jn	12	RQD	3	RQD	5/20	hardness	10
Jr	1,5	j.spacing	10	Jn	3,34	weathering	5
Ja	2,0	j.condition	6	Js	0,70-0,60	j.spacing	10
Jw	1,0	water	10	Jr	1,5	j.continuity	0
SRF	5,0	orientation	-15	Ja	4,0	j.gouge	3
						orientation	5
total rating	0,12/0,2		29		82,5/330-70/282		59

Table 7.11 Classification of the rock masses downstream of spillways with energy dissipators (flip buckets).

7.5 Conclusions.

Field data of more dams should be added in the future, for better comparison and evaluation of the different systems. The modified classifications do not provide a clear distinction between rock masses that have undergone different degrees of erosion. The reasons are mainly as follows:

- the remaining (investigated) rock has resisted erosion and does not represent the parts that have been removed (Van Schalkwyk, 1989),
- the rock mass parameters and the relative ratings that are used in the classification systems are not necessarily applicable to erodibility,
- the effect of hydraulic parameters - flow characteristics and the geometry of the spillway - has not been taken into account,
- the effect of the operating time has not been taken into account,
- the use of descriptive and sometimes ambiguous phrases in the definition of geotechnical information required for the rock mass classifications results in less reliable results.

CHAPTER 8 REMEDIATION.

Geotechnical factors control the selection of appropriate cost-effective, remedial and preventative, engineering techniques, capable of minimizing existing and potential spillway channel erosion, reducing downstream impact and providing a high degree of safety and performance of spillway structures.

Remedial action options range from a "do nothing" alternative to expensive blankets of reinforced concrete, covering the entire spillway discharge channel (Cameron et al., 1988). Repair work which has been done downstream of spillways of dams includes concrete lining (Copeton, Blue River, Salinas Dam), grouting and anchoring (Alder Dam), lining of the sides of a pool with massive walls of rolled concrete incorporating drainage galleries and stressed anchors (Tarbela Dam), protection of vertical surfaces of exposed beds with shotcrete and wire mesh (Lake Brownwood, Salinas Dam), filling, levelling and seeding all the irregularities (Saylorville Dam), backfilling of fault zones with concrete (Alder Dam), excavation and reshaping of the eroded area where headcutting and knick-points occur (Lake Brownwood Dam), construction of bypass spillways (Kariba, Picote Dam).

Methods, which have been used and proposed for remediation are as follows:

- Grouting

with which fissures, joints, cavities would be sealed off against water in rock. Cement- or chemically-based mixtures are injected into voids and open discontinuities that cannot normally be reached by workers or equipment (Cameron et al., 1988). It serves to consolidate and strengthen the rock mass by increasing the rock strength and bearing capacity and reducing the deformation of the rock.

- Lean concrete (low-cement content)

which is a void and open discontinuity-filling material for conditions where high structural loads are not present. It will prevent plucking and increase erosion resistance of the rock mass.

- Shotcrete

consists of dry mixed mortar components (cement and aggregate), mixed together with water. It can be placed with a low water/cement ratio and therefore can achieve high compressive strength. It should not be a primary channel surface material and it should be protected from uplift pressures (Cameron et al., 1988).

- High strength concrete

can be reinforced or unreinforced. It provides the rock with additional abrasive resistance and compressive strength to the spillway area (Cameron et al., 1988). It may be susceptible to undercutting erosion.

- Soil cement

where small percentages of cement are added and mixed with soil, prior to compaction. It is used to fill in relatively large voids. It has high shear strength and very low compressibility, when placed in thick (more than 1m) bodies (Cameron et al., 1988).

- Dental concrete

is used to fill in joint-bounded surfaces on dipping, jointed bedrock. The resulting smooth channel surfaces are designed to keep the water from flowing into open joints and creating uplift forces that can force apart and separate individual joint blocks for plucking. Where discontinuity frequency is excessively high, the ability of dental concrete to hold channel surface blocks to the spillway is probably minimal (Cameron et al., 1988).

- Rock bolting

is a method of distributing compressive forces across discontinuity surfaces, either to resist sliding or to bring individual discontinuity bounded rock mass blocks closer together, in tight contact and to offer resistance to uplift (Cameron et al., 1988).

- Wire mesh

stretched over the channel and anchored to the rock would be adequate, if weathering is not extensive (Cameron et al., 1988). It is subject to corrosion and damage, but it is acceptable for a limited life. Plastic-coated and double-twist wire mesh are improved types.

- Relief of uplift pressures

which are created in a rock mass with widely spaced discontinuities or where the rock has been grouted or covered by concrete has to be provided. This can be done by means of boreholes terminated below the most distinct lateral bounding discontinuity and backfilled with small diameter granular material, graded to act as a barier to sedimentation, plugging of the relief hole, yet porous enough to allow pressure relief (Cameron et al., 1988).

- Cut-off walls

might be installed to an appropriate depth, at the top of the spillway and/or at other locations along the channel. Materials of which the wall could be constructed are concrete, sheet piles, logs, gabions, e.t.c. It is effective when the wall is keyed into a competent rock unit with minimal erosion potential and when a series of walls is used (Cameron et al., 1988).

- Dissipators

are constructed of concrete blocks, gabions, rip-rap or any other durable material that will effectively dissipate flow energy to preclude unacceptable erosion (Cameron et al., 1988). Stair-step energy dissipators are formed to slope backwards, into the upstream section of a spillway, creating numerous inclined surfaces that dissipate flow energy as the water travels up each surface. Rip-rap is normaly used in the outlet channel adjacent to the downstream end of the structure. To resist scour from high exit velocities, the rip-rap should be bedded on a graded material. The grading should be such that the underlying material can not be washed out.

- Ski-jump or flip bucket structures

are aimed to throw the water well clear of the spillway. There is a possible distinction between the ski-jump spillway where the jet leaves the chute essentially horizontally, and the flip-bucket where the jet is deflected upward to induce disintegration in the air (Thomas, 1976). In case of a steep area of competent rock covering another weak rock mass (knick-point), a modified ogee-weir (ski-jump structure) can be installed in this particular area to shoot away the water and reduce the possibility of undercutting. It is less expensive than a complete channel lining and sometimes more effective (Cameron et al, 1988).

