PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS

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PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS: EXECUTIVE SUMMARY

SCOPE OF THIS REPORT

The Water Research Commission (WRC) commissioned the CSIR in 1992 to conduct an investigation into plunge pool scour reproduction in physical hydraulic models. A steering committee was appointed to guide the project and review the results.

The steering committee chaired by Mr D S van der Merwe of the WRC included other members Mr H C Chapman (WRC), Mr F P Marais (WRC), Mr D B Badenhorst (BKS Inc), Mr K R K Blake (formerly of CSIR, Environmentek), Dr J M Jordaan (Dept. of Water Affairs and Forestry), Prof A Rooseboom (University of Stellenbosch), Dr A Schall (Dept. of Water Affairs and Forestry), Prof A van Schalkwyk (University of Pretoria), Dr D H Swart (formerly of CSIR, Environmentek), Mr A H van Tonder (CSIR, Aerotek) and Mr J A Zwamborn (formerly of CSIR).

The study consisted of three phases:

- Phase 1 - Initial investigations
- Phase 2 - Extended tests using a weak cement/sand mix, plus a repeat of coke nut tests
- Phase 3 - Aggregate method

This executive summary report was compiled by Mr. W J van Aswegen. The report on Phase1 was compiled by Mr K R K Blake, Phase 2 by Ms E Dunkley and Phase 3 by Mr W J van Aswegen all of the CSIR.

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1. INTRODUCTION

Following modelling work carried out in the late 1980's both in South Africa and Australia, it became evident that the ability to accurately predict the size of scour holes forming downstream of dam overflow spillways and flip buckets was lacking. In 1991 the CSIR carried out a literature survey on this topic and came to the conclusion that work needed to be done on the physical modelling aspects of plunge pool scour reproduction. The CSIR then approached the Water Research Commission (WRC) for guidance and funding. A contract was signed with the WRC and work on Phase 1 of the project commenced in 1992. The aim of the study was to find the most appropriate model material for plunge pool scour reproduction.

Phase 1 of the project was successfully completed in September 1995.The report on the results **CSIR Report EMAS-C 95098,** was compiled by Mr K R K Blake. After the resignation of Mr K R K Blake, the project leadership was taken over by Ms E Dunkley.

Phase 2 of the project evaluated the direction set in Phase 1. A model material consisting out of cement based binder with coke nuts (degassed anthracite) as the aggregate was recommended in Phase 1 as best material, although the results were not entirely satisfactory. The temporary unavailability of the degassed anthracite resulted in a series of tests performed with a model material consisting out of cement based binder and sand. Further tests with degassed anthracite were performed once the material was available. Phase 2 of the project was concluded in October 1998. **CSIR Report ENV/S-C 98096** describes the tests conducted and the results obtained in this second phase of the study.

Phase 1 and Phase 2 focused on cemented, lightweight materials, such as coke nuts. However the repeatability of the tests when using such material was not very good. The binding strength of the cement and lime, once mixed with water, could not be accurately controlled.

The method developed during the Maguga dam physical model study to reproduce the physical characteristics of a plunge pool scour hole was subsequently proposed by the steering committee. This was applied in the Kariba dam model as **Phase 3** of the project, and was concluded in March 2000.

Phase 3 of the project concluded the current investigations into the most appropriate material to be used to reproduce the plunge pool scour below dam outlet structures in scaled physical hydraulic models. The work was carried out under the project leadership of Mr W J van Aswegen. **CSIR Report ENV-S-C 2000-038** described the tests conducted and the results obtained for this third phase of the study.

The executive summaries of Phase 1, Phase 2 and Phase 3 are included in this combined executive summary report.

PHASE 1
PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS INITIAL INVESTIGATIONS

CSIR REPORT NUMBER: EMAS-C 95098

Free trajectory jets are often used as a means to pass excess flood waters from a dam to the river below. This inevitably results in the formation of a scour hole downstream of the dam wall. If this scour hole is extensive, as is often the case, it may endanger the dam's ancillary structures and even the stability of the dam wall itself. It is, therefore, imperative that the ultimate size of the scour hole is known at the design stage of a dam.

The mechanism of scour, however, is an extremely complex process due to the influence of various hydraulic, hydrological and geological factors. Some of these factors are difficult to assess theoretically and limited success has been had when trying to reproduce them experimentally. It is not surprising, therefore, that a reliable method of predicting the size and depth of scour holes is still not available. Although some empirical formulae have been proposed to predict such scour holes, in most cases, scale physical models are constructed to try to reproduce what will ultimately happen in the prototype. This research project set out to investigate the most appropriate materials to be used to model the bed rock below a dam wall, for use in such physical models. At the same time it also looked at the reliability of the available empirical formulae proposed to date.

Nineteen scour prediction formulae were used to predict the scour depths at various known sites and the results compared with the actual scour data. Analysis of these results showed that while certain formulae worked with some degree of success at certain given sites, problems always arose when using the formulae at other sites. It would appear that the proposed formulae do not take into account all the factors influencing the formation of the hole, or that the data used to derive the formulae did not represent the ultimate scour depth. In hindsight, it would seem that a reasonably accurate generic scour prediction formula is not possible. The engineer is thus forced to turn to scale modelling to obtain a more accurate prediction of the depth of the scour hole.

To test the proposed model bed materials, an existing dam had to be modelled in the laboratory, such that its scour hole could be used as a criterion for the tests. However, due to the lack of appreciable depth of scour below suitable dams within South Africa, no sites could be found for this purpose. Therefore, the search had to be extended further north and the Kariba Dam on the Zambezi River was chosen. The scour hole below this dam has extended almost 75 m below the river bed and annual surveys have been kept of the progress of the hole since the dam's construction.

During the extensive model testing phase of this study, a number of aspects were highlighted concerning the accurate reproduction of scour holes. The most important of these was the sensitivity of the formation of the scour hole to the flow patterns in, and around, the eroding hole. The final scour depth was found to vary greatly if the local topography was not exactly reproduced in the model, or if the material being eroded from the hole was not ejected as it would be in reality. Coupled to the latter was the effect of the confinement of the developing hole on the plunging jet. This confinement tended to enhance the erosive power of the jet by concentrating its energy onto a smaller area, resulting in a much deeper scour hole. Thus, the use of cohesive materials to represent the bed rock in the model, which could sustain the steep sides expected in reality, gave deeper, more realistic scour holes. Testing with non-cohesive materials lead to shallower scour holes that, if taken as correct, may result in an under designed system.

This study investigated a wide range of possible materials to represent the bed material in scale models. The criterion against which the materials were tested was their ability to reproduce the scour hole formed at the Kariba Dam. It was found that the loose material collecting in the scour hole adversely affected the ultimate depth of the hole. In reality this material would erode into pieces small enough to be ejected from the hole and transported downstream. To resolve the problem, a very light aggregate (coke nuts) was used, that could easily be ejected from the scour hole once it was freed from the binding matrix. Thus, the scour hole maintained a shape similar to that expected in prototype, thereby ensuring that the flow patterns in and around the hole were correct.

None of the materials tested gave ideal results, however, the one material that did behave remarkably better than all the others consisted of cement based binder with coke nuts (degassed anthracite) as the aggregate, in the ratio (by volume);

LIME	CEMENT	COKE NUTS	WATER
1	0,04	7,2	0,26

Due to financial limitations, further investigations into this material could not be carried out. It was, therefore, recommended that this be taken up at a later stage to further develop the ability to accurately predict the depth of scour holes below dams.

PHASE 2
PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS: EXTENDED TESTS USING AND WEAK CEMENT/SAND MIX, PLUS A REPEAT OF COKE NUT TESTS

CSIR REPORT NUMBER: ENV/S-C 98096

The information gathered in this report is a continuation of the work carried out by Mr K R K Blake (CSIR, Environmentek) concerning physical modelling aspects of plunge pool scour reproduction. The aim was to find a mix of material to reproduce the depth and shape of the scour hole below the Kariba dam.

The approach was to pursue the direction set in Phase 1 by Mr Blake and to carry out further investigations into this material by performing additional tests. However, the coke nuts became temporarily unavailable. It was thus decided to test another mix consisting of a cement-based binder with sand. Four tests were carried out, using different proportions of cement: sand: water. Tests with coke nuts, in the ratio recommended in Phase 1, were also performed once the material became available.

Only the mix tested in Test 4, the proportion of which is :

SAND	CEMENT	WATER
1	0,021	0,124

seemed to give a good enough reproduction of the shape of the scour hole formed below the Kariba dam. The depth of scour was however still insufficient, indicating that weaker sand: cement mix should be tested.

Once coke nuts became available the recommendations in Phase 1 to continue these tests was carried out. The results, however, proved to be different from those obtained by Mr Blake, due to the inconsistency of the strength of the cement used as the binder element.

PHASE 3
PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS: LOOSE AGGREGATE METHOD

CSIR REPORT NUMBER: EMAS-C 2000-038

This report is a continuation of research by the CSIR, which started in 1992. Upon recommendation of the Steering Committee for this project, the WCR extended the budget in May 1999 for additional model tests, based on a method developed during a physical model study of the Maguga Dam, carried out at the University of Stellenbosch.

The method developed during the Maguga Dam physical model study, to reproduce the physical characteristics of a plunge pool scour hole, was applied in the Kariba Dam model. This method involved the use of loose aggregate to model the scour hole. A total of seven tests were conducted in the model. Besides the scour prediction, the tests had as additional objectives:

- to demonstrate repeatability,
- to optimise aggregate size,
- to evaluate the relative density of the test materials,
- to evaluate the effect of flood conditions.

Test results show an underestimate of between 15% and 23% of the maximum Kariba scour depth. The natural angle of repose for all materials tested was the main characteristic influencing the model test results. If all other material characteristics could stay the same and the angle of repose would change to a value closer to that found in prototype, the correct scour depth and profile might be achieved.

The loose aggregate method of scour prediction has produced the best results to date in the Kariba model. This method, if applied with the necessary expertise and knowledge of how a physical hydraulic model reacts during the scour process, can produce valuable results.

RECOMMENDATIONS

Future research should focus on the development of the binder element used in the model material. The binder element must be able to produce consistent and controllable strengths over a short period of time. A binder element other than normal cement and lime must be considered.

A model material with a gel or resin binder element can be developed in collaboration with the University of Stellenboch as a joint research project.
This is an ideal opportunity as a master's project.

The most current Kariba dam scour information and geotechnical information must be obtained if research is to continue. The Council for Geoscience is currently conducting a comprehensive survey of the scour hole below the Kariba dam wall. The council's study will contain valuable information linking geotechnical aspects and the current scour hole dimensions.

PHASE 1

INITIAL INVESTIGATIONS

by

KRK Blake

Division of Water, Environment and Forestry Technology, CSIR

PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS

SCOPE

Following modelling work carried out in the late 1980's both in South Africa and Australia, it became evident that the ability to accurately predict the size of scour holes forming downstream of dam overflow spillways and flip buckets was lacking. In 1991 the CSIR carried out a literature survey on this topic and came to the conclusion that work was needed to be done on the physical modelling aspects of plunge pool scour reproduction. The CSIR then approached the Water Research Commission (WRC) for guidance and funding. A contract was signed with the WRC and the three-year project commenced in 1992. The results of the study are given in this report.

The project was guided by a steering committee, chaired by Mr D S van der Merwe of the WRC. Other members of the committee were Mr H C Chapman (WRC), Mr F P Marais (WRC), Mr D B Badenhorst (BKS Inc), Mr K R K Blake (formerly of CSIR, Environmentek), Dr J M Jordaan (Dept. of Water Affairs and Forestry), Prof A Rooseboom (University of Stellenbosch), Dr A Schall (Dept. of Water Affairs and Forestry), Prof A van Schalkwyk (University of Pretoria), Dr D H Swart (formerly of CSIR, Environmentek), Mr A H van Tonder (CSIR, Aerotek) and Mr J A Zwamborn (formerly of CSIR).

The work was carried out under the project leadership of Mr K R K Blake with the guidance of, initially, Mr J A Zwamborn and later Mr K S Russell. Mr F W van Dulm was responsible for the model construction and Mr S J van Wyk carried out the majority of the model testing in the laboratory. The basic manuscript of this report was compiled by Mr Blake. After the resignation of Mr Blake, the project leadership was taken over by Ms E Dunkley.

K S Russell
PROJECTS MANAGER

Stellenbosch, South Africa
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NOMENCLATURE

a	dimension of a side of a cubic block	(m)
A	maximum pulsation amplitude of forces within the rock matrix	(m)
b	jet or flow width	(m)
B	maximum tearing load exerted on the bed rock	(m)
C_x	head loss coefficient at section x	(-)
d	representative rock size	(m)
D	scour depth measured from the deepest point to the tailwater level	(m)
F_r	Froude number	(-)
g	acceleration due to gravity	(m/s^2)
h	tailwater depth above the original bed level	(m)
H	fall height, measured between upstream and downstream water levels	(m)
Ja	joint alteration number	(-)
Jn	joint set number	(-)
Jr	joint roughness number	(-)
Js	relative ground structure number	(-)
K	empirical coefficient	(-)
K_n	Kirsten number	(-)
L	length	(m)
M_s	mass strength number	(-)
n_d	jet nozzle diameter	(m)
N	Martins' factor	(-)
p	jet perimeter at the level of tailwater	(m)
P	unit stream power	(W/m^2)
q	flow per unit width	($m^3/s/m$)
Q	total flow	(m^3/s)
r	jet radius	(m)
R	hydraulic radius	(m)
R_e	Reynolds' number	(-)
RQD	rock quality designation	(%)
s	mean surface disturbance thickness of the jet	(m)
t	jet thickness at impact with the tailwater	(m)
T	time averaged tearing load exerted on the bed rock	(m)
v	average flow velocity	(m/s)
v_x	average flow velocity at section x	(m/s)
v_*	jet velocity at impact with the tailwater	(m/s)
V_e	minimum velocity of jet to entrain air	(m/s)
W_b	Weber number	(-)
β	ratio of air to water by volume	(-)
λ	friction coefficient	(-)
θ	jet impingement angle	(degrees)
κ	empirical coefficient	(-)
ρ	fluid density	(kg/m^3)

1. INTRODUCTION

1.1 Project Objectives

The aim of this research project is to develop and test physical model materials or techniques that would more accurately reproduce scour in plunge pools.