- Stilling basins

are usually associated with overflow dams of the gravity type. Concrete walls and floors must be adequately anchored to rock and measures must be taken to preclude fluctuating pressures from entering any drainage system. "Dentates" are often provided to assist in dissipation of energy (Thomas, 1976). In addition, downstream of free falling jets, protection because of the developed hydraulic pressures can be succeeded by a water cushion of great tailwater depths or stilling basin (Hartung et al., 1973). The length and depth can be determined in the same manner as for the conventional stilling basin. Eschavez et al.(1985) proposed a new type of stilling basin which reduces excavation costs. It has an aeration step, a ramp, two rows of baffle blocks and, at the end, a combined weir, that gives the required tailwater for each discharge. This aerated stilling basin does not have cavitation problems and can be used for velocities higher than those generally considered safe. It requires much less excavations than the traditional high head stilling basins. Energy is dissipated adequately and the basin drains after discharge.

- Flow rerouting

reduces the possibility of water going over a spillway and does not improve the downstream area in any way. This option is very expensive and it should be considered as a last resort. It would be important to evaluate the entire reservoir area for another possible location of the spillway. There are situations where relocation of the spillway would be less expensive than an additional one (Cameron et al., 1988).

- Air admission

may be provided by use of steps or ramps, under the discharged jet, near the concrete surface to be protected. Cavitation damage and its prevention in hydraulic stuctures is of increasing concern to designers and operators of large dams. Relief is sought by forced aeration of the wall jets. The resulting air water mixture can have a considerably reduced compressibility, bulk modulus and therefore reduce or eliminate the cavitation damage potential in risk situations.

- Removal of wood vegetation and/or erosional outliers

such as access roads, fences, boulders, a fault scarp or similar feature or vegetation which can cause turbulent flow concentration, which in turn increases the erosive forces and increases the velocity and flow rate in portions of the area.

Choosing the most appropriate combination of techniques for a given spillway is further complicated by hydraulic design variables, geotechnical conditions, public safety, downstream impacts and the importance and present use of the reservoir (Cameron et al., 1988). Regular maintenance is vital to the success of an erosion and sediment control system. Control measures must be inspected frequently and repaired as soon as problems arise.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS.

This research was based on a literature survey and study of a number of existing dams with unlined spillways. Although a number of important conclusions could be made, considerable difficulty was experienced because of the lack of well-described, preliminary geological and hydrological information on the various cases. Strong correlation between the observed erosion and the geological and hydraulic factors has been established.

The basic character of the rock (sedimentary, metamorphic or igneous) is the starting point, since that indicates the origin of the rock and its major components.

Knowledge of the tectonic history provides information on faulting and jointing. If there have been no significant fault movements, then the bedding planes and other joints must be studied. A careful geological survey of an appreciable area surrounding the site of the work in question is essential. Detailed characterization and mapping, with special attention to the rock mass and lithostratigraphic and structural discontinuities which play a major role in the erosion of the rock in the area downstream of the spillway, are necessary.

The survey of the jointing of the rock mass should include every factor describing the nature of the joints in terms of length, continuity, spacing, aperture, infilling material and joint surface properties (roughness, waviness etc) and factors describing the spatial position and the orientation of the joints. Stereographic projection containing all the details of the conducted joint survey and the parameters concerning the water jet (direction of flow and water force) are important for the understanding of the conditions of each site.

Fracturing of the rock mass, leads to the formation of rock blocks. Depending on the properties of the joints, these blocks will be either removable or not. The shape and the size of the blocks, determined by the orientation and spacing of the discontinuities, influence the permeability and the strength of the rock mass, and consequently the resistance to further fracturing, movement and erosion. If the blocks are relatively small (joint spacing <300mm) and the joint walls have no contact and they are smooth without filling, the blocks will be prone to movement or complete removal by uplift and plucking, the result being in general an extensive and quickly formed scour. Tight, rough joint surfaces exhibit high shear strength and consequently great resistance to erosion process. Filling and its properties should be recorded as a parameter affecting the erodibility of the rock mass.

Except under severe conditions, unweathered joints will be very resistant to erosion, while the orientation of open, weathered and erodible joints may be of great significance. Once the rock is highly jointed (more than three joint sets), the conditions are very unfavourable and the joint orientation is less important. The direction of the various joint sets that determine the shape of the rock block, compared with relation to the direction of the water flow, affects the likelihood of erosion. Unfavourable conditions induce movement of the rock blocks during water discharge and favourable conditions contribute to stability and erosion resistance. Three continuous joint sets contribute to critical conditions because they form possibly removable blocks, and if the degrees of freedom are more than three and the other joint parameters assist removal of the rock in a downstream direction and decrease stability, the rock blocks forced by water discharge will move and create space for more blocks to advance and for erosion to be developed. The most unfavourably orientated joints are those that dip with a great angle in a downstream direction, and strike perpendicular to the flow. In addition, a joint set dipping at a shallower angle than the gradient of the spillway area and striking perpendicular to the water flow direction, constitutes a plane along which movement can occur. Sliding can also take place along a line of intersection of two joint sets, if the line is directed parallel to the water flow and dips with a smaller angle than the gradient of the downstream area.

CSIR, Barton's, Weaver's and Kirsten's classification systems were used and some adjustment of the favourability or not of the joint orientation was made. The classifications concerned the rock mass which withstood the erosion, so that the results were not always in accordance with the observed erosion extent. Classification of the rock masses would be applicable during the initial stages of erosion before removal of the rock blocks and scour occur and before the hydraulic parameters start playing an important role in the behaviour of the rock.

Removability of the rock mass in an unlined spillway depends on the impact and transporting power of the discharged water. Discharge, head drop, velocity of the water and duration of the outflow event are important factors. The greater their values, the higher the scour potential will be.

In channel spillways, the kinetic energy may be entirely due to horizontal velocity. Water flowing parallel to a rock surface, enters cracks in the rocks, exerts pressure on the bottom of the rock and on sides, while water flowing on the top causes shear stress along the top surface. The resistance of the rock to uplift depends on its shape, weight, shear forces between adjoining blocks, joint surface and filling material of the joints. On the bedrock downstream of free-falling jets, the impacting jet exerts a combination of forces which vary as the pool develops. Upon entering the tailwater, the jet produces dynamic pressure fluctuations on the river bed which are transmitted into and along fissures in the rock, causing it to break up into blocks. The process continues until an ultimate depth is reached. Both the horizontal and the vertical velocities of the jet falling on the downstream area cause rolling actions in the plunge pool.

Detailed maps and information for the engineering design and the geometry of the downstream area, as well as hydrological history during the operating time of the spillway since its completion, are important for the calculation of the forces and pressures developed on the downstream rock mass and for the correlation among the various sites.

As the roughness increases due to removal of rock blocks during water flow, unstable conditions are induced because of the protruding blocks that give rise to considerable water forces. The location and geometry of changes in the channel gradient (abrupt waterfalls, as a series of closely spaced stair-steps or as gentle, subtle changes) are often controlled by discontinuities and contribute to the roughness of the surface of the downstream area.

To avoid erosion damage, preliminary studies are necessary so that the most suitable design of the spillway, according to the downstream conditions of the rock mass, will be chosen. It would be appropriate for a spillway channel to be cleared with a high pressure jets to provide better conditions for investigating the erodibility of the rock mass. If the channel consists of a resistant rock mass, it could be left unlined with a smooth, slight downstream dipping surface. Every existing obstruction to the water flow should be removed. In case of indications of potential erosion development, precautionary measures such as concrete lining, grouting, anchoring and/or energy dissipators should be provided. It would often be advisable, in case of good rock conditions downstream of a free-falling jet, for a plunge pool to be excavated and shaped before the first outflow discharge. Then, the tailwater must first be penetrated by the jet and much energy is lost before reaching the rock where damage could be done. This would prevent the development of the pool in a way which would endanger the structure. In the case of free-falling jets and poor rock conditions, a stilling basin should be planned.

Yearly inspection of the sites, by means of field survey and mapping, is recommended to provide accurate and useful reports for the forecast of the rock mass response against future flood events. In case of scouring or erosion, appropriate remediation such as grouting, re-excavation and re-shaping of the area, removal of any erosional outliers etc, should follow, depending on the geological and hydraulic parameters of the specific area.

Development of the key block theory for application on the rock masses downstream of the spillways should assist in the prediction of the erodibility. In the future, more work is needed in order to determine the effect of hydraulic parameters such as velocity and power of the water flow, discharge and duration of the outflow, on the erodibility. Laboratory and in situ tests to determine the rock material properties such as compressive strength, as well as shear strength along the major joints, are considered of great necessity. Therefore, more dams should be visited and detailed information of the geological and hydraulic factors be obtained. Then, classification of the data into different categories according to the spillway type is essential. More data will improve correlation between different cases and the parameters that affect erodibility of the rock mass.

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APPENDIX 1

CONVERSION FACTORS

CONVERSION FACTORS	SYSTEMS IMPERIAL	SI
Length	1ft	0,3048m
	1in	25,40mm
Dry density	1pcf	$1{,}601 \times 10^{1}$kg/m^3
Volume	1morgen foot	$2{,}610 \times 10^{6}$m^3
	1acre foot	$1{,}233 \times 10^{3}$m^3
Pressure or stress	1psi	$6{,}894 \times 10^{3}$Pa
Force	1kilopond	$9{,}806 \times 10^{0}$N
Permeability	1ft/year	$0{,}9659 \times 10^{-8}$m/s

SI unit prefixes

PREFIX	tera	giga	mega	kilo	milli	micro	nano	pico
SYMBOL	T	G	M	k	m	u	n	p
MULTIPLIER	10^{12}	10^{9}	10^{6}	10^{3}	10^{-3}	10^{-6}	10^{-9}	10^{-12}

SI symbols and definitions

N = Newton = kg m s^{-2}
Pa= Pascal = kg m s^{-2}
J = Joule = kg m s^{-2} m
W = Watt = kg m s^{-2} m s^{-1}

APPENDIX 2

INFORMATION ABOUT EVERY VISITED SITE

<u>DATA</u>

NAME OF THE DAM	Kammanassie Dam
COMPLETION YEAR	1923
NAME OF THE RIVER:	Kammanassie
NEAREST TOWN	Oudtshoorn
PROVINCE	Cape
SPILLWAY TYPE	Overflow
HEIGHT OF THE CREST(m)	
LENGTH OF THE CREST(m)	183
MAX.DISCHARGE CAPACITY(m^3/s)	2 830
WATER FLOW DIRECTION(^{o})	356/301
ROCK TYPE	Greywacke (sandstone)
DRY DENSITY(Kgr/m^3)	1 900-2 600
SCHMIDT HAMMER HARDNESS	49
UN.COMPR.STRENGTH(MPa)	40-225
MODULUS OF ELASTICITY(GPa)	32-85

JOINT SURVEY LOCALITY	1	2	3	4
RQD(%)	5	5	5	5
No OF JOINT SETS	6	7	5	7
JOINT SPACING(mm)	<300	1 000	<300	<300 400
JOINT SEPARATION(mm)	<1	>5	1-5	<1 5
J.DIP ANGLE(^{o})	28 30 88 85 82 60	80 88 80 88 75 56 30	14 70 80 77 80	15 25 70 75 88 76 80
J.DIP DIRECTION(^{o})	232 348 282 258 203 074	171 256 358 028 106 204 193	305 199 175 290 076	303 183 198 293 256 103 178
J.STRIKE(^{o})	322 078 012 348 293 164	261 346 088 118 196 294 283	035 289 265 020 166	033 273 288 383 346 193 268
GOUGE		staining only soft,sandy	no filling	no filling
ROUGHNESS	slightly rough	rough to slightly rough	rough and undulating	slightly rough
J.WATER CONDITION	no	moisture only	moist	damp
EROSION ESTIMATED(m)	1,50			

JOINT SURVEY LOCALITY

1: Right flank of the unlined channel.

2: Along the part of the channel, immediately downstream of the spillway.

3: Near the turn of the left concrete wall of the channel.

4: Downstream of the concrete road, crossing the unlined channel

RESERVOIR RECORD – SPILLWAY DISCHARGE

RESERVOIR Kammanassie

STATION No J3R01

All figures are in $10^6 m^3$

YEAR

MONTH	1922	1923	1924	1925	1926	1927	1928	1929	1930	1931	1932	1933	1934	1935	1936	1937	1938	1939	1940	1941	1942	1943	1944	1945
Jan											11,8													
Feb											0,1													
Mar											1,2													
Apr																								
May														60,5										
Jun														2,2										15,1
Jul																								10,9
Aug														2,4										
Sep				14,8										9,3										
Oct											3,4													
Nov																								
Dec																								

YEAR

MONTH	1946	1947	1948	1949	1950	1951	1952	1953	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969
Jan						21,3																		
Feb						0,1																		
Mar										0,6														
Apr																								
May																								
Jun									3,2													17,2		
Jul						1,8			5,6													4,6		
Aug						2,7			65,2													1,3		
Sep						4,4			8,2													0,5		
Oct								12,0	1,8													0,1		
Nov					3,8			5,2	0,1										3,5					
Dec								0,3																