The results of this research will hopefully improve the ability of design engineers to determine the expected rates, depths and extent of scour in plunge pools below dam spillways. From the rate and extent of scour, the need (or not) for additional protective measures can be determined, which should lead to a safer and more economic design practice.

In addition, the research could be applied to existing dams where updated safety requirements may dictate additional protection works. Site-specific model tests could again be employed to determine the available safety margin and therefore the need for additional protection works.

1.2 Scour Below Dams

Free trajectory jets are often used as a means to pass excess flood waters from a dam to the river below. Three methods are commonly used; an overflow spillway situated on the crest of the dam, gates situated at some point in the dam wall itself and chute spillways with flip-buckets. All three rely on the dissipation of energy by the subsequent aeration and break up of the jet as it travels through the air. Besides this, aeration may also be increased by careful design of the spillway structure itself. The amount of energy left at the end of the fall determines the scour *potential* of the jet. The actual resultant scour depends on this, as well as numerous additional factors to do with the bed and site conditions. Figure 1 shows the definitions and symbols used in this report.

When the jet impacts on the river bed downstream of the dam, its energy is initially partly dissipated by excavating a scour hole. The accurate prediction of the depth and extent of this scour hole is essential to ensure the stability of the dam wall as well as the side slopes of the downstream channel. Scour prediction formulae, scale models and the interpretation of the results, have been shown to be misleading on occasion. This problem was highlighted by the Kariba Dam (Zambia/Zimbabwe). Analysis of model tests carried out during the design stage prior to the construction of the dam gave an estimated final scour depth of 30 m (Häusler 1983). However, after prolonged usage of the gates in the first five years of operation (far more than the expected usage) the scour hole reached a depth in excess of 50 m. Further model studies were conducted in 1971 (ICOLD 1993), from which the estimated ultimate scour depth was increased to 60 m. By 1979 however, after 17 years of operation, the scour hole had extended to 71 m below the original bed level. Furthermore, it is thought that Kariba may still not be at its ultimate scour depth (Häusler 1983). Ironically, the 1961 model tests, using a 1:150 scale model and a cohesive bed, indicated an ultimate scour depth of 76 m below the bed level (SOGREAH 1962). The results of these tests are shown in Figure 2, with the actual 1981 scour hole superimposed on the figure. However, at the time this scour was considered unlikely to occur in the sound gneiss bed rock of the prototype and the conceptual design was maintained (ICOLD 93).

The problem of accurately predicting scour depths was again raised during model tests for the proposed Katse Dam (Lesotho). In this case two scale models were operated, a 1:30 scale sectional model at SOGREAH in France and a comprehensive 1:70 scale three-dimensional model at the CSIR in South Africa, both of which used loose gravel to represent the bed rock. Although at the time this was thought to be acceptable, the question of how realistic this approach was, could not be adequately answered.

1.3 The Mechanism Of Scour

The development of a scour hole is an extremely complex problem, due to the influence of various hydraulic, hydrological and geological factors. These include the break up of the jet both in the air and in the water, the subsequent behaviour of the jet within the confined space of the scour hole, and the behaviour of the rock mass under the dynamic action of the jet. While certain of these factors are straight forward, others are difficult to assess theoretically and limited success has been had when trying to reproduce them experimentally.

Looking at the path of a free trajectory jet, it is first passed either over a crest, through a gate or down a chute and through a flip-bucket or ski-jump. Any of these structures could be, and are normally, fitted with some form of aeration device designed to cause a certain degree of break up of the jet as it leaves the structure. The jet then normally travels through the air for a distance before entering the plunge pool, where the jet will continue to break up. The degree of break up has been shown to depend on factors such as the fall height, degree of initial turbulence within the jet and the shape of the jet, among others. Various studies have been done on the minimum fall height required for an initially solid jet to break up completely (Ervine *et al* 1980). However, the case of a solid jet exiting the spillway structure is extremely rare and generally the fall height would not be nearly great enough for complete break up, even with the added aeration from the spillway. In fact it would appear that in most cases an unbroken water core remains after the fall. Häusler (1983) goes as far as suggesting that, because of this, the effect of aeration should be ignored or at most considered by a careful reduction of the solid jet width at impact. Although this may seem to err on the side of caution, case studies have shown that with the present accuracy achievable in scour prediction, it may be the better course.

On entering the plunge pool, the jet continues to break up by diffusion until it impinges on the river bed. How much of the initial core remains at this stage will, obviously, be a factor of the depth of the water the jet travels through, which is the depth of the water cushion. The presence of the bed causes the jet to deflect upstream and downstream of the impingement area and dynamic pressures build up on the surface of the bed rock. As early as 1963, tests by Yuditskii (and later others) have shown that these surface pressures are transmitted through vertical joints to the underside of the rock blocks. In certain cases, due to the stochastic nature of these forces, the time averaged pressures beneath the rock blocks resulted in an uplift force that exceeded the total downward forces on the blocks. These results were confirmed in work carried out by Hartung and Häusler (1973).

The differences in phase between the turbulent hydraulic pressure fluctuations that act on the bedrock's surface and those in the bedrock itself cause differential pressures to build up within the fractured rock, exerting forces on individual blocks. These forces loosen the blocks, cause them to wear within their matrix and finally project them into the flow. When the combined effects of the forces normal to the loosened block and the tangential drag forces from the local boundary flows become sufficiently large, the block becomes dislodged. Thus the block could be physically lifted upwards and, under the right conditions, entrapped by the highly turbulent flow and carried away.

The rate of build up of the hydraulic forces within the rock mass, and the effectiveness of the subsequent lifting of these blocks, is dependant on the condition and orientations of the rock mass discontinuities. Thus, each rock mass is likely to behave differently. In particular, rocks with open joints or joints with soft infill will be penetrated more quickly than intact rock with tight joints. When the orientation of the bedding planes of the rock are roughly perpendicular to the impacting jet, the sudden removal of overlying rock strata may cause the adjacent underlying rocks to exfoliate due to their internal stresses, as was reported at the Burdekin Falls Dam in Australia (Otto 1989). In such cases the rate of erosion would be greatly accelerated.

As more bed material is eroded, the live scour hole forms. The dislodged blocks recirculate within the live scour hole and abrade until they are small enough to be ejected as part of the downstream wash load. This process continues until the live scour hole deepens to the point where so much energy is lost by the plunging jet to its environment through turbulent diffusion that no further erosion occurs and a state of equilibrium is reached.

The scour mechanism of progressive pressure build up and block displacement implies that modelling the plunge pool bed with loose gravel (to represent the rock mass after it has broken up) may lead to an underestimation of the final scour hole depth. This is apart from the fact that the loose gravel will form equilibrium slopes on the sides of the scour hole that are unrelated to the properties of the rock mass it represents, distorting the flow patterns and possibly the results. The problem was investigated by Gerodetti (1982) who conducted two separate model tests using gravel with and without a cohesive binder to represent the bed material. He found that the scour depths were approximately 20 % and 30 % deeper with the cohesive binder for two different models.

2. SCOUR PREDICTION FORMULAE

2.1 Formulae Proposed In Literature

Numerous equations have been proposed to predict the scour resulting from free trajectory jets. The few that are detailed here are the ones that were found to be of particular interest in this study. For a complete description of the formulae proposed to date, the reader is referred to references such as Whittaker and Schleiss (1984), Mason and Arumugam (1985) or Spurr (1985).

One of the earliest formulae put forward (and still frequently quoted today) is attributed to Veronese (1937). He postulated that the scour depth D below the tailwater level (Figure 1), caused by a flow per unit width q, falling through a height H, could be expressed in the form:

$$D = 1{,}9\ q^{0{,}54}\ H^{0{,}225} \qquad \text{.......1}$$

Although widely used, the formula does not take into account the effects of tailwater depth h, the degree of air entrainment β or the characteristics of the bed material. Comparisons with later work show that for a small range of fall heights the power of H may, in effect, account for some of these factors. This limits the range of validity of the formula considerably and restricts its use to free fall jets with no artificial aeration (not a very common situation).

Martins (1973a) tested the scour in a model due to a plunging jet using bed material comprising of equal, cubic, comparatively large blocks (of edge length a), systematically arranged, without cohesion. From ninety model tests the author derived a formula to predict the scour depth;

$$D = 0{,}14\ N + 1{,}7\ h - \frac{0{,}73\ h^2}{N} \qquad \text{.......2}$$

where

$$N = \sqrt[7]{\frac{Q^3\ H^{1{,}5}}{a^2}}$$

By the nature of the equation, it can be shown that the maximum scour depth will be attained at a given tailwater depth, namely;

$$h = 0{,}48\ N \qquad \text{......3}$$

Note that this figure is misquoted in the paper, Martins 1973b. The fact that a non-zero tailwater depth gives the maximum scour depth has been found to be the case by other authors as well, for example Johnson (1967). This is one of the few formulae where the total flow Q is used instead of the more usual flow per unit length q. When tested by Martins against two prototype cases, the Picote Dam (Portugal) and the Kondopoga Dam (USSR), the formula gave predictive scour depths to within 15 %.

Otto (1989) reporting on Martins' work noted that although Martins had not accounted for the impingement angle θ, there did appear to be a trend in the data. Notably, with an impingement angle of 40° scour depths were consistently deeper than when the impingement angle was 70°. This is in contrast to the work of Chee and Kung (1971) (see below) where higher impingement angles resulted in greater ultimate scour depths.

Of particular interest to this project is the work carried out by Chee and Kung (1971). They derived a scour prediction formula from thirty-three model tests where the bed material in the model was represented solely by loose, non-cohesive granular material.

$$D \;=\; 3{,}30\; H \left[\frac{q^2}{g\; H^3}\right]^{0{,}3} \left[\frac{H\;\theta}{d_m}\right]^{0{,}1} \qquad \text{.......4}$$

Using the non-cohesive material in the plunge pool was justified by the authors with the argument that this is the *normal* method of simulating the downstream river bed to arrive at the most critical scour. This was based on the hypothesis that more energy is available in the model, as the jet does not have to fracture and loosen the in-situ bed rock, and therefore the scour would be more extreme. Work by the writer and others have shown this to be incorrect and that the most critical scour is not reached with a non-cohesive bed material.

Chee and Kung, recognised some short falls in their method and postulated that their formula only gave the stable depth and shape of the scour hole, *if it were excavated prior to any spillage from the structure*. In this respect, their formula may arguably be quite accurate, but of little use to the designer. Comparisons with prototype data revealed that the maximum depth of scour did exceed that extrapolated from the model tests. The authors surmised that the flow patterns within the developing scour hole would be different from those of a loose bed model (with its associated shallower sloped sides). It was thought (and based on the model test results of this study, the writer supports this hypothesis) that the focusing of the jet on entry to the confined scour hole leads to a greater energy dissipation at the bottom of the hole and therefore deeper scour.

In recent years, efforts have been made to rationalise scour formulae, notably Mason and Arumugam (1985), Whittaker and Schleiss (1984) and Spurr (1985). From these studies it was evident that the numerous scour formulae proposed could be categorised into groups based, primarily, on the parameters used in each formula. The largest group, (seventeen formulae) are of the form (where K is an empirical coefficient and d a representative rock size of the material in the plunge pool);

$$D = K \frac{q^{x} H^{y}}{d^{z}} \qquad ....5$$

Mason (1985) took seventy-three sets of known scour data (twenty-six from prototypes and forty-seven from model studies) and inserted each of them into these formulae in turn to check each formula's reliability. It should be noted that in all cases the prototype bed material was assumed to have a mean diameter d_m = 0,25 m and a d_{90} = 0,30 m, due to insufficient information. The formula that gave the best results for the model data was one proposed by SOFRELEC (1980), where;

$$D = 2{,}30\ q^{0{,}60}\ H^{0{,}10} \qquad ....6$$

Mason found that the formulae that gave the best results for model studies were not the same ones that gave the best results for the prototype cases. It would appear, therefore that some factor was missing from the formula that was different in the model and prototype cases. Mason did not state this conclusion, but did develop a new formula that also incorporated the tailwater depth h. He had to include the acceleration due to gravity g to ensure that a dimensional balance could be achieved, giving the form of the expression;

$$D = K \frac{q^{x} H^{y} h^{w}}{g^{v} d_m^{z}} \qquad ....7$$

This basic formula was used to process the same set of model data as before and the appropriate values of the coefficients x, y, w and z were obtained by varying each in turn ,to minimise the coefficient of variation of the results. The value of v was calculated to give an overall dimensional balance, which yielded the equation for model data as;

$$D = 3{,}27 \frac{q^{0{,}60} H^{0{,}05} h^{0{,}15}}{g^{0{,}30} d_m^{0{,}10}} \qquad ....8$$

As with previous formulae however, this formula gave poor results when checked against prototype data. Mason then attempted to satisfy both sets of data by varying the exponents x and y as well as the proportionality constant K, but this expression still overestimated scour depths in some cases by up to 100 % and in other cases, underestimated depths by up to 50 %. The difference in accuracy of the formula between model and prototype data suggests that there is still **at least** one factor missing from the analysis.