MONTH	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981
Jan												
Feb												
Mar												
Apr												
May												
Jun												
Jul												
Aug												
Sep												
Oct												
Nov												
Dec												

NAME OF THE DAM	Koos Raubenheimer Dam
COMPLETION YEAR	1971
NAME OF THE RIVER	Klein Le Roux
NEAREST TOWN	Oudtshoorn
PROVINCE	Cape

SPILLWAY TYPE	Overflow
HEIGHT OF THE CREST(m)	
LENGTH OF THE CREST(m)	0,0
MAX.DISCHARGE CAPACITY(m^3/s)	935
WATER FLOW DIRECTION(^{o})	226

ROCK TYPE	weathered sandstone with veins of clay
DRY DENSITY(Kgr/m^3)	1 900-2 600
SCHMIDT HAMMER HARDNESS	40
UN.COMPR.STRENGTH(MPa)	20-170
MODULUS OF ELASTICITY(GPa)	25-70

JOINT SURVEY LOCALITY	1		2				
RQD(%)			10				
No OF JOINT SETS			5				
JOINT SPACING(mm)			50-300				
JOINT SEPARATION(mm)			<5				
J.DIP ANGLE(^{o})	50	83	50	78	45	48	80
J.DIP DIRECTION(^{o})	163	058	171	228	013	041	058
J.STRIKE(^{o})	253	148	261	318	103	131	148
GOUGE			clayey	no	no	no	no
ROUGHNESS			slightly rough,planar				
J.WATER CONDITION			no				
EROSION ESTIMATED(m)	0,60						

JOINT SURVEY LOCALITY
1: main joint sets on the flanks
2: floor of the spillway channel

RESERVOIR RECORD - SPILLWAY DISCHARGE
RESERVOIR Koos Raubenheimer
STATION No J3R03
All figures are in $10^6 m^3$

MONTH	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985
Jan													
Feb													
Mar													
Apr													
May													
Jun													
Jul													
Aug					0,8								
Sep					0,3								
Oct					0,1								
Nov													
Dec													

DATA

NAME OF THE DAM	Donkerpoort Dam
COMPLETION YEAR	1959
NAME OF THE RIVER.	Klein Nyl
NEAREST TOWN	Nylstroom
PROVINCE	Transvaal
SPILLWAY TYPE	Overflow
HEIGHT OF THE CREST(m)	5,3
LENGTH OF THE CREST(m)	42,5
MAX.DISCHARGE CAPACITY(m^3/s)	
WATER FLOW DIRECTION(^{o})	129
ROCK TYPE	greywacke/sandstone
DENSITY(Kgr/m^3)	2 500
SCHMIDT HAMMER HARDNESS	48
UN.COMPR.STRENGTH(MPa)	80-200
MODULUS OF ELASTICITY(GPa)	60-80

JOINT SURVEY LOCALITY	1			2						3				
RQD(%)				35						35				
No OF JOINT SETS	3			6						5				
JOINT SPACING(mm)										100	100	400		
JOINT SEPARATION(mm)										<5	<5	1		
J.DIP ANGLE(^{o})	15	60	85	08	55	80	63	77	77	10	61	84	84	79
J.DIP DIRECTION(^{o})	099	257	337	141	241	345	275	116	206	185	243	339	102	309
J.STRIKE(^{o})	189	347	067	231	331	075	005	206	296	275	333	069	192	039
GOUGE				no, staining only						no, staining only				
ROUGHNESS				slightly rough						slightly rough				
JOINT WATER CONDITION				minor inflow						minor inflow				
EROSION ESTIMATED(m)	0,30													

JOINT SURVEY LOCALITY

1: left flank of the unlined channel.

2: first position along the unlined channel.

3:second position further downstream, along the unlined channel

NAME OF THE DAM	Hans Strydom Dam
COMPLETION YEAR	1980
NAME OF THE RIVER	Mogol
NEAREST TOWN	Ellisras, Vaalwater
PROVINCE	Transvaal
SPILLWAY TYPE	Overflow
HEIGHT OF THE CREST(m)	1,00
LENGTH OF THE CREST(m)	192
MAX.DISCHARGE CAPACITY(m^3/s)	1 716
WATER FLOW DIRECTION(°)	003-358
ROCK TYPE	weathered quartzitic sandstone
DENSITY(Kgr/m^3)	2 000
SCHMIDT HAMMER HARDNESS	52,5
UN.COMPR.STRENGTH(MPa)	50-170
MODULUS OF ELASTICITY(GPa)	38-70

JOINT SURVEY LOCALITY	1	2	3	4	5
RQD(%)	35	5	5	35	35
No OF JOINT SETS	3	3	3		
JOINT SPACING(mm)		<300	<300	50-200	<300
JOINT SEPARATION(mm)	5 <5	<10	1-5	<10	<10
J.DIP ANGLE(°)	45 80 85	45 60 65	24 47 84	25 85 85	05 85 87 90
J.DIP DIRECTION(°)	135 198 135	030 208 138	024 218 313	048 085 264	109 146 033 322
J.STRIKE(°)	225 288 225	120 298 228	114 308 043	138 175 354	199 236 123 052
GOUGE		soft, clay	free, easily broken	material	
ROUGHNESS	rough	slightly rough	slightly rough	slightly rough	slightly rough
J.WATER CONDITION		no	moist only		

EROSION ESTIMATED(m) 0,30-7,00

SCOURED GORGE DIRECTION(°) 297

JOINT SURVEY LOCALITY
1: right flank of the spillway channel
2: downstream bedrock, close to the spillway wall
3: some metres further downstream of the second position
4: close to the scoured gorge
5: area downstream of the scoured gorge

RESERVOIR RECORD - SPILLWAY DISCHARGE
RESERVOIR Hans Strydom
STATION No A4R01
All figures are in $10^6 m^3$

	YEAR								
MONTH	1980	1981	1982	1983	1984	1985	1986	1987	1988
Jan		36,0							
Feb		98,3							
Mar		60,5							
Apr		19,0							
May		14,3							
Jun		9,0							
Jul		7,9							
Aug		7,0							
Sep		5,3							
Oct									
Nov									
Dec									

MAXIMUM DAILY AVERAGE FLOW 78,65 cubic metres per second

DATE OF MAX DAILY AVERAGE FLOW 7 February 1981

DURATION OF THE FLOW 293 days

NAME OF THE DAM	Marico Bosveld d
COMPLETION YEAR	1933 (spillway r[illegible])
NAME OF THE RIVER	Groot Marico
NEAREST TOWN	Zeerust
PROVINCE	Transvaal
SPILLWAY TYPE	Overflow
HEIGHT OF THE CREST(m)	3,32 (old structure 3,59)
LENGTH OF THE CREST(m)	70 (old structure 130)
MAX.DISCHARGE CAPACITY(m^3/s)	1 243 (old structure 1 440)
WATER FLOW DIRECTION(^{o})	000
ROCK TYPE	Quartzite
DENSITY(Kgr/m^3)	2 700
SCHMIDT HAMMER HARDNESS	32,1
UN.COMPR.STRENGTH(MPa)	30-130
MODULUS OF ELASTICITY(GPa)	40-56
RQD(%)	0-35
No of JOINT SETS	4
JOINT SPACING(mm)	50-300
JOINT SEPARATION(mm)	1-5 (often >5)
GOUGE	no, joint walls only staining
ROUGHNESS	slightly rough
J.WATER CONDITION	minor inflow
EROSION ESTIMATED(m)	3,00-10,00 (when the spillway failed)

RESERVOIR RECORD - SPILLWAY DISCHARGE

RESERVOIR Marico Bosveld

STATION No A3R01

All figures are in $10^6 m^3$

MONTH	1935	1936	1937	1938	1939	1940	1941	1942	1943	1944	1945	1946	1947	1948	1949	1950	1951	1952	1953	1954	1955	1956	1957
Jan						0,7	2,2			2,5													
Feb		11,9					1,9			53,4		18,0									11,1	7,6	1,9
Mar		19,7			9,0	0,6				12,7	5,1	13,6									0,7	1,5	7,3
Apr		17,7			0,2	0,3			2,9	1,1	1,1	6,2									0,3	0,1	
May		13,8							5,9													0,0	
Jun		5,6							1,9	15,5													
Jul		0,2							1,4	4,0													5,0
Aug																							0,0
Sep									8,4	2,7													1,5
Oct		dam breaches							7,2	3,4													2,1
Nov						6,7			30,7	5,5													0,0
Dec					0,5	0,0			8,3	0,1												0,9	0,3

MONTH	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981
Jan	1,9	0,0		0,1						6,7					0,2			0,3	18,3		41,2	1,1	0,1	18,7
Feb	1,2			3,2						3,9					1,2			0,9	11,5	0,2	25,1	1,2	0,1	1,3
Mar	0,0			0,1						4,7	0,3				2,8			5,3	35,2	0,4	9,7	0,1	0,1	0,5
Apr	0,0			4,6						12,7	0,3			2,4	1,5			10,6	6,2	0,5	5,3	0,1	0,0	0,1
May				0,0						0,0	0,0				0,0			1,4	5,0	0,2	3,0	0,1	0,0	0,1
Jun				0,1						0,0								0,0	0,6		1,6	0,1		0,0
Jul																			0,5	0,1	1,2			0,0
Aug																			0,2		1,2			
Sep																					1,3			
Oct																			1,5	1,3	1,0			
Nov																			0,3	0,0	0,3	0,5		
Dec	0,0		1,0															3,5	0,0	3,7	0,0	0,0	0,5	

MONTH	1982	1983	1984	1985	1986	1987	1988
Jan							
Feb							
Mar							
Apr							
May							
Jun							
Jul							
Aug							
Sep							
Oct							
Nov							
Dec							

DATA

NAME OF THE DAM	Bell Pa
COMPLETION YEAR	
NAME OF THE RIVER	Sterkspruit
NEAREST TOWN	
PROVINCE	Natal
SPILLWAY TYPE	Overflow, partially lined
HEIGHT OF THE CREST(m)	
LENGTH OF THE CREST(m)	
MAX.DISCHARGE CAPACITY(m^3/s)	
WATER FLOW DIRECTION(^{o})	037
ROCK TYPE	Dolerites
DENSITY(Kgr/m^3)	2 900
SCHMIDT HAMMER HARDNESS	
UN.COMPR.STRENGTH(MPa)	
MODULUS OF ELASTICITY(GPa)	

JOINT SURVEY LOCALITY			1					2	
RQD(%)									
No OF JOINT SETS			4					5	
JOINT SPACING(mm)					$\pm$300				
JOINT SEPARATION(mm)			open					open	
J.DIP ANGLE(^{o})	05	05	83	86	10	17	75	88	90
J.DIP DIRECTION(^{o})	326	226	231	035	270	156	236	055	346
J.STRIKE(^{o})	056	316	321	125	360	249	326	145	076
GOUGE	soft, sandy material in some subhorizontal joints								
ROUGHNESS	slightly rough, undulating surfaces								
J.WATER CONDITION	no								
EROSION ESTIMATED(m)	1,50								

JOINT SURVEY LOCALITY
1: close to the downstream end of the spillway slab
2: further downstream of the first position

DATA

NAME OF THE DAM	Hartebeespoort Dam
COMPLETION YEAR	1925 (additional work in 1971)
NAME OF THE RIVER	Crocodile
NEAREST TOWN	Pretoria
PROVINCE	Transvaal
SPILLWAY TYPE	Overflow, gated side, partially lined channel
HEIGHT OF THE CREST(m)	4,6
WIDTH OF THE CHUTE EXIT(m)	38,6
MAX.DISCHARGE CAPACITY(m^3/s)	2 322
WATER FLOW DIRECTION(^{o})	039
ROCK TYPE	Quartzite
DRY DENSITY(kg/m^3)	2 700
SCHMIDT HAMMER HARDNESS	40,5
UN.COMPR.STRENGTH(MPa)	50-180
MODULUS OF ELASTICITY(GPa)	57-73

JOINT SURVEY LOCALITY			1					2				3	
RQD(%)			5					10				5	
No OF JOINT SETS			6					4				3	
JOINT SPACING(mm)	500	4	70	800	400	80	2	350	100	20	40	250	20
JOINT SEPARATION(mm)	1	2	10	2-3	1	15	200	2-3	1	10	1	1	5
J.DIP ANGLE(^{o})	76	80	75	85	65	20	43	86	68	20	73	72	90
J.DIP DIRECTION(^{o})	062	152	210	084	125	002	161	157	126	002	154	131	001
J.STRIKE(^{o})	152	242	300	174	215	092	287	347	216	092	244	221	091
GOUGE	sandy							sandy	sandy			sandy	
ROUGHNESS	slightly rough					slightly rough						rough	
J.WATER CONDITION					moisture							no	
EROSION ESTIMATED(m)	1,50												