A point that has not been addressed in the preceding formulae derivations is that none of the prototype data can be said (with any certainty) to be at its ultimate scour depth. Thus, any formula derived from such data must include the time of formation of the scour hole if it is to have any chance of correctly reproducing scour depths. This may be one of the reasons why Mason's general formula, derived from both model and prototype scour data (and does not take the time of formation into account) gives poorer results than the formula derived from model data alone.

2.2 The Effects Of Aeration

In the case of free trajectory jets falling into plunge pools, the break up of the jet during the fall will entrain a certain volume of air while the broken surface of the plunge pool will trap yet more. Obviously, the amount of air entrained will greatly effect the scour potential of the jet striking the bed rock. This has been demonstrated experimentally by, among others, Johnson (1967). Apart from the reduction in the size of the solid core of the jet, the aeration itself increases the dissipation of potential energy of the jet as it passes through the tailwater cushion.

Various attempts have been made at predicting the volume of air entrainment (air to water ratio β). As early as 1954, Oyama *et al* presented a formula to predict the volume of air entrained by a circular jet (with nozzle diameter n_d and fall height H) plunging into a liquid, based on the Froude, Weber (W_b) and Reynolds' (R_e) numbers.

$$\beta = 0,75\ R_e^{1,33} \left[\frac{\sqrt{W_b}}{R_e}\right]^{2,18} \left[\frac{v^2}{g\ H}\right]^{-0,447} \left[\frac{H}{n_d}\right]^{0,281} \qquad ....9$$

Several authors have worked on the subject since, most of whom have concentrated on circular jets, for example Lin and Donnelly (1966) and Henderson *et al* (1970). Particular attention was paid to the break up length of the jet, this being a major factor in the amount of aeration entering the plunge pool.

Baron (1971) found that the break up length of a circular jet could be described as a function of the Weber and Reynolds numbers;

$$\textit{Break up length} \quad = \quad n_d \frac{1{,}7\ W_b}{(\ 10^{-4}\ R_e\ )^{0{,}625}} \qquad \text{......10}$$

Other authors have come up with similar variations of this formula for slightly differing circumstances. Van de Sande and Smith (1973), for instance, noted that there was a definite difference in the entrainment characteristics between what they termed low and high velocity jets (the cutoff being at about 4,5 m/s). The significance of this is apparent when attempting to reproduce air entrainment in scale models. While the fall velocity of the prototype will almost always be well above 4,5 m/s, the modeller must take care to ensure that the model is large enough for the same to be true in the laboratory.

Horeni (1956) was one of the first authors to look at rectangular jets. He obtained the expression for the break up length of a rectangular jet being discharged horizontally as;

$$\textit{Break up length} \quad = \quad 7{,}8\ (\ R_e\ )^{0{,}319} \qquad \text{......11}$$

Ervine (1976) addressed the problem of rectangular jets in more detail and derived the expression to estimate air entrainment;

$$\beta \quad = \quad 0{,}26 \left(\frac{b}{p} \right) \left(\frac{H}{t} \right)^{0{,}446} \left(1 - \frac{V_e}{v_*} \right) \qquad \text{.....12}$$

where β	=	ratio of air to water by volume	(-)
b	=	jet width	(m)
p	=	jet perimeter	(m)
H	=	fall height	(m)
t	=	jet thickness at impact	(m)
V_e	=	minimum velocity of jet to entrain air	(m/s)
v_*	=	jet velocity at impact $\approx \sqrt{2gH}$ (for small H)	(m/s)

For a wide jet, the jet perimeter can be approximated as twice the jet width and therefore the above formula can be simplified to;

$$\beta \quad = \quad 0{,}13 \left(\frac{H}{t} \right)^{0{,}446} \left(1 - \frac{V_e}{v_*} \right) \qquad \text{.....13}$$

Ervine *et al* (1980) took the matter further by considering the turbulence intensity as a function of the undulating surface of the jet. This work, however, was restricted to circular jets. The authors concluded that the air entrainment of a circular jet falling vertically into a confined pool cannot be described solely by the Froude, Weber and Reynolds' numbers. Internal turbulence plays a major role in the degree of air entrainment, jet surface roughness and break up length of the jet. They found that for a jet of radius r and a mean surface disturbance thickness s, the air entrainment can be found from;

$$\beta = 14{,}0\left[\left(\frac{s}{r}\right)^2 + 2{,}0\left(\frac{s}{r}\right) - 0{,}1\right]^{0{,}6} \qquad .....14$$

This formula is stated as only valid for:

- Jet velocities up to 6 m/s
- Nozzle diameters from 6 to 25 mm
- Turbulence intensities of 0,3 to 8 %
- Fall heights less than the break up length of the jet.

and therefore, the formula is of limited practical use.

Ragola (1981) carried out work with two dimensional rectangular jets and obtained the expression for air entrainment;

$$\beta = 0{,}78 * 10^{-9} F_r^{1{,}51} R_e^{1{,}1} \qquad .....15$$

In this case, where as the Froude number is based on the jet thickness at impact, the Reynolds number is based on the energy head over the weir, a factor that could vary according to the crest discharge coefficient.

Mason (1987), having reviewed the available research into aeration and following discussions with Ervine, suggested that the air entrainment of plunging jets could be generally and approximately assumed to take the form;

$$\beta = \kappa\left(\frac{H}{2r}\right)^{0{,}5} \quad \text{for circular jets} \qquad .....16$$

$$\beta = \kappa\left(\frac{H}{t}\right)^{0{,}5} \quad \text{for rectangular jets} \qquad .....17$$

Where the value of κ is dependant on the turbulence intensity and varied from 0,2 to 0,4 for circular jets and 0,1 to 0,2 for rectangular jets.

In a laboratory investigation Mason (1987, 1989) investigated the scour of a submerged jet that he aerated with a known volume of air. To obtain the required volume of air, Mason reverted to Ervine's 1976 formula for calculating air entrainment. These tests were two-dimensional and the author recognised the fact that the third dimension may give totally different results. The jet also struck the bed of his model at 45° but no cognisance was given to this in the resultant formula.

From the results of his study, Mason proposed a new scour prediction formula, where the fall height factor of his previous formula was replaced with an aeration factor and the various coefficients altered;

$$D = 3{,}39 \frac{q^{0,60}\,(1 + \beta)^{0,30}\,h^{0,16}}{g^{0,30}\,d_m^{0,06}} \qquad .....\mathbf{18}$$

Mason, following Ervine, continued to use the value of 1,1 m/s as the minimum velocity of the jet to entrain air. Earlier, however, McKeogh and Elsawy (1980) found, during experimental tests on the effects of turbulence in the jet, that this value varied with turbulence level, from 0,8 to 2,8 m/s. The effects of this variation on the proposed formula are not known at this stage. Also, the estimation of the value of β is subjective and in later literature (Mason 1993) the author accepts that his simpler 1985 formula is the most practical expression for normal design use at dams.

2.3 Forces On The Bed Material

The first attempt to quantify the dynamic forces acting on the bed rock due to the action of the plunging jet, was carried out by Yuditskii (1963). The approach identified the erosion process as a balance between the weight of the rock blocks and the water forces acting on, as well as **within**, the rock mass.

Yuditskii measured the net forces acting on single model blocks as a function of time. Although his measurement system would record static loads very well, it is questionable as to how well it would have recorded dynamic loads. Never-the-less, Yuditskii managed to determine the required forces to remove blocks of rock and drew up graphs from which it is theoretically possible to predict the final depth of scour.

Based on the assumption that the maximum pressure pulses were responsible for the detachment of the block from its surrounds through hydrofracturing, Yuditskii proposed that the maximum tearing load *B* be given by;

$$B = T + \frac{A}{2} \qquad \text{.......19}$$

where	T	=	Time averaged tearing load.	(m water)
	A	=	Maximum pulsation amplitude.	(m water)

Once the rock block was detached from its surrounds, the time averaged load would increase due to the changes in the local flow conditions and if great enough, remove the block. Yuditskii determined the minimum forces required to move the blocks in separate tests, from which he could predict ultimate scour depths. This approach is unique in that the actual loads on the rock blocks are responsible for their detachment.

Despite the limitations of the experimental study, some basic findings can be highlighted;

- The forces on the rock blocks decrease as the tailwater depth increases.
- The maximum force is independent of the block size for small scour depths.
- For larger scour depths the maximum load will decrease as the block size increases.
- The time averaged load on the rock blocks will increase as the block size increases, which means that in certain cases, larger blocks will be more easily lifted than smaller blocks.

Anastasi *et al* (1979) investigated the stability of anchored stilling basin slabs. They found that differences between surface pressures on the slab and the internal joint pressures are responsible for the dislocation of the slabs. This was reinforced by Reinius (1986) while investigating the pressure distribution around blocks due to the action of a flow parallel to the surface. Any protrusion into the flow allowed high pressures to be transmitted into the joints between the blocks. These pressures could create uplift forces on the blocks, greater than the surface pressure. Reinius suggested that the pressure fluctuations of turbulence may add to the uplift forces and the vibrations may affect the blocks' stability.

Häusler (1983) demonstrated the effect of dynamic pressure in a crack, using a simple model. A single horizontal joint was manufactured in a concrete slab 1,0 m by 1,0 m by 70 mm (sealed on all sides). A 1 mm diameter hole was drilled from the top surface to the crack, and the slab placed under a 60 mm diameter plunging jet below a 600 mm deep water cushion. When the hole in the slab was covered, no additional pressures were measured inside the slab. Once opened to the impinging jet, however, pressures higher than the surface pressures were recorded over the entire area of the crack inside the slab, for a short time. Thereafter the slab *hydrofractured* and disintegrated as a result of these internal pressures.

2.4 Geotechnical Classification Of The Bedrock

In the majority of formulae proposed to date, the bed rock has been either represented by a single characteristic parameter such as d_m or, worse still, ignored altogether. The omission of other rock mass parameters in predictive formulae, must lead to errors that account (at least partly) for the wide scatter between predicted and observed data. In recent years, some authors have attempted to systematically classify the bed rock based on its scour resistance.

According to Spurr (1985), the resistance of a plunge pool to scour may be described by the geometric and mechanical characteristics of its bedrock, the most important of which are;

a) The rock mass' resistance to hydrofracture, as governed by its tensile properties, degree of fracture, faulting and bedding plane spacing.

b) The rock's resistance to erosive shear forces exerted on its surface by the action of the jet and governed by its cohesive strength.

Spurr put forward an "energy approach to estimating scour" which included the scour resistance of the rock mass as a major contributing factor. The geological parameters were represented by the uniaxial compressive strength σ_c of the rock mass as a main useful measure of the bedrock erosion strength. An insitu validation of this approach has only been carried out for a single prototype condition. It is, therefore, not certain to what degree the representation of the bedrock erosion resistance by the uniaxial compressive strength is valid.

Woodward (1981, 1984) on the other hand, investigated the prototype behaviour of fourteen spillways in Australia. He argued that the properties of strength and hardness of the intact rock were not relevant for the prediction of scour, because the scouring mechanisms of removing joint block by joint block, will be governed more by the jointing than by the properties of the intact rock. Woodward, therefore, stated that the average distance between natural fractures within the rock mass (fracture frequency) would be the most important rock mass property. The author used the Rock Quality Designation value (RQD) as a measure of this because of the ease of visualizing the physical appearance of a drill core. (RQD represents the percentage of the total length of a core that consists of separate pieces 100 mm or longer). However, this system is not always suitable as, in certain circumstances, it can give misleading values. In these cases it may be necessary to use the Fracture Log (natural breaks in drill core per metre length) to gain information about the bedrock.

Woodward also recognised that the degree of weathering of both the rock mass and the joint fill material are important factors. He drew up a table of scour resistance of the rock as functions of the state of weathering of the rock mass and joint fill material (Table I). It should be noted that this table does not take hydrofracturing into account.

Table I suggests that a rock mass be potentially erodible if the joint fill material is highly weathered. Thus, according to Woodward, a sound rock mass may still be erodible if the joint filling is highly weathered or at least thick enough to prevent rock to rock contact of the blocks. Other studies have shown that the rock mass is also potentially erodible by a plunging jet if the joints will allow water penetration, and thereby water pressure build up within the rock mass. Overall, this means that the process of erosion is **fracture controlled** and therefore has little dependence on the hardness, strength and durability of the intact rock blocks.

TABLE I : ASSESSMENT OF THE SCOUR RESISTANCE OF ROCK
After Woodward (1981)

<table>
<tr><td></td><td colspan="6">DEGREE OF WEATHERING OF THE JOINT FILL MATERIAL</td></tr>
<tr><td>ROCK MASS</td><td>COMPLETELY WEATHERED</td><td>EXTREMELY WEATHERED</td><td>MODERATELY WEATHERED</td><td>SLIGHTLY WEATHERED</td><td>STAINED JOINTS</td><td>FRESH ROCK</td></tr>
<tr><td>COMPLETELY WEATHERED</td><td colspan="2" rowspan="5">POTENTIALLY ERODIBLE ROCK MASS</td><td colspan="4" rowspan="2">DOES NOT EXIST</td></tr>
<tr><td>EXTREMELY WEATHERED</td></tr>
<tr><td>MODERATELY WEATHERED</td><td rowspan="3">SCOUR RESISTANT UNLESS HIGHLY SHEARED</td><td colspan="3"></td></tr>
<tr><td>SLIGHTLY WEATHERED</td><td rowspan="2"></td><td colspan="2"></td></tr>
<tr><td>STAINED JOINTS</td><td></td><td></td></tr>
<tr><td>FRESH ROCK</td><td colspan="6">DOES NOT EXIST BY DEFINITION</td></tr>
</table>

Kirsten (1982) proposed a geotechnical classification system for the excavation of natural materials, which was independent of the method of excavation. While this was not aimed at the scouring of a rock mass, some of the principles behind this system are relevant and have been adapted to a scour potential based classification system (van Schalkwyk *et al* 1994, Annandale 1994). The author identified the following characteristic parameters to define the excavatability of the rock;

1) Strength of parent material
2) Insitu density
3) Degree of weathering
4) Seismic velocity

5) Block size
6) Shape of excavation relative to equipment

7) Block shape
8) Block orientation

9) Joint roughness
10) Joint fill material
11) Joint separation

As some of these parameters are dependant on each other, the author grouped them into four variables:

a)	Mass strength number (items 1 to 4).	:	Ms
b)	Block size number (items 5 to 6).	:	RQD/Jn
c)	Relative ground structure number (items 7 to 8).	:	Js
d)	Joint strength number (items 9 to 11).	:	Jr/Ja

Using these variables Kirsten proposed a classification system;

Kirsten Number $$K_n = M_s \cdot \frac{RQD}{Jn} \cdot Js \cdot \frac{Jr}{Ja} \qquad .....20$$

Tables for calculating the variables (slightly modified for use in the hydraulics field) are given in Appendix A. At a workshop held on Erosion of Emergency Spillways under the auspices of the Soil Conservation Service (USA), Kirsten stated that his classification system could be applied to hydraulic erosion with very little change, (van Schalkwyk 1992). The factor describing the relative ground structure number would need some adjustment and a new parameter describing the channel bed irregularities would have to be defined. Never-the-less, both van Schalkwyk and Annandale have applied the Kirsten Number in its present form with success (with some slight modifications to the rating tables).