JOINT SURVEY LOCALITY
1: channel 1
2: channel 2, before the gradient changes
3: channel 3, near the flank of the spillway

DATA

NAME OF THE DAM	Wagendrift Dam
COMPLETION YEAR	1963
NAME OF THE RIVER	Bushmans
NEAREST TOWN	Estcourt
PROVINCE	Natal

SPILLWAY TYPE	Double arch wall, free falling jet	
HEIGHT OF THE CREST(m)	35,00	
LENGTH OF THE CREST(m)	120	
MAX.DISCHARGE CAPACITY(m^3/s)	1 300	
WATER FLOW DIRECTION(^{o})	062	
ROCK TYPE	Shale (right) and dolerite (right and left)	
DENSITY(Kgr/m^3)	2 500–2 600	2 900
SCHMIDT HAMMER HARDNESS	44,7	58,5
UN.COMPR.STRENGTH(MPa)	60–200	130–330
MODULUS OF ELASTICITY(GPa)	53–78	94–116

JOINT SURVEY LOCALITY	1	2	3
RQD(%)	35	55	30
No of JOINT SETS	5	4	6

JOINT SPACING(mm)	100	300	500	400	500	600	300	800	800	500	900	500	200	200	500
JOINT SEPARATION(mm)	5	34	4	30	2	50	1	1	7	30	25	7	1-5	1-5	20
J.DIP ANGLE(^{o})	05	83	80	85	78	76	89	90	08	75	88	85	72	85	05
J.DIP DIRECTION(^{o})	017	011	167	111	075	344	239	301	012	042	167	122	067	020	310
J.STRIKE(^{o})	107	101	011	201	165	074	329	031	101	132	257	212	157	110	040
GOUGE		soft, silty				hard	no	hard		no	hard, sandy				
ROUGHNESS	sr	s	s	sr	sr		rough					rough			
J.WATER CONDITION			moisture				no					minor inflow			
EROSION ESTIMATED(m)	1,50														

JOINT SURVEY LOCALITY
1: shale of the right apron
2: dolerite of the right apron
3: dolerite of the left apron

RESERVOIR RECORD – SPILLWAY DISCHARGE
RESERVOIR Wagendrift
STATION No V7R01
All figures are in $10^6 m^3$

MONTH	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974
Jan			25,3	51,4	41,2			22,3	23,1	27,1	6,1	78,8
Feb		0,0	20,4	53,7	164,0			32,5	21,1	40,9	15,1	93,0
Mar		1,6	2,9	4,1	42,0	8,0	40,1	10,5	12,7	69,8	18,9	76,0
Apr			2,0	0,0	53,7	3,3	39,9	2,8	9,9	15,5	20,9	33,2
May			0,7		7,6		11,1	1,4	3,1	5,6	6,1	10,3
Jun			4,6		1,0		6,0	1,0	1,0	1,9	1,8	3,8
Jul			2,1				4,2		0,6	0,4	0,1	1,2
Aug			1,5				0,8		2,3		1,8	
Sep			0,5					3,8	0,5		0,1	
Oct		16,0					10,2	13,6			10,8	
Nov		32,4	1,3	8,7			7,6	12,9	2,1		16,3	
Dec		31,2	3,7	18,7			32,4	6,9	16,5	1,2	15,0	

MONTH	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986
Jan	38,2	97,7	7,7	51,0	26,5	24,7	39,7	31,3		38,3		43,5
Feb	86,3	74,0	29,6	48,3	35,3	25,6	94,9			15,5	75,4	54,6
Mar	49,8	131,0	18,8	43,6	48,6	41,5						
Apr	17,1	26,7	12,5	31,5	9,7	11,9						
May	6,9	9,8	0,1	11,0	11,0	5,5				7,8		
Jun	3,0	1,4		5,5	4,8							
Jul		0,2										
Aug		0,0			10,0							
Sep					6,2							
Oct		12,8			9,7							
Nov	8,9	12,2		11,8								
Dec	57,2			68,1		31,4	13,1				42,7	39,2

MONTH	1987	1988
Jan	40,2	
Feb		
Mar	43,9	
Apr		
May		
Jun		
Jul		
Aug		
Sep	79,0	
Oct	51,4	
Nov		
Dec		

PEAK OUTFLOW DISCHARGE	687,1 cubic metres per second
MAXIMUM DAILY AVERAGE FLOW	359 cubic metres per second
DATE OF MAX DAILY AVERAGE FLOW	29 September 1987
DURATION OF THE FLOW	5 days

	DATA
NAME OF THE DAM	Craigie Burn Dam
COMPLETION YEAR	1963
NAME OF THE RIVER	Mnyamvubu
NEAREST TOWN	Greytown
PROVINCE	Natal
SPILLWAY TYPE	Arch wall, free falling jet
HEIGHT OF THE CREST(m)	35,3
LENGTH OF THE CREST(m)	121,9
MAX.DISCHARGE CAPACITY(m^3/s)	340
WATER FLOW DIRECTION(^{o})	117(left side)/077(centre)/027(right side)
ROCK TYPE	Dolerite
DENSITY(Kgr/m^3)	2 900-3 000
SCHMIDT HAMMER HARDNESS	
UN.COMPR.STRENGTH(MPa)	
MODULUS OF ELASTICITY(GPa)	

JOINT SURVEY LOCALITY			1			2	
RQD(%)			45			45	
No OF JOINT SETS			4			3	
JOINT SPACING(mm)	200	600	150	250		200	
JOINT SEPARATION(mm)	20	1	15	5		1-14	
J.DIP ANGLE(^{o})	85	88	89	22	85	80	10
J.DIP DIRECTION(^{o})	292	057	017	168	052	302	212
J.STRIKE(^{o})	022	147	107	258	142	012	302
GOUGE	sandy	no	sandy stain		soft, sandy or washed out		
ROUGHNESS				slightly rough			
J.WATER CONDITION				moisture only			
EROSION ESTIMATED(m)	1,00						

JOINT SURVEY LOCALITY
1: the left downstream area
2: the right downstream area

RESERVOIR RECORD - SPILLWAY DISCHARGE
RESERVOIR Craigie Burn
STATION No V2R01
All figures are in 10^6m^3

MONTH	1963	1964	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986
Jan			1,6	0,6				0,3	5,4	1,9		10,3	0,4	12,9	1,4	S	S		S				0,9	18,0
Feb		1,1	2,1	0,8	2,3			3,7	2,8	2,0		9,0	7,9	12,6	1,5	S	S		S				35,2	
Mar		0,0			0,3		1,1		0,8	5,3	0,8	2,9	0,8	11,4	3,3	S	S		S					
Apr		0,0			1,2	0,1	1,0		0,2	0,5	3,8	2,3		3,2	1,3	S	S							
May							0,3		1,2	0,0	1,6	0,1		0,2			S							
Jun		0,5	0,0				0,2				0,0	0,1				0,0	0,1							
Jul																	0,1							
Aug									2,6															
Sep			0,5						0,2															
Oct		0,5						5,9			0,2			0,7		S								
Nov		1,1						0,9	0,4		1,6			0,9	S	S								
Dec		0,8					2,9	1,3	3,3		3,2		4,7		S	S							9,1	1,1

MONTH	1987	1988
Jan	1,7	
Feb	11,1	4,4
Mar	7,2	19,0
Apr		
May		
Jun		
Jul		
Aug		
Sep	8,7	
Oct	11,7	
Nov		
Dec		

PEAK OUTFLOW DISCHARGE	366,5 cubic metres per second
MAXIMUM DAILY AVERAGE FLOW	169,0 cubic metres per second
DATE OF MAX DAILY AVERAGE FLOW	29 September 1987
DURATION OF THE FLOW	11 days

NAME OF THE DAM	Roodeplaat Dam
COMPLETION YEAR	1959
NAME OF THE RIVER	Pienaars
NEAREST TOWN	Pretoria
PROVINCE	Transvaal

SPILLWAY TYPE	Arch wall, free falling jet
HEIGHT OF THE CREST(m)	50,4
LENGTH OF THE CREST(m)	143
MAX.DISCHARGE CAPACITY(m^3/s)	970
WATER FLOW DIRECTION(^{o})	030/342

ROCK TYPE	intermediate to basic volcanic rocks
DENSITY(Kgr/m^3)	2 770
SCHMIDT HAMMER HARDNESS	58,25
UN.COMPR.STRENGTH(MPa)	130-310
MODULUS OF ELASTICITY(GPa)	83-110

JOINT SURVEY LOCALITY	1					2				
RQD(%)	30									
No OF JOINT SETS	5					5				
JOINT SPACING(mm)	300									
JOINT SEPARATION(mm)	<10									
J.DIP ANGLE(^{o})	90	60	72	12	55	90	85-55	25	55	72
J.DIP DIRECTION(^{o})	253	210	017	176	109	350	093	355	054	289
J.STRIKE(^{o})	343	300	107	266	199	080	183	085	144	019
GOUGE	weathered rock					no				
ROUGHNESS	slightly rough, rough									
J.WATER CONDITION	moisture only									
EROSION ESTIMATED(m)	0,30 (on the flanks), 1,5-2,3 (plunge pool depth)									

JOINT SURVEY LOCALITY

1: rock mass on the left side

2: rock mass on the right side

RESERVOIR RECORD - SPILLWAY DISCHARGE

RESERVOIR Roodeplaat

STATION No A2R09A

All figures are in $10^6 m^3$

MONTH	1970	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988	1989
Jan			8,9		4,4	30,8	11,9	1,4	68,4		13,1	3,8	4,2	2,3					1,3	
Feb		3,6	0,1		2,0	15,8	17,7	5,1	28,4	0,1	14,0	4,8	3,1							6,1
Mar		0,3	1,4		0,5	5,9	7,0	3,8	14,3	0,7	4,1	5,4						4,6	2,1	1,6
Apr		6,6	0,2		1,9	16,6	4,7	4,0	4,3		0,3	0,1						0,3	1,3	
May		0,3			0,1	4,7	3,4	1,7	1,5										0,0	
Jun		0,0				2,5	2,3	0,8	0,8											1,5
Jul		0,1				1,5	1,2	0,7	0,8											
Aug						0,8	0,9	0,1	1,0											
Sep						0,5	0,6	0,1	1,8											
Oct							4,4	0,5	1,0											
Nov		0,5				3,3	7,8	0,7	0,5	7,1	2,2									
Dec		7,2	0,2	2,9	2,3	4,5	5,0	2,4		3,6	5,2		0,9							

PEAK OUTFLOW DISCHARGE	961 cubic metres per second
MAXIMUM DAILY AVERAGE FLOW	207 cubic metres per second
DATE OF MAX DAILY AVERAGE FLOW	27 January 1978
DURATION OF THE FLOW	357 days

NAME OF THE DAM	Goedertrouw Dam
COMPLETION YEAR	1982
NAME OF THE RIVER	Mhaltuze
NEAREST TOWN	Eshowe
PROVINCE	Natal

SPILLWAY TYPE	Concrete wall with flip bucket
HEIGHT OF THE CREST(m)	10,5
LENGTH OF THE CREST(m)	160
HEIGHT OF FLIP BUCKET EXIT(m)	7,0
WIDTH OF FLIP BUCKET EXIT(m)	160
ANGLE OF FLIP BUCKET EXIT(^{o})	33
MAX.DISCHARGE CAPACITY(m^3/s)	7 000
WATER FLOW DIRECTION(^{o})	106

ROCK TYPE	weathered tillite
DENSITY(Kgr/m^3)	2 500-2 700
SCHMIDT HAMMER HARDNESS	55,6
UN.COMPR.STRENGTH(MPa)	100-270
MODULUS OF ELASTICITY(GPa)	64-100

RQD(%)		27	
No OF JOINT SETS		3	
JOINT SPACING(mm)	60	200	600
JOINT SEPARATION(mm)	<40	<30	<30
DIP ANGLE(^{o})	89	86	19
DIP DIRECTION(^{o})	263	356	077
STRIKE(^{o})	353	086	167
GOUGE	salty	salty	weathered rock
ROUGHNESS	smooth	smooth	slightly rough
WATER CONTENT		no	
EROSION ESTIMATED(m)	3,00		

RESERVOIR RECORD - SPILLWAY DISCHARGE
RESERVOIR Goedertrouw
STATION No W1R01
All figures are in $10^6 m^3$

MONTH	1979	1980	1981	1982	1983	1984	1985	1986	1987
Jan									
Feb									
Mar									
Apr									
May									
Jun									
Jul									
Aug									
Sep									73,9
Oct									
Nov									
Dec									

PEAK OUTFLOW DISCHARGE	589,5 cubic metres per second
MAXIMUM DAILY AVERAGE FLOW	
DATE OF MAX DAILY AVERAGE FLOW	28 September 1987
DURATION OF FLOW	4 days