Van Schalkwyk *et al* (1994a), as part of a comprehensive study on the erodability of rock in unlined spillways, looked at forty-four points from eighteen dam sites, representing nine major rock types with varying degrees of erosion at various points along each spillway. Although this study was primarily aimed at erosion in channels, four free fall spillways were included for comparison. The author plotted the Kirsten Number, calculated for each point, against a chosen hydraulic parameter. From the work of Pitsiou (1990) it was evident that flow velocity was not entirely satisfactory as the hydraulic parameter and based on the work by Rooseboom and Mülke (1982) and others, he used the unit stream power. As increases in turbulence intensity will result in increased stream power and increases in the magnitude of fluctuating pressures, estimates of the unit stream power should therefore represent the relative magnitude of the fluctuating pressure and thus the erosive power of the water. This was calculated from (van Schalkwyk *et al* 1994b);

$$P = \rho g q S \qquad (\mathrm{W/m^2}) \text{ for channels} \qquad \text{.......21}$$

$$P = \rho g q \frac{H}{t} \approx 3 \rho g q \qquad (\mathrm{W/m^2}) \text{ for free fall} \qquad \text{.......22}$$

Note that the formula for free fall structures is an approximation based on the assumption that the jet thickness $t = H/3$. This is derived from empirical results and is probably accurate for solid jets, but looses accuracy once the jet starts to break up. Therefore, for free fall jets from high head structures, the original form of Equation 22 should rather be used if the necessary information is available. Van Schalkwyk's results are shown graphically in Figure 3, with the free fall dams highlighted. From the graph it is evident that there is a clear distinction between the classes of erosion. The author concludes that this indicates that the Kirsten value and the unit stream power give a good correlation. Furthermore, this relationship was found applicable to the straight drop as well as sloping channels.

Annandale (1994) followed a very similar line, using 150 field observations, including 137 made by the Soil Conservation Service of the USA. The author made one small change in that he used the rate of energy dissipation per unit width of flow (W/m) instead of the unit stream power ($\mathrm{W/m^2}$). Annandale derived a suite of formulae to calculate the rate of energy dissipation for the various flow conditions encountered in the analysis of his field data. By plotting the data on a log-log scale (Figure 4), it was possible to identify a scour threshold, from which it is theoretically possible to predict the possibility of scour at future sites.

2.5 Scale Modelling

Hydraulic model studies are normally undertaken during the design phase of a dam project, to optimise the spillway and appurtenant works and to investigate the expected scour in the plunge pool downstream of the dam. While the operation of the spillway may be accurately reproduced in the model, with the possible exception of aeration effects, the reproduction of actual scour depths is still questionable. The main reason for this is that a suitable material to represent the properties of the bed rock in the plunge pool has not been found to date. Never-the-less, models still provide some indication of what scour may be expected which, with experienced analysis, can provide reasonable estimates.

Other authors have also followed the modelling line of thought. For instance, in his thesis on the scour potential at Burdekin Falls Dam (Australia), Otto (1989) concludes, "To estimate the final scour depth at the Burdekin Falls Dam the author proposes specially designed scaled hydraulic model tests rather than the application of scour formula". He then goes on to suggest that a cohesive binder be used in the model bed material, to give a more realistic representation of the bed rock.

The most common method of representing the bed rock in scale models is as a movable bed of non-cohesive or cohesive material. In a few cases cubical blocks have also been tried. The use of non-cohesive granular material has a major drawback, widely recognised by various authors, namely it assumes equilibrium slopes unrelated to the properties of the rock mass it is supposed to represent. Therefore, the geometric boundaries of the model are totally different from the actual case and subsequently, the flow patterns are not correctly reproduced. This results in a large shallow scour hole that does not confine the plunging jet to any extent. Lateral flow and energy dissipation occur freely and the resultant ultimate scour is considerably shallower than it would be in reality. Note that this is in contrast to the argument that more energy is available in such a model as the jet does not have to fracture and loosen the bed rock blocks and therefore scour would be more extreme (Chee and Kung, 1971).

To overcome these problems, cohesive materials are used as the model bed rock. The binder allows the bed material to form vertical slopes, and (theoretically) reproduce the actual scour hole shape and flow patterns. Several authors have experimented with cohesive materials in physical hydraulic tests and in certain cases comparisons have been carried out between tests incorporating cohesive material and tests incorporating non-cohesive material.

A suitable cohesive material that would more accurately represent scour has been sought for some time. Johnson (1977) carried out tests using binders with granular material. The binders tried were clay, cement, a mixture of cement and sawdust, grease, and paraffin wax. The major difficulty was finding a binder that would maintain its characteristics with time.

Johnson found that the most versatile mixture was made with the following proportions (by volume):

Chalk powder	11,6 %
Cement	0,25 %
Water	18,55 %
Aggregate	69,6 %

The chalk powder provided a non-cohesive filler, having a grain size similar to that of the binding material, which acted as a strength control. Clay could also be used in place of cement but it tends to collect in the laboratory circulation system when large quantities are used. Representation of the rock resistance to scour was adjusted by varying the size of the aggregate used.

It was found that considerable precautions were required during manufacture to produce a homogeneous material in the model. In the case of cement as a binder, a minimum setting time of around two hours is necessary before submerging in water.

Although Johnson used the above mix to successfully reproduce scour below an existing dam, the mode of scouring was not necessarily correctly modelled. Therefore, any extension of the results from the model tests, automatically become questionable.

Gerodetti (1982) carried out tests on two different models where he used both loose aggregate and a cohesive material for the bed material and compared results. For the weakly cohesive material he used a mix very similar to that of Johnson, viz. (by volume):

Chalk powder	13,1 %
Cement	1,2 %
Water	6,0 %
Sand	32,1 %
Aggregate	47,6 %

He found that the scour depths were approximately 20 % and 30 % deeper with the cohesive binder for the two different models. This emphasises the point that model tests with non-cohesive beds can give optimistic results, a problem that could have catastrophic results.

3. COMPARISON OF PREDICTED SCOUR DEPTHS

3.1 Plunge Pool Scour At Katse Dam

3.1.1 The Lesotho Highlands Project

The Katse Dam, presently under construction on the Malibamatso River, is to form the main collecting reservoir for the Lesotho Highlands Water Project. This project is designed to enable Lesotho to export water to its neighbour, South Africa, generating hydroelectric power en route. The project is scheduled for construction in at least four phases over a period of thirty years, to transfer ultimately up to 70 m^3/s of water to South Africa. The dam itself will be a double curvature concrete arch, roughly 180 m high with a maximum crest length of 760 m. The lake created by the dam will have a volume of 2 000 x 10^6 m^3. Spillage of excess flood waters will be handled through an ungated crest spillway consisting of ten, 15 m wide bays. Due to the geology of the site, the allowable scour depth in the plunge pool below the dam wall had to be restricted and this necessitated the construction of a tailwater dam approximately 250 m downstream of the main wall. Hydraulic model studies were undertaken to optimise the design of the spillway crest arrangements on the main dam wall and to minimise the height, and therefore the cost, of the tailwater dam.

Two crest designs were tested, a Roberts-type splitter system and a serrated bucket deflector system. The latter, although not previously used on an arch dam, was attractive because it is capable of achieving large dispersion and downstream projection of the jet, with accompanying large-scale aeration. For a more detailed description refer to Furstenburg *et al* (1991).

3.1.2 Hydraulic model testing of Katse Dam

To optimise the disperser arrangements, and to select the most effective spillway layout and associated tailwater dam, two scale models were constructed. The first of these was built at the SOGREAH laboratory in Grenoble, France. It was a sectional model of the spillway crest to an undistorted scale of 1:30 and included a non-cohesive movable bed to facilitate assessment of the relative scour for different spillway configurations.

The spillway was improved (with respect to scour depths) in two ways, by improving air entrainment through the air slot and by adding springing blocks on the spillway face to divide the water jet. Both actions had the same ultimate effect, namely, the increased aeration and breakup of the jet during its fall. The results of these improvements with respect to scour depths were considerable (Table II). At the full flow of 6 080 m^3/s the scour depth (below original bed level) was reduced by 55 %. This highlights the importance of the degree of aeration on the scour potential of a falling jet.

TABLE II : SCOUR DEPTHS IN THE 1:30 KATSE DAM MODEL

DISCHARGE (m^3/s)	SCOUR* (m) Initial Design	SCOUR* (m) Final Design	SCOUR REDUCTION (m)
3 320	2	1	1
4 010	5	2	3
6 080	11	5	6

* Below initial bed level in model

The second model was built at the CSIR laboratory in Stellenbosch, South Africa. This comprehensive model included the arch dam wall with spillway, the tailwater dam and about 1 800 m of the river downstream. It was constructed to an undistorted scale of 1:70 and, similar to the previous model, incorporated a non-cohesive movable bed to study scour depths. With the design of the spillway optimised, the only other method of reducing the depth of scour in the plunge pool was by raising the tailwater level (Table III). In total the tailwater was raised by 18 % which in turn reduced the scour by 18 %. This demonstrates the effect that the tailwater depth has on the scour potential of the falling jet.

TABLE III : SCOUR DEPTHS IN THE 1:70 KATSE DAM MODEL

DISCHARGE (m^3/s)	TAILWATER DEPTH (m)	SCOUR* (m)
6 000	20,6	11
6 000	24,4	9

* Below initial bed level in model

3.1.3 Prediction of scour at Katse Dam

Following the model studies, the writer compared some of the scour depths with those predicted using various formulae (Table IV). It should be noted that the model dimensions were used in calculating the values and the results were then scaled up to the prototype afterwards. This affects the results of formulae such as those of Veronese (1937) (Equation 1) and Mason (1987) (Equation 18) but the remaining three formulae shown give the same results (when scaled up or down) for the prototype and the model dimensions. The first five columns show the test data, scaled up to prototype dimensions.

Column 6 shows the results from the 1937 Veronese formula (Equation 1), which in most cases over-predicts the depth of scour found in the model. Notably, when the fall height of the jet was relatively small (< 60 m), the predictions come closer to the model results. In these tests (which represented a flood overtopping the partly completed wall) there was very little break-up of the jet before it entered the tailwater, which effectively removes one of the major factors affecting the depth of scour.

Column 9 shows the results using Mason's (1985) formula (Equation 8) derived from the data of forty-seven model studies. On average this formula gave the best predictions, but as can be seen, the predictions still vary by up to 20 % either side of the model values. This may be considered acceptable in some cases, but in this particular case the scour depth had to be known to within approximately 5 %. There is very little difference between the results of this formula and that of his later formula that incorporated air entrainment (Column 10, Equation 18), despite the fact that the second formula was derived from two-dimensional laboratory tests.

The results of the SOFRELEC (Equation 6) and INCYTH formulae are included for interest. Despite their simplicity, they produce predictions almost as accurate as the Mason (1985) formula. This most likely implies that the formulae were derived from laboratory tests not dissimilar to those of the Katse Dam tests and it is therefore expected that their predictions may not be as good for different conditions.

On the whole the five formulae used give a reasonable indication of the scour depth, that is, none predict a scour depth that is totally different to what was found in the model. It is only the degree of accuracy that varies with the formulae in this case.

TABLE IV : MODELLED VERSUS PREDICTED SCOUR FOR THE KATSE DAM
(Values scaled up after calculation)

H_f (m)	q ($m^3/s/m$)	d (m)	h (m)	D (m)	VERONESE 1937 (m)	SOFRELEC 1980 (m)	INCYTH 1981 (m)	MASON 1985 (m)	MASON 1987 (m)
148,8	6,7	1,0	11,6	12	14	12	13	10	12
148,4	13,3	1,0	13,2	22	20	18	18	15	17
148,1	18,2	1,0	14,3	24	24	22	21	18	20
147,5	23,0	1,0	15,5	25	27	25	24	21	23
147,4	28,0	1,0	16,3	25	30	28	26	24	25
147,1	32,5	1,0	17,1	27	33	31	28	26	28
145,2	20,0	1,0	17,4	25	25	23	22	20	22
144,1	26,7	1,0	19,4	27	30	27	25	24	25
144,5	40,0	1,0	20,6	32	37	35	31	30	32
143,5	33,3	1,0	20,8	29	33	31	28	27	29
143,4	40,0	1,0	21,7	33	37	35	31	31	32
140,7	40,0	1,0	24,4	33	37	34	31	31	33
140,5	41,7	1,0	24,7	34	37	35	31	32	33
140,2	41,7	1,0	25,0	33	37	35	31	32	33
145,8	35,5	0,3	18,7	27	35	32	29	32	32
145,8	35,5	0,3	18,7	29	35	32	29	32	32
145,8	35,5	1,0	18,7	25	35	32	29	28	29
145,8	35,5	1,0	18,7	26	35	32	29	28	29
145,9	24,0	1,0	17,3	23	28	25	24	22	24
146,5	20,5	1,0	16,2	19	26	23	22	20	22
146,4	40,5	1,0	18,7	30	37	35	31	30	32
145,8	35,5	0,5	18,7	27	35	32	29	30	31
64,6	59,6	1,0	19,7	29	38	41	31	37	36
64,9	36,9	1,0	16,6	24	29	30	24	27	27
66,1	69,5	1,0	19,7	34	42	45	34	41	39
67,7	83,4	1,0	19,7	41	46	50	37	45	44
59,2	20,7	1,0	19,4	24	21	21	18	19	20
58,2	20,7	1,0	19,2	23	21	21	18	19	20
58,9	55,2	1,0	23,4	31	36	38	29	36	35

Note : D = scour below bed level + tailwater depth.

3.2 Plunge Pool Scour At Kariba Dam

3.2.1 Kariba Dam design

Kariba Dam is situated on the Zambezi River, 400 km downstream of the Victoria Falls, on the border of Zambia and Zimbabwe. It impounds one of the largest manmade lakes in the world with a maximum storage capacity of 181 000 x 10^6 m^3. The dam wall is a double curvature concrete arch with a maximum height of 128 m and a non-overspill crest length of 117 m. Due to the nature of the valley shape, 40 % of the length exceeds a height of 100 m. The dam design by André Coyne, was unprecedented for such a wide valley, being only 13 m thick at the crest and 18,5 m thick at the toe.

Excess flood waters are passed through six flood gates each 9,4 m wide by 9,7 m high, having their sill levels 33 m below the crest of the wall. Combined they can discharge a maximum of 9 500 m^3/s and the water jet is thrown more than 100 m downstream before striking the tailwater. When the decision to proceed with the construction of Kariba was taken in 1955, only four gates were planned with a combined capacity of 6 300 m^3/s. This was deemed adequate to pass the estimated probable maximum flood (PMF) based on the hydrological data available at the time. However, during the construction of the wall, on 25 March 1957, a flood with a peak flow of 8 200 m^3/s arrived at Kariba. This prompted a re-evaluation of the design flood for the dam, but the gates were found to still be adequate. The following year saw an even larger flood at Kariba with a peak discharge of 16 000 m^3/s on 5 March 1958. This flood was uncomfortably close to the design flood calculated for the dam the previous year. The extreme (PMF) flood values were reassessed once again and this time the number of gates had to be increased from four to six to pass the new extreme flood.

3.2.2 Formation of the scour hole

The initial model tests, carried out at SOGREAH (France), suggested that a scour hole approximately 30 m deep could be expected in the plunge pool (assuming construction programmes were followed). To minimise the effect of the scour hole on the dam, the wall was thickened at mid-height to attract loads to the abutments and thus reduce the cantilever stresses at the base of the dam.

Originally Kariba was to have two separate power stations, one on each bank which, when completed, would reduce spillage through the flood gates to about once in five years. As the right bank station was to be built simultaneously with the wall (completed in 1962) and the left bank station would follow immediately after that, it was accepted that annual spillage would occur in the initial years until the left bank station was completed. Rapid erosion would therefore take place in the plunge pool, but this was considered an acceptable risk for the limited period. However, by 1967 the scour hole had reached a depth of 50 m and it was realised that there could still be a considerable delay in the construction of the left bank power station. A review was undertaken which recommended the construction of a bypass tunnel on the left bank to relieve the pressure on the plunge pool. In the end the decision was taken to complete the left bank power station and the bypass tunnel was not constructed, despite the fact that the four units of the power station were only commissioned between 1975 and 1977. Thus prolonged spillage took place annually (with the exception of 1973 due to a drought) for a period of fifteen years (Figure 5).

The progress of the subsequent scour hole has been surveyed annually since the construction of the dam to ensure the stability of the banks (Figures 6, 7 and 8). Some of the scour levels recorded from these surveys are shown in Table V. Note that these depths are measured from the river bed level and do not include the tailwater depth. Due to an unprecedented drought since 1981, no spillage from the dam has occurred since then and therefore no further scour surveys have been undertaken.

TABLE V : ANNUAL SCOUR BELOW RIVER BED AT KARIBA DAM

YEAR	1962	1964	1965	1966	1970	1975	1976	1981
SCOUR+	29	42	47*	52	57	64	64	71

+ Metres below original bed level * Estimated

3.2.3 Hindcast prediction of scour at Kariba Dam

The characteristics of the Kariba Dam were fed into the same formulae that were used for Katse Dam, to see what each predicted as the resultant scour depth (Table VI). The first five columns show the site data, with the present scour depth of 71 m below the original bed level. Three cases were considered, a low flow of 800 m^3/s through one gate, equivalent to one gate approximately half open (Row 1) and then the full flow of 1 585 m^3/s per gate (Row 2 and 3) with two tailwater depths equivalent to one and six gates fully open respectively. It should also be borne in mind that it is possible that Kariba has not yet reached its ultimate scour depth.

TABLE VI : ACTUAL VERSUS PREDICTED SCOUR DEPTHS FOR KARIBA DAM

GATES (No.)	H (m)	q ($m^3/s/m$)	d (m)	h (m)	D 1981 (m)	VERONESE 1937 (m)	SOFRELEC 1980 (m)	INCYTH 1981 (m)	MASON 1985 (m)	MASON 1987 (m)
0,5	91,6	85,1	1,0	8,5	79,5	57,9	52,0	40,3	41,0	26,6
1	97,3	168,6	1,0	11,5	82,5	84,8	78,8	57,6	64,8	34,0
6	95,5	168,6	1,0	24,5	95,5	84,5	78,7	57,4	72,5	34,2

Note : D = scour below bed level + tailwater depth.

From the results in Table VI, it is apparent that the case of one gate open with maximum discharge gives the best predictions. This is not surprising, as the formulae are dependant on the flow per metre width of crest; therefore the worst scour condition would theoretically occur with one gate open at full discharge. Opening further gates would distribute the flow over a larger area, and the subsequent increasing in the total flow would raise the tailwater level.

Notably, in this case the Veronese formula (Equation 1) came closest to the present scour depth, over predicting by a mere 3 %. This was followed closely by the formula of SOFRELEC (Equation 6) which under predicted by only 4 %. Ironically, the only formula to incorporate air entrainment (Mason 1987) (Equation 18) gave particularly poor results in this case, under predicting by 59 %. This is in contrast to the Katse Dam model study, where this formula gave reasonable results.

3.3 Other Studies

3.3.1 Pongolapoort Dam

Scour downstream of the chute and flip-bucket spillway of the Pongolapoort Dam was measured following a major flood event. During the flood, spillage was measured as 17,4 $m^3/s/m$ which resulted in a scour depth, from the tailwater level to the bottom of the hole, of 13 m. The depths predicted for this case, by the formulae used earlier (Table VII) all overestimate this scour. This may of course be because the formulae are predicting the ultimate scour depth and it is not known whether the scour at Pongolapoort has reached its final depth yet.

TABLE VII : ACTUAL VERSUS PREDICTED SCOUR DEPTHS FOR PONGOLAPOORT DAM

H (m)	q ($m^3/s/m$)	d (m)	h (m)	D (m)	VERONESE 1937 (m)	SOFRELEC 1980 (m)	INCYTH 1981 (m)	MASON 1985 (m)	MASON 1987 (m)
64,0	17,4	0,25	6,0	13,0	22,6	19,3	16,7	17,1	14,2

Note : D = scour below bed level + tailwater depth.

The only formula to come close to the present scour is the latest one of Mason (Equation 18), which incorporates air entrainment effects. However, if the actual scour has not reached its ultimate depth, then this formula is likely to under predict the actual scour depth.

Two different models were constructed during the planning and construction of the dam, one at a scale of 1:52 and the other at a scale of 1:40. These predicted scour depths of 15 m at a unit discharge of 29,0 $m^3/s/m$ and 10 m at a unit discharge of 15,7 $m^3/s/m$ for the two models respectively. The 1:40 model test result used a comparable flow but was on the low side of the actual measured scour (10 m versus 13 m). Even at the higher flood used in the 1:52 model the scour depth was only 15 % deeper than that measured on site. It is thought that the two models' testing times were too short when compared to the actual conditions experienced and hence the tests may have been stopped before ultimate scour depths were reached.

3.3.2 Cabora Bassa Dam

The Cabora Bassa Dam makes use of eight middle-level gated outlets. A total of 13 100 m^3/s can be passed through the gates, which is equivalent to a unit discharge of 275 $m^3/s/m$. During the design stage, model tests were carried out at a scale of 1:75, to investigate the potential scour depths expected below the dam wall. The movable bed material was composed of gravel with a d_{50} = 28 mm (equivalent to 2,1 m in prototype) and also incorporated a cementatious binder. These model tests predicted a maximum scour depth of 75 m (measured from the tailwater level to the bottom of the hole). In February 1982 this depth was measured on site as 68 m (Breusers and Raudkivi 1991). The scour depths predicted from formulae are shown in Table VIII.

TABLE VIII : ACTUAL VERSUS PREDICTED SCOUR DEPTHS FOR CABORA BASSA DAM

H (m)	q ($m^3/s/m$)	d (m)	h (m)	D (m)	MODEL 1982 (m)	VERONESE 1937 (m)	SOFRELEC 1980 (m)	INCYTH 1981 (m)	MASON 1985 (m)	MASON 1987 (m)
100,0	275,0	2,1	40,0	68,0	75,0	111,2	106,0	74,1	97,4	40,7

Note : D = scour below bed level + tailwater depth.

Except for Mason's 1987 formula (Equation 18) all the formulae over predict the scour depth found on site. As with both Kariba and Pongolapoort, there is no guarantee that the ultimate scour depth has been reached. It is of interest to note that in this case the model, which used a cohesive bed material, came very close to the scour actually experienced. The range of predicted scour depths (40,7 to 111,2 m) is also particularly large in this instance. A fact that could be attributed (to some extent at least) to the angle of the jet, which strikes the tailwater at approximately 45°. None of the formulae tested here make any allowance for the angle of impact of the jet.

3.3.3 Dams in Turkey

Yildiz and Üzücek (1994) compared the actual depths of scours measured at three dams in Turkey, the Karakaya, Keban and Kiliçkaya Dams, with those predicted by several scouring equations. These included the formulae of Veronese (1937), Mason (1985) and Mason (1987). To these the writer has added the formulae of SOFRELEC (1980) and INCYTH (1981) for comparison with the other data in this report. The predictions of these five formulae for the three dams are shown in Table IX.

TABLE IX : ACTUAL VERSUS PREDICTED SCOUR DEPTHS FOR THE KARAKAYA, KEBAN AND KILIÇKAYA DAMS IN TURKEY

	H (m)	q ($m^3/s/m$)	d (m)	h (m)	D (m)	VERONESE 1937 (m)	SOFRELEC 1980 (m)	INCYTH 1981 (m)	MASON 1985 (m)	MASON 1987 (m)
KARAKAYA	148,0	96,3	0,65	8,3	37,5	68,9	58,7	48,4	47,0	33,1
KEBAN	143,2	32,6	0,65	5,4	23,0	38,1	30,6	27,9	23,0	23,4
KILIÇKAYA	97,0	12,1	0,65	2,4	12,0	20,4	16,2	15,4	11,0	14,1

Note : D = scour below bed level + tailwater depth.

The three first equations all over estimate the scour by up to 84 %. Mason's formula from model data (1985), however, gives a spread of values, overestimating the scour by 25 % at the

Karakaya Dam, precisely predicting the scour at Keban Dam and under predicting the scour by 8 % at the Kiliçkaya Dam. This, therefore may appear to be the better equation for these situations, if one accepts that the scour holes are at, or close to, their ultimate depths. Given the fact that these three dams are not that old, this may well not be so, in which case one of the earlier formulae may be giving more realistic predictions.

Yildiz and Üzücek conclude that the inclusion of the grain size in Mason's formulae improves predictions at these sites, but what is really lacking is prototype scour data. Consequently, it will be a major contribution to the investigations carried out on the subject if more data obtained from carefully monitoring prototype scour development are made available in the literature.

3.4 Lessons To Be Drawn From The Case Studies

3.4.1 Reliability of scour prediction formulae

Although the predictions of only five formulae are shown here, they are the best of nineteen tried by the writer. Nevertheless, the accuracy was only of the order of 20 % for individual cases. Furthermore, it was not the same formula that gave this best accuracy in each instance. So, while the 20 % accuracy may be adequate in certain cases, the question of which formula to choose for the *correct* prediction is unanswerable.

It would appear that while extensive work has been carried out towards finding a generic equation to predict scour depths, a satisfactory solution is still not available. The literature has shown that the most popular form of such an equation was;

$$D = K \frac{q^x H^y}{d^z}$$

In certain cases some of the variables have been omitted altogether and/or others added. Although this form may be convenient, in that all the variables are easily defined and measurable, it has failed to produce the required results. By varying the coefficients, the formula has been made to *work* for particular cases, but as soon as the given site conditions are changed, the accuracy of the formula is lost. It is evident, therefore, that the site conditions, which are not fully represented in this form of the predictive equation, play a major role. In essence, this means that some form of detailed description of the bed material needs to be incorporated into the equation. This in itself would be a mammoth task, and it is doubtful if there is adequate information available at this stage to achieve meaningful results.

In this regard, very little effort has been made to date to investigate the dynamic forces on the bed material and the geological and hydraulic parameters which control the break up thereof. Although there are indicators of how geological parameters can influence the scour behaviour of a rock mass, there is no generally accepted concept of how to quantify their contribution to the scour resistance of the rock mass, or how they could be incorporated into a prediction of the extent of the scour. In hindsight, given the variability in conditions at each site, particularly with respect to bed rock, it may be unrealistic to expect a single generic formula to yield applicable results.

It can therefore be concluded that, at this point in time, formulae can only be used to give an approximate indication of the expected scour depth.

4. DAM SITE SELECTION FOR MODEL TESTS

4.1 Site Criteria

From the outset it was not expected to find a dam site that would be 100 % ideal. Therefore, the criteria that the perfect dam site should possess were ranked in order of importance. Obviously, certain criteria would be compulsory and any dam not conforming to these basic requirements could be eliminated immediately. After analysis of the desirable requirements it was found that only four were compulsory, which are:

a) The dam spillway must have a free-fall jet.
b) No protective aprons must exist in the plunge pool.
c) Flood or spillage histories must be available.
d) Substantial scour must have occurred in the plunge pool.

In addition to these requirements, several other requirements had to be met for the *ideal* site. These could be regarded as 'nice-to-have' but not essential. The main criteria in this category are:

e) Minimal or no aeration of the free-falling jet.
f) Scour history.
g) Tailwater rating curves.
h) Uniform bed rock within the plunge pool.
i) Uniform/regular jointing of the bed rock.
j) No debris barrier due to the scouring in the plunge pool.
k) Easy access to the site.

Having identified the desirable criteria, these were then compared to the available dam sites (Section 4.2) and the most suitable chosen.

4.2 Available Sites

Initially a list of the larger dams within South Africa was drawn up. Based on this and from the studies of Pitsiou (1990) and van Schalkwyk (1991) as well as personal communications with Prof. van Schalkwyk and Dr J Jordaan, it was found that only four of the dams satisfied the first three criteria and none the fourth. That is none of the suitable dams had had substantial scour (this is considered further in Section 4.3).

The four possible sites considered were:

Craigie Burn	near Greytown	in Natal
Kat River	near Seymour in E. Cape	
Roodeplaat	near Pretoria	in Transvaal
Wagendrift	near Escourt	in Natal

Details of the dams, bed rock types, Kirsten Numbers and estimated scour depths are given in Table X.

TABLE X : DETAILS OF AVAILABLE DAMS

Dam / Feature	Craigie-Burn	Kat River	Roodeplaat	Wagendrift
Wall	Conc. Arch	Conc. Arch	Conc. Arch	Conc. Arch
Spillway	Overflow	Overflow	Overflow/Splitters	Overflow
Height Spillway (m)	35,3	~ 37,7	50,4	35,0
Length Spillway (m)	121,9	120	143,0	120 (102,7)
Max Capacity (m^3/s)	340	---	970	1300
Rock Type	Dolerite	Dolerite	Basic Volcanic	Shale : Dolerite
Density (kg/m^3)	2900	(2900)	2700	2500 2900
Kirsten No.	1300(lft) 1730(rght)	1700 4000	3570	400 2230
Peak Outflow (m^3/s)	366,5	114,8	961,0	687,1
Max Daily Ave(m^3/s)	169,0	---	207,0	359
Max. Scour (m)	0,8	~ 1,5	0,3	1,5 1,0
NOTES	Heavily jointed (adverse direction)		Heavily jointed	From 1976 to 1987 scour surveys carried out. No major faults

4.3 Best Suited Site

Roodeplaat (formerly known as Pienaars River Dam) fails the first of the non-compulsory criteria in that the crest incorporates splitters to aerate the nappe. From a modelling point of view this is undesirable as the additional aeration may introduce uncertainties, as aeration effects are not accurately scaled in Froude models. This makes the Roodeplaat Dam the least desirable of the four.

The remaining three dams have little to differentiate them on the basis of the desired requirements. One does stand out as having certain advantages however, and that is the multiple-arch Wagendrift Dam. This dam effectively has two similar curved spillways, separated by a buttress-support. The rock below each section is slightly different, due to the fact that dolerite intrusions have replaced the original shale in the left hand plunge pool area. The advantage in this is that only one half of the spillway need be modelled and two different bedrocks could be tested, with only minor changes to the spillway.

Further-more, according to Pitsiou (1990), from about 1976 through to 1989, regular scour surveys of the plunge pool area have been carried out. In 1989, due to progressive scouring in the plunge pool, thick reinforced concrete slabs with rock bolts were constructed in both the right and left plunge pools. Although ordinarily the concrete aprons would make the dam site useless (Section 4.1) the fact that scour surveys were taken means that the required data is still available and the possibility exists for the scour progress to be linked to specific flood events.

It was concluded that the best dam site within South Africa for the purposes of this study was the Wagendrift Dam on the Bushman River. However, the lack of appreciable scour does present a problem. The proposed scale model would be expected to reproduce the scour at the chosen site to within a certain accuracy. Due to the scaling, and the uncertain nature of the scouring, it is not expected that this accuracy will be better than ± 1 m. The scour at Wagendrift (prior to the concrete aprons) is only 1,5 m deep, which is of the same order as the accuracy the model can be expected to achieve. Thus any attempt to use such a shallow scour to determine optimum semi-cohesive modelling material would be misleading. For this study, the dam must have a scour hole in excess of 20 m deep, if any degree of accuracy is to be maintained. It has to be concluded, therefore, that there were **no suitable sites**, for the purposes of this study, found within South Africa.

4.4 Possible Sites Outside South Africa

From the conclusions drawn regarding dams within South Africa, it was evident that the prime requirement to look for was actually *substantial scour within the plunge pool* and in hind-site this should have been the first criterion.

Having found nothing in South Africa, the search was extended to neighbouring states. It was kept in mind that by attempting to stay within southern Africa the general characteristics of the bed rocks would not be very different from those within South Africa.

One site overshadowed all others here, and that was the Kariba Dam on the Zambezi River. With the possible exception of access, this site was almost perfect, satisfying all the given criteria. The scour hole in the plunge pool has extended almost 75 m below the river bed and annual surveys have been kept of the progress of the hole since the construction of the dam (Section 3.2.2). It was therefore decided to use Kariba as a basis for the model tests.

5. KARIBA DAM MODEL

5.1 Model Scale

The scale of a physical model is normally a compromise between construction costs and the achievable accuracy, limited by the available physical resources. In this particular case, an additional factor was also taken into account, that of the work done on the Katse Dam model study. A lot of information was available from these model tests (CSIR 1990), the model for which was constructed at a scale of 1:70. For ease of comparison and to avoid any problems regarding possible scaling effects, it was prudent to ensure that the Kariba Dam model was built at a similar scale.

Despite the extremely deep scour hole formed at Kariba compared to that expected at Katse, it was still possible to build the Kariba Dam model at the desired scale of 1:70, making use of the existing Katse Dam model superstructure to curtail costs (Figure 9).

The model was operated according to the Froude Law of similitude as the forces acting on the flow were predominantly gravitational. Providing there is correct geometrical similarity between model and prototype, then it follows (apart from scale effects due to viscosity) that the ratio between inertial and gravitational forces on all particles of the fluid will be of equal magnitude in the model and in the prototype. Therefore, the geometry of flow will be preserved throughout the model.

Using suffix *p* to denote prototype and *m* model dimensions, the following scale relationships will hold;

$$\textit{Linear Ratio} = \frac{L_p}{L_m} = S = 70$$

$$\textit{Area Ratio} = \frac{L_p^2}{L_m^2} = S^2 = 4\ 900$$

$$\textit{Volume Ratio} = \frac{L_p^3}{L_m^3} = S^3 = 343\ 000$$

$$\textit{Velocity Ratio} = \frac{V_p}{V_m} = \sqrt{S} = 8{,}367$$

$$\textit{Discharge Ratio} = \frac{V_p}{V_m} \cdot \frac{L_p^2}{L_m^2} = S^{2,5} = 40\ 996{,}3$$

Note that all dimensions in this report refer to the prototype, unless specifically stated.

5.2 Scale Effects Through The Gates

It is difficult at a scale of 1:70 to reproduce a hydraulic surface smooth enough to represent the steel lined gates in the dam wall. In the model these were manufactured from timber and varnished to provide a smooth surface. It was therefore necessary to calculate the estimated head losses expected in the model, compared to those of the prototype, to ensure that similarity was maintained.

The total head losses Δh through each gate can be given as:

Δh = entry losses + friction losses + contraction losses + exit losses

which in equation form is:

$$\Delta h = C_1 \frac{v_1^2}{2g} + \frac{\lambda L}{4R} \cdot \frac{v^2}{2g} + C_2 \frac{v_2^2}{2g} + C_3 \frac{v_3^2}{2g}$$

where	$C_1\ C_2\ C_3$	=	Headloss coefficients at sections 1, 2 and 3.
	$v_1\ v_2\ v_3$ =		Average flow velocity at sections 1, 2 and 3.
	v	=	Average flow velocity through the gate.
	g	=	Gravitational acceleration.
	λ	=	Frictional coefficient.
	R	=	Hydraulic radius.
	L	=	Length of conduit.

The individual head losses were calculated using the relevant data at each section through the gate, for the prototype and for the model. The latter result was then scaled up to compare it with that of the prototype.

As the entrance to the gates was a smooth formed section, the entrance headloss coefficient was taken as C_1 = 0,05. The exit of the gates on the other hand was abrupt with sharp edges and so the exit headloss coefficient was taken as C_3 = 1,0. The headloss coefficient for the constriction within the gate body was calculated from the general formula $(A_1/A_2 - 1)^2$, where A_i represents the cross-sectional flow area at section i. The headloss coefficient was found to be C_2 = 0,195.

The friction coefficient λ was found from the Moody diagram (Moody 1944) and was taken as 0,016 for the prototype and 0,025 for the model. The actual headloss calculations are summarised in Tables XI and XII.

TABLE XI : PROTOTYPE HEAD LOSSES THROUGH GATES

DATA	UNIT	ENTRANCE	FRICTION	CONTRACTION	EXIT
FLOW	m^3/s	1584	1584	1584	1584
LENGTH	m	---	13,15	---	---
X-AREA	m^2	123,4	104,1	111,6	77,4
HYDRAULIC RADIUS	m	2,77	2,53	2,63	2,20
VELOCITY	m/s	12,84	15,22	14,19	20,47
$v^2/(2g)$	m	8,40	11,80	10,27	21,35
REYNOLDS No.	—	---	1,54E+08	---	---
e/(4R)	—	---	0,0004	---	---
LAMBDA	—	---	0,0160	---	---
COEFFICIENT	—	0,05	0,021	0,195	1,0
Δh	m	0,42	0,25	2,00	21,35

TABLE XII : MODEL HEAD LOSSES THROUGH THE GATES

DATA	UNIT	ENTRANCE	FRICTION	CONTRACTION	EXIT
FLOW	m^3/s	0,0386	0,0386	0,0386	0,0386
LENGTH	m	---	0,188	---	---
X-AREA	m^2	0,0252	0,0212	0,0228	0,0158
HYDRAULIC RADIUS	m	0,0396	0,0361	0,0376	0,0314
VELOCITY	m/s	1,533	1,823	1,695	2,446
$v^2/(2g)$	m	0,120	0,169	0,146	0,305
REYNOLDS No.	—	---	2,63E+05	---	---
e/(4R)	—	---	0,0021	---	---
LAMBDA	—	---	0,0245	---	---
COEFFICIENT	—	0,05	0,032	0,195	1,0
Δh	m	0,006	0,005	0,029	0,305

The **total** headlosses are:

Prototype	=	24,02 m
Model	=	24,15 m (scaled up)

The model therefore reproduced the gate characteristics very well and there was no need to adjust the sizing of the gates (Figure 10).

5.3 Model Construction

The model was designed and built according to the drawings supplied by Dr P J Mason of Sir Alexander Gibb and Partners with the kind permission of the Zambezi River Authority (ZRA). Later, modifications were made to the downstream topography, after more detailed drawings were received from Mr B Goguel of Coyne *et* Bellier.

To reduce costs, use was made of the existing Katse Dam model in EMATEK's Hydraulics Laboratory in Stellenbosch. The extent of the model was chosen to ensure that the boundaries were sufficiently removed from the test areas as not to impose artificial conditions. Upstream of the dam wall, 600 m of the lake was represented with a further 100 m being used as a stilling area for the incoming water. Downstream, only the immediate plunge pool area was modelled and a control gate used at the downstream end to maintain the correct tailwater levels. The distance modelled, from the toe of the dam wall to the control gate, was approximately 360 m (Figure 9).

Water was supplied to the model from the overhead constant-head tank, via a 500 mm diameter pipe that was connected to an elevated right-angled V-notch box above the model. Three valves, one 100 mm and two 300 mm diameter, were used to control the flow into the box. It was found that once set at the required flow, no further correction was required for the duration of the test as the flow remained perfectly stable.

5.4 Tailwater Levels

A rating curve of the tailwater levels for given flows was supplied by Sir Alexander Gibb and Partners (Figure 11). It was found that this approximated to the curve:

$$LEVEL\ (m\ asl) = 379{,}8846 + 0{,}214835\ (\sqrt{FLOW - 333.0594}\ )$$

To achieve these levels in the model during testing, it was necessary to provide some form of weir. This is normally done with a tailwater gate that can be raised or lowered to suit and effectively acts as a sharp crested weir. Initial calculations showed that the gate would have to be raised progressively with increased flow to achieve the desired tailwater levels. Due to the variability of flow expected during testing, adjusting the gate continuously was out of the question and a better system was sought.

The problem was solved by installing a stepped weir that increased in width with flow, such that the desired tailwater levels were maintained. To get the first approximation of the sizes for such a gate, an appropriate formula was found for the initial system. This could be approximated by the formula of Kindsvater and Carter (Ackers *et al* 1978). The various coefficients were based on the flow being partially contracted with a width-of-river to a width-of-opening ratio of approximately 0,4.

$$Q = C\ (\ 1 + 0{,}01\ \frac{h}{p}\ )\ (\ b + 0{,}003\ )\ (\ h + 0{,}001\ )^{1{,}5}$$

where Q	=	flow	(m^3/s)
C	=	weir coefficient	(-)
h	=	head above weir crest	(m)

p	=	height from river bed to weir crest	(m)
b	=	breadth of weir	(m)

The normal value of C is about 1,7. However, from the calibration runs it was found that in this case C was (on average) approximately 4,27. This increase was probably due to the unusually turbulent flow characteristics close to the weir caused by the plunging jet.

When calculating the size of the weir, special note had to be taken of the limits of validity of the formula, that is:

$p > 0{,}1$ m : h > 0,03 m : b > 0,15 m : (River width - b) > 0,2 m : h/p < 2

Using the above formula, the shape of the proposed stepped weir was calculated. It was accepted that as the new weir would change the flow conditions, the formula used would no longer be valid and further modification of the weir, once in place, would be necessary. The final calibration of the weir gave tailwater levels that were very close to those desired (Table XIII and Figure 11). The actual weir openings at given levels are also shown in Table XIII.

TABLE XIII : TAILWATER WEIR SHAPE

FLOW (m^3/s)	WEIR CREST LEVEL (m asl)	REQUIRED TAILWATER (m asl)	ACTUAL TAILWATER (m asl)	WIDTH (m model)	CUMULATIVE WIDTH (m model)
1 584	377,0	387,5	388,1	0,279	0,279
3 167	377,0	391,3	391,6	0,279	0,279
4 750	387,5	394,2	394,3	0,554	0,833
6 333	387,5	396,5	396,8	0,554	0,833
7 917	387,5	398,6	398,5	0,554	0,833
9 500	387,5	400,5	400,3	0,554	0,833

5.5 Test Flows / Hydrograph

Ideally, the tests should have been run with a hydrograph simulating the actual spillage experienced at Kariba Dam, since its commissioning. However, due to the dynamic nature of flows through the dam during the twenty years of operation, this would not be feasible in a model of this size. An attempt was made, therefore, to produce a simplified hydrograph. Similar flows (that occurred within a single period between scour surveys) were averaged to form single flows and some of the short dips in the flow (less than five days) were removed. This, however, still gave a very complex hydrograph. By superimposing the actual scour depths onto the hydrograph, changes in scour depth could be associated with particular flow phenomena. It was then possible to see that certain periods of low flow apparently had little or no effect on the size of the scour hole. For testing purposes, these could then be left out. The actual cutoff level, at which to remove lower flows, could not be determined with any accuracy, so an arbitrary value of 2 000 m^3/s was taken. The resultant simplified hydrograph is shown in Figure 12.

The simplified hydrograph was now 1 256 days long, having been trimmed from the original 2 247,5 days of spillage. Scaling this according to the Froude law would produce a **model hydrograph** approximately 150 days long, or in other words, it would take almost half a year to run one test. Fortunately, it is known that the time scale of the actual scour process is not the same as the flow rate time scale of the model. It is thought that this time scale can be approximated by roughly ten times the Froude time scale (Zwamborn 1969). If this is so, the length of the hydrograph can be reduced to 15 days. This is obviously still impractical for any testing sequence.

As the tests were being run to find a suitable modelling material, it was possible to choose materials with substantially faster scour rates. In this regard, it was thought (based on previous laboratory experience) that an average test should last between 0,5 and 2,0 hours. This time period was, therefore, used when testing possible model materials. It was found however, that in most cases, the final scour depth in the model was reached in less than 0,5 hours (Section 6).

In lieu of a hydrograph, the model materials were tested with constant flows, representing the maximum flows for a given number of open gates. A summary of the flows used is given in Table XIV.

TABLE XIV : FLOWS USED IN MODEL TESTS

GATES OPEN No.	FLOW (ACTUAL) m^3/s	FLOW (MODEL) l/s
1	1 584	38,6
2	3 167	77,3
3	4 750	115,9
4	6 333	154,4
5	7 917	193,1
6	9 500	231,7

6. MODEL TEST RESULTS

Several materials were considered for the representation of the bed material in models. Certain of these were rejected at the outset or after investigation of samples made up in the laboratory, mainly for practical reasons. These included certain binders such as paint, both enamel and PVA, grease and paraffin wax as well as all materials that change strength with time when saturated.

Also considered during the course of the testing was the possibility of using small, very weak, cement blocks in place of the aggregate. These would break up, once removed from their binder by the action of the jet. The conflicting demand for a cement weak enough to break up under the action of the jet while still being strong enough to remain intact during mixing (with a binder) and placing in the model, eventually led to the dismissal of this idea.

The materials and methods that did show varying degrees of success are described in detail below. Except for the initial block tests, all the tests were carried out with a preformed scour hole, slightly larger than that presently found at Kariba. This meant that only limited quantities of mixes were required for tests and made it easy to see where the scour had exceeded that expected.

6.1 Concrete Blocks

Some 2 000 concrete blocks were manufactured specifically for the purposes of these tests. For practical purposes the blocks were all the same size and measured 50 x 25 x 25 mm (referred to hereafter as standard blocks). These were placed in three orientations with respect to the impacting jet; vertically, normal to the river and parallel to the river. The blocks were placed by hand and a loose packing was maintained at all times, such that, although the blocks were touching, they could be lifted with relative ease. Although this may not be truly representative of a jointed bed rock, which could have a tight packing, it was argued that with time the blocks in the actual bed rock would have to loosen before being removed. Placing the blocks loosely in the model would merely speed up this process and possibly also allow for the fact that a model may not reproduce forces equal in magnitude to those experienced in nature. The top surface of the layer of blocks was not deliberately distorted (to form irregular bedding planes) however, due to some irregularity in the blocks themselves, steps between individual blocks varied form nothing to as much as 3 mm.

Tests were conducted by placing the blocks in the model such that the average top surface of the blocks was at the desired level. Increasing flows (Table XIV) were then passed through the dam for periods of thirty minutes at a time. Damage (if any) was inspected between each flow but no repair work was carried out between flows.

6.1.1 Tests in the Katse Dam model

Whilst awaiting permission to use the Kariba Dam for the model testing, the Katse Dam model was reinstated to carry out some exploratory tests on the blocks. The blocks were placed in the model such that the top layer was at original river bed level. Note that at this stage not all the standard blocks were available for testing, so use was made of other larger blocks as well in some tests.

It was found that scour was very limited, and in only one case did the scour extend to the second layer of blocks. The actual mechanism of the initial scour did, however, appear to be correct. The first few blocks to move were ejected vertically (presumably by hydraulic pressures under the blocks), after which the entire system would collapse.

The only test where scour extended into the second layer of blocks (and therefore gave a reasonable scour depth) was compared to an equivalent scour test carried out with the model with loose gravel as the bed material. The difference in scour levels was approximately 7 m, the gravel giving a deeper scour (Figure 13). It was of particular interest to note that the maximum scour level was reached at the **same flow rate** in both instances.

From these limited tests, it would appear that the initiation of the scour is not strongly linked to the size of the block, however the depth of scour may be. Once scour initiates, it is rapid to a certain level whereafter it remains constant for a range of increasing flows. This is easily understood in the case of concrete blocks (fixed step height), but is not easily explained in the case of loose gravel where a smooth curve is expected.

The fact that the blocks did not scour to the depths that were experienced with the loose gravel, could be explained in one of two ways. Either the blocks are too large, in which case a much smaller block would have to be used. (This would be impractical however, as the labour cost to manufacture and pack the numbers of blocks required would be prohibitive.) Or, the block-concept is not correctly representing the natural bedrock in the model situation.

6.1.2 Tests in the Kariba Dam model

Having constructed and calibrated the Kariba Dam model, the first scour tests were undertaken, using the concrete blocks, tested earlier in the Katse model, to represent the bed material. As there were too few blocks to completely cover the expected scour area, from the known scour depth up to the original river bed level, the final level of the top of the blocks was left lower than the original bed level. This was based on the premise that the scour would in all probability extend beyond this level anyway. It did not, of course, take into account any confining effect the hole might have on the jet at that level.

The flow was increased gradually up to the peak flow expected for six gates. However, very little movement of the blocks was found to take place throughout the range of flows. It was apparent that either the blocks did not work at all, or the confining effect of the hole is considerably more significant than first expected. To check which was the case, it was decided to construct the actual scour hole out of weak cement so that the jet would be properly confined. Again the blocks were placed in the hole up to the same level as before and the flow was then increased gradually.

By the time the flow had reached 50 % of the peak (three gates open), every block had been removed from the hole. It was evident that the confining effect of the hole plays a **major role** in the depth of scour. Further more, the test showed that contrary to initial indications, the chosen block size was too small to be used without some form of cohesive binder to reduce the rate of scour. However, the method of getting a binder onto the blocks uniformly, such that tests would be repeatable, is problematic. Furthermore, in most cases tested later, the material could not be used twice due to the remnants of binder on the material. Obviously, due to the cost of manufacture, for the blocks to be viable, they would have to be reusable. It was found that attempting to wash the blocks was extremely labour intensive and therefore not viable. The option of increasing the block size is not considered desirable as the height of the block dictates the scour depth steps, that is, the achievable accuracy is limited to the size of the block.

Purely for interest sake, the model fixed scour hole was filled with loose aggregate, of the same shape and size as used in previous model tests. Being smaller than the blocks, it was expected that the aggregate would be washed out of the hole very quickly. However, it was found that this was not so. The upstream end of the hole was not eroded as quickly as the rest of the hole, such that after a flow with four gates open, some aggregate still remained (Figure 14). At first it was thought that this must have something to do with the flow patterns in and around the aggregate as opposed to the blocks. However, following later tests (Section 6.6) it was found that the aggregate was too heavy to be easily expelled from the hole. The blocks, having a lower specific gravity required less force per unit area to move them, a finding quantified by Yuditskii (1963) (Section 2.3).

6.2 Aggregate With Bentonite Based Binder

In choosing a suitable cohesive binder to start testing with, the work of van Schalkwyk *et al* (1994a) was drawn on. From their studies it appeared that the best material for these purposes would be a mix of sand and bentonite clay. The aggregate used in all but the last few tests was a uniform 19 mm granite stone (representing a rock about 1,3 m diameter in prototype) from Tygerberg quarries in Cape Town. The stone was specially chosen for modelling, as the chips tended to be more cubic than flat in shape and therefore more representative of what would be expected in prototype. No attempt was made to try to replicate any *typical* grading curve of material.

6.2.1 Jet tests on samples

Initial trials suggested that a mix of sand, bentonite and aggregate in the ratio 1:2,5:12 would be appropriate. The sand to bentonite ratio (binder) was based on the original mix of van Schalkwyk *et al.* The ratio of binder to aggregate was initially taken such that the binder should approximately fill the voids between the aggregate, as the joint fill material would in jointed rock.

This mix however gave poor results in the first tests (Section 6.2.2). It is thought that this may have been due to two factors, firstly, the low permeability of the mixture prevented the ingress of water into the bed rock matrix and therefore prevented the build up of internal water pressures. Secondly, the binder itself may have been too cohesive for the model scale and conditions. Samples were, therefore, prepared using weaker binders and a significantly lower ratio of binder to aggregate.

When subjecting the samples to jets of water, it appeared that this mixture may erode too quickly. Therefore, the mix used for the next model test had a slightly greater binder to aggregate ratio, although the strength of the binder was not changed.

6.2.2 Tests in the Kariba Dam model

The first test with the 1:2,5:12 mix gave surprisingly poor results, despite the indications of the initial test samples. With only one gate open, no significant scour occurred at all. Increasing the number of gates started the scour process, but this did not extend very deep. Even running the model for two hours, with three gates open, resulted in limited scour. After the flow of six gates (with no repair work in between) the scour had reached a level of 343,7 m asl, only 5,3 m deeper than the scour attained after the flow from two gates and about 38 m short of the actual scour at Kariba (Figure 15).

To check the effect of time on the scour process in the model, the test was then continued, with all six gates open for five hours. The scour progress was measured every half hour. After the initial half hour the scour dropped 4,5 m to 339,2 m asl but then progress slowed and by the end of the five hours the scour had only reached a level of 336,0 m (Figure 16).

It became evident that a more porous matrix was required to simulate the scour process correctly. After some tests with cretestone as the binder, (Section 6.3) further tests were conducted with the bentonite based binder. In the first of these, the bentonite clay ratio was reduced dramatically, from 2,5 to 0,5 and the aggregate increased slightly, to give a final mix of;

SAND	BENTONITE	AGGREGATE
1	0,5	15

In this case the scour, after running three gates for twenty minutes, was extensive, the deepest point being at 325,5 m asl (Figure 17). The scour hole, however, was too large in plan and had collapsed along the sides. It appeared that the binder was too weak. For the next test this was improved by increasing the bentonite content from 0,5 to 0,75. The subsequent increase in binder to aggregate ratio was not adjusted as this would add to the increase in the cohesiveness of the matrix as a whole.

The resultant scour hole, after running three gates again, was more confined and reached a level of 329,4 m asl. Further testing with six gates took the level down to 307,3 m asl (very close to that desired) but as before the hole was too large in plan, particularly in the downstream direction (Figure 18). It was concluded that the general matrix strength was too low. The first tests showed that increasing the binder strength by increasing the bentonite content or increasing the matrix strength by increasing the binder volumes, restricted the scour progress. A stronger binding material was, therefore, needed.

6.3 Aggregate With Cretestone Based Binder

6.3.1 Jet tests on samples

Four samples were made up to test the viability of using cretestone as a binder for the aggregate. The mixes tested were;

	SAND	CRETESTONE	AGGREGATE
Sample 1	0	1	18
Sample 2	1	1	36
Sample 3	1	1	15
Sample 4	1	2	15

From the tests on the samples it was found that the best binder strength was that of Sample 2 the but the most appropriate binder to aggregate ratio was that of Sample 4. Thus, for the model test a mix of 1:2:35 was used.

6.3.2 Tests in the Kariba Dam model

With a flow of six gates for half an hour, all the material was washed out of the downstream end of the scour hole (Figure 19). It at first seemed that the mixture had been too weak, although the test samples did not show this. However, on closer investigation, it was found that the cretestone had started to dissolve whilst being saturated for a few hours before the test. This therefore, meant that cretestone was unsuitable as a test material and no further testing was carried out with it.

6.4 Aggregate With Plaster Based Binder

6.4.1 Jet tests on samples

To address the problem of the cretestone breaking down when saturated, the possibility of using plaster-of-paris was investigated. Plaster-of-paris has an extremely low dissolution rate and has the advantage of setting rapidly. This means that it is possible to carry out tests the same day that the material is placed. A disadvantage, however, is the cost of plaster-of-paris, this being relatively high compared to other materials tested. Some samples were prepared in the following proportions;

	SAND	PLASTER-OF-PARIS	AGGREGATE
Sample 1	0	1	18
Sample 2	1	1	35
Sample 3	1	2	35

Only the weakest mix (1:1:35) showed promise, the others being too strong.

6.4.2 Tests in the Kariba Dam model

From the outset, it was found that the rapid setting of the plaster-of-paris was problematic. Only small amounts could be mixed at a time and this led to problems of homogeneity within the material in the model. Never-the-less, with some practice, the construction team managed to get this down to a fine art and reliable tests could be run.

The initial mix of 1:1:35 was made up in small batches (due to the quick setting time of the plaster-of-paris) and placed in layers in the model. This material, however, was immediately eroded away by the flow from three gates. Although it was evident that a stronger mix was required, it was also identified that some of the problem may lie in the fact that the material was cast in thin layers in the model. Each layer was effectively set by the time the next was placed on it. Therefore, the bonding between layers would not have been as strong as that within each layer.

To avoid the thin layer problem the next mix was placed in the hole in heaps. The mix was also strengthened by increasing the amount of plaster-of-paris, as well as increasing the ratio of binder to aggregate. The final mix was 1:1,5:20. After subjecting the material to a flow from three gates for thirty minutes, the resultant scour hole looked quite promising. The hole went down to a depth of 346,2 m asl (Figure 20). Increasing the flow to that from six gates, however, eroded a very large hole, some material being ejected in large blocks. That is, there appeared to be little cohesion between the heaps of material placed during mixing. This was probably due to loose stones rolling down the wet heaps as they were placed and forming a boundary layer of loose material between each heap. The fact that large blocks were found downstream also suggests that the mix may have been too strong.

An improved casting method was needed when placing the material in the model. To this end it was decided to try casting in blocks, using templates to maintain vertical sides on each block. This method would avoid the problems experienced from having thin layers of material and at the same time would not allow loose material to collect on the sides. The ratio of binder to aggregate was also reduced (1:1,5:30) to bring down the strength of the material slightly.

As with the previous test, the scour hole caused by the flow from three gates for thirty minutes was promising (Figure 21). The deepest point was at 336,4 m asl, almost ten metres deeper than the previous test. However, when increasing the flow to six gates, the material was still washed out of the hole.

As the plans of the actual plunge pool had now arrived, it was decided at this point to extend the model downstream to beyond the hydroelectric tailrace outlets. This would allow the complete river topography to be included in the model which should help ensure that flow conditions are reproduced realistically. There had been some concern about the effect that the proximity of the tailwater gate may have on the flow conditions at the downstream end of the hole (Alam 1994 and others).

The first mix to be tried in the new model had the same strength binder as the previous two tests, but with a binder to aggregate ratio in between these two, namely 1:1,5:24. The difference in results, primarily due to the better flow conditions in the model were dramatic. Even after one hour, with three gates open, very little scour had occurred. Thus, where this mix had been too weak in the previous tests, it was now too strong. The scour that did occur, however, was along the joints of the blocks in the direction of the flow, showing that despite the effort put into the casting method, the matrix was still not totally homogeneous.

As it was not feasible to mix the entire batch of material in one go and thereby ensure homogeneity, the block casting method was modified to that of bars, placed perpendicular to the flow direction. It was thought that the weaker joints would have less effect on the scour outcome if they were across the flow rather than in line with it. To reduce the strength of the material, the ratio of binder to aggregate was reduced again, giving a new mix of 1:1,5:28.

The test results were very promising in that a reasonably shaped scour hole was formed from the flow of three gates. The scour hole extended down to 332,2 m asl, the deepest achieved with three gates (Figure 22). Further scouring, still with only three gates increased this depth to a level of 326,6 m asl, but the hole was too large in plan, particularly at the downstream end. Continuing the test with six gates and then with three gates again, only increased the depth of the hole by 5,6 m to 321,0 m asl (Figure 23). The size of the hole was, however, now far too large in the longitudinal direction.

The previous test was repeated with the ratio of binder to aggregate increased slightly to give the same mix used previously when the material was cast in blocks. (1:1,5:24). It was thought that this might restrict the scour hole size. However, it was found that the scour still initiated at the joints of the bars (Figure 24) and this could not be taken as a true representation of what would happen in reality.

It therefore had to be conceded that despite the advantages of using plaster-of-paris, the difficulties of mixing and placing it, made it impractical as a modelling material.

6.5 Aggregate With Cement Based Binder

6.5.1 Jet tests on samples

Following the work of Johnson (1977) and Gerodetti (1982), it was decided to test binders based on a mix of hydrated lime and ordinary portland cement. Some twenty-seven samples of nine different mixes were made up so that an idea of the most appropriate mix could be gauged. A jet, with a nozzle diameter of 4 mm, discharging at 0,28 l/s was passed across the samples at a speed of 0,011 m/s. At the end of each run the number of stones loosened by the jet were counted and removed. The loss of binding material on the surface of the rocks was also recorded after the first run. The results of these tests are shown in Table XV.

Of the twenty-seven samples tested, only the weakest mixes showed significant scouring when subjected to two passes of the jet (Samples 4 to 9). It was decided to start the testing in the Kariba Dam model with the weakest sample mix, namely 1:0,02:7,2. This was similar to the mix recommended by Johnson in his original work (Section 2.5).

In some instances, large differences were picked up between samples of the same mix, as can be seen from Samples 15 and 18.

TABLE XV : JET TESTS ON SAMPLES

SAMPLE	MIX RATIOS			APPROX. % VOIDS FILLED	1st RUN LOOSE STONES	LOSS OF SURFACE BINDER	2nd RUN LOOSE STONES
	LIME	CEMENT	STONE				
1	1	0,02	6,0	85	3	50	8
2	1	0,02	6,0	85	4	50	9
3	1	0,02	6,0	85	5	50	10
4	1	0,02	6,6	77	10	70	32
5	1	0,02	6,6	77	15	70	38
6	1	0,02	6,6	77	10	70	32
7	1	0,02	7,2	71	8	100	45
8	1	0,02	7,2	71	8	100	42
9	1	0,02	7,2	71	4	100	40
10	1	0,04	6,1	85	13	50	23
11	1	0,04	6,1	85	13	70	43
12	1	0,04	6,1	85	14	70	36
13	1	0,04	6,7	77	10	60	27
14	1	0,04	6,7	77	3	60	17
15	1	0,04	6,7	77	0	5	0
16	1	0,04	7,3	71	2	25	12
17	1	0,04	7,3	71	2	25	12
18	1	0,04	7,3	71	1	25	4
19	1	0,10	6,4	85	0	5	1
20	1	0,10	6,4	85	0	0	0
21	1	0,10	6,4	85	0	5	0
22	1	0,10	7,1	77	0	5	1
23	1	0,10	7,1	77	0	15	1
24	1	0,10	7,1	77	0	0	1
25	1	0,10	7,7	71	0	0	0
26	1	0,10	7,7	71	0	5	0
27	1	0,10	7,7	71	0	5	0

6.5.2 Tests in the Kariba Dam model

The first test used a mix ratio lime:cement:aggregate of 1:0,02:7,2. After a flow from three gates for only three minutes, the scour hole appeared to be forming well. At this point the scour hole had already reached a level of 333,9 m asl. However, further scouring enlarged the hole in the downstream direction rather than in depth. After a total of thirty-nine minutes with three gates open, the hole had only deepened by a further 5 m (Figure 25). On inspection, it appeared that the material was too weak to maintain the steep sides expected in the hole, despite the indications of the samples. With the flatter sloped sides, the energy dissipation within the tailwater cushion was greater and hence the scour potential was greatly reduced.

The test was rerun with the cement content of the mix doubled. This, however, made the binder too strong and no spontaneous erosion occurred, even when the flow was increased to six gates. Scour only progressed after manually initializing the process by forming a small hole in the centre of the bed (about 300 mm in diameter and 150 mm deep). The resultant scour hole was well shaped in plan but limited in depth (Figure 26). This was thought to have been in part due to the fact that some of the loose material remained in the hole preventing further scouring and possibly also distorting the flow patterns within the hole.

6.6 Coke Nuts With Cement Based Binder

6.6.1 The need for a lighter material

One of the problems picked up from the preceding tests was that the loose material in the scour hole had a significant effect on the ultimate shape of the hole. The tests were carried out using a 19 mm aggregate (representing 1,33 m in prototype) with a specific density of 2,6. That is, according to standard scaling practice, the density of the material was not scaled down in the model. This material tended to collect in the scour hole and form a slope up the downstream side of the hole, at its natural angle of repose of about 30°. Thus the steep sides found in the prototype scour hole, and reproduced quite well on the left and right sides of the model scour hole, could not form on the downstream end of the model scour hole. The shallower slope changed the flow characteristics in the hole significantly, such that the impinging jet tended to flow through the hole rather than impinge on the bed. The result was a longer hole with a relatively flat downstream slope and subsequently less scour depth than experienced in prototype.

As the problem with the downstream slope seemed to be hinged on the water's ability to remove the loose material from the scour hole, it was decided to try testing with an aggregate that was considerably lighter than the granite used previously. It was felt that this was justified, in that the scour process in the model relied on the removal of the aggregate from the binder matrix rather than a material movement process.

6.6.2 Coke nuts

The material chosen was "coke nuts", an anthracite that has had its volatile gases removed by heating. In theory the water should be able to lift this material, once freed from the binder matrix, out of the hole and thereby negate any influence of the loose material on the scour hole formation.

The coke nuts were supplied in bulk by the local company Cape Gas, Cape Town. These were then sieved to keep only the pieces within the range 20 mm to 25 mm for the tests. The nuts themselves were angular, tending towards cubic in shape. Due to their nature, the coke nuts were porous and absorbed some water when tested in the model. As the coke nuts were always saturated for some time before testing, as well as throughout the duration of the tests, this saturated state was taken as *normal.* The coke nuts were found to vary in specific density, having a mean of 1,6 ± 0,2 in the saturated condition (that is, in the state they would be in during a test). This was well below the specific density of 2,6 of the aggregate used in previous tests.

When mixed with the cement based binder the final matrix was fairly porous with the coke nuts tending to be almost completely covered in a thin layer of the binder. Although this would have added to their mass, it did not appear to hinder the water's ability to expel them from the scour hole and by the time the coke nuts had been washed downstream, most of the binder had been removed. It is not known how much of this abrasion took place before the material was ejected from the scour hole.

6.6.3 Tests in the Kariba Dam model

The first test to be run with the coke nuts in place of the granite aggregate made use of the same mix ratio of lime, cement and aggregate as the previous test, namely 1:0,04:7,2 with a water content of 0,26 units. This was placed in the model and allowed to set for four days before testing. A flow from three gates was then passed through the model for only four minutes. During this time the scour hole reached a level of 343,4 m asl. After twenty-one minutes the scour had deepened by 9,1 m, but thereafter remained constant despite running the model for over 1,5 hours (Figure 27). Increasing the number of gates increased the scour depth slightly, the final level reached being 326,2 m asl. Although this was still approximately 20 m too shallow, the formation of the shape of the scour hole was good.

From Figure 27, it can be seen that the up and downstream slopes of the scour hole are not as steep of those found at Kariba (Figure 2). To improve on this, it was decided to run the next test with a slightly stronger binder mix. It was thought that if the correct side slopes could be achieved, the confinement effect of the hole would take care of the scour depth. Simultaneously, the ratio of binder to aggregate (coke nuts) was increased, whilst the ratio of cement to water was decreased slightly, giving a design mix of 1:0,06:6,0 with a water content of 0,3 units. Despite the small increments in the dry materials, the increase in strength of the material was enough to prevent any scour at all. In total, the material was subjected to one hour with a flow from three gates followed by thirty-five minutes from all six gates. Attempts to manually initiate scour by forming a small hole in the centre of the bed also failed.

After close inspection of the material it was decided to rerun the test with the same strength binder and water content, but with the binder to aggregate ratio similar to that of the first test in this series, that is a mix of 1:0,06:7,2 (water content 0,3 units).

When subjected to the flow of three gates for only five minutes, the bed material washed out (Figure 28). It was immediately evident that something was wrong with the mix, it appeared that an error had been made in the amount of water used as the binder had flowed, after it was placed in the model. The test was repeated with one slight modification, the