DATA

NAME OF THE DAM	Vygeboom Dam
COMPLETION YEAR	1971
NAME OF THE RIVER	Komati
NEAREST TOWN	Badplaas
PROVINCE	Transvaal
SPILLWAY TYPE	Concrete lined chute with flip bucket
HEIGHT OF THE CREST(m)	42,3
LENGTH OF THE CREST(m)	
HEIGHT OF FLIP BUCKET EXIT(m)	2,7
WIDTH OF FLIP BUCKET EXIT(m)	41,5
ANGLE OF FLIP BUCKET EXIT(^{o})	40
MAX.DISCHARGE CAPACITY(m^3/s)	2 617
WATER FLOW DIRECTION (^{o})	209
ROCK TYPE	granite - granitic gneiss
DENSITY(Kgr/m^3)	2 700
SCHMIDT HAMMER HARDNESS	56,25
UN.COMPR.STRENGTH(MPa)	110-280
MODULUS OF ELASTICITY(GPa)	72-102

RQD(%)			55		
No OF JOINT SETS			5		
JOINT SPACING(mm)	200	100	200	300	40
JOINT SEPARATION(mm)	3	<2	<1	<1	<1
J.DIP ANGLE(^{o})	05-15	70	85	58	85
J.DIP DIRECTION(^{o})	352 238 116	130	185	214	077
J.STRIKE(^{o})	082 328 206	220	275	304	167
GOUGE		no			
ROUGHNESS	rough	rough	rough	rough	rough
J.WATER CONDITION		no			
EROSION ESTIMATED(m)	0,25				

RESERVOIR RECORD - SPILLWAY DISCHARGE

RESERVOIR Vygeboom

STATION No X1R03

All figures are in $10^6 m^3$

MONTH	YEAR																	
	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987	1988
Jan			20,6	66,2	30,9	59,0	10,7	54,3	7,8	27,1	16,4	39,4		38,3	9,3	13,3		38,1
Feb			34,1	69,3	58,4	52,9	19,3	27,2	5,9	39,9	43,6	11,5		39,7	31,8	24,3	13,2	14,7
Mar			27,2	33,7	29,7	35,7	14,6	26,2	4,1	26,8	39,1	5,4		18,9	20,2	8,8	24,2	17,9
Apr			42,4	31,5	25,1	32,9	13,4	13,6		14,9	16,1	2,5		18,9	5,6		7,5	7,8
May		17,1	23,1	25,5	14,3	22,7	5,3	7,9		2,2	9,4	0,7		5,7	5,4	2,2	2,4	2,8
Jun		11,4	13,3	19,6	9,9	13,3	3,5	5,3		3,9	4,4			3,9				1,0
Jul		7,5	10,5	17,3	7,7	11,7	1,7	6,3		1,9	2,8			5,8				2,2
Aug		10,3	7,5	12,6	6,6	9,2	1,4	4,3		0,8	0,9			0,6				
Sep		7,1	7,3	9,2	4,6	6,6	4,3			0,0	1,8			0,0				
Oct		9,3	25,2	4,2	3,5	6,3					0,1			2,5			20,8	12,7
Nov		18,8	29,8	1,0	10,3	13,3		0,9			2,2			8,3			56,7	5,5
Dec		20,1	59,4	16,0	28,6	14,8	16,7	4,0	1,6		10,8			12,4			52,7	

PEAK OUTFLOW DISCHARGE	129 cubic metres per second
MAXIMUM DAILY AVERAGE FLOW	70,9 cubic metres per second
DATE OF MAX DAILY AVERAGE FLOW	8 February 1974
DURATION OF THE FLOW	110 days

DATA

NAME OF THE DAM	Gamkapoort Dam
COMPLETION YEAR	1969
NAME OF THE RIVER	Gamka
NEAREST TOWN	Prince Albert
PROVINCE	Cape

SPILLWAY TYPE	Gated with double curvature flip bucket
HEIGHT OF THE CREST(m)	37,5
LENGTH OF THE CREST(m)	108,9
HEIGHT OF FLIP BUCKET EXIT(m)	2,7
ANGLE OF FLIP BUCKET EXIT(^{o})	
MAX.DISCHARGE CAPACITY(m^3/s)	4 531
WATER FLOW DIRECTION(^{o})	142/102

ROCK TYPE	quartzitic sandstone
DENSITY(Kgr/m^3)	2 600
SCHMIDT HAMMER HARDNESS	58,5
UN.COMPR.STRENGTH(MPa)	110-285
MODULUS OF ELASTICITY(GPa)	76-104

JOINT SURVEY LOCALITY	1	2
RQD(%)	0-5	20
No of JOINT SETS	3(+3secondary)	3(+1secondary)
JOINT SPACING(mm)	10 200	200
JOINT SEPARATION(mm)	1-5	1-5
J.DIP ANGLE(^{o})	26 64 85 60 53 80	30 50 80 80
J.DIP DIRECTION(^{o})	236 158 057 193 233 106	332 152 056 146
J.STRIKE(^{o})	326 248 147 283 323 196	062 242 146 236
GOUGE	yes no no no no no	yes no no no
ROUGHNESS	slightly rough, rough	slightly rough, rough
J.WATER CONDITION	no	no
EROSION ESTIMATED(m)	2,00	

JOINT SURVEY LOCALITY
1: downstream of the bottom flip bucket
2: further to the left of the first point,downstream of the bottom flip bucket

RESERVOIR RECORD - SPILLWAY DISCHARGE
RESERVOIR Gamkapoort
STATION No J2S06(since 1971),J2R06A(since 1982),J2R006-A01(since 1988)
All figures are in $10^6 m^3$

	YEAR																
MONTH	1971	1972	1973	1974	1975	1976	1977	1978	1979	1980	1981	1982	1983	1984	1985	1986	1987
Jan				2,1		2,3									3,7		0,0
Feb				87,8		57,5											
Mar				21,6		109,0											
Apr				6,4													
May						4,5		7,2									
Jun						4,5		0,2									
Jul						0,0											
Aug	133,0							0,1									
Sep	8,1																
Oct																	
Nov																3,2	
Dec																46,5	

PEAK OUTFLOW DISCHARGE	499 cubic metres per second
MAXIMUM DAILY AVERAGE FLOW	374 cubic metres per second
DATE OF MAX DAILY AVERAGE FLOW	5 February 1976
DURATION	6 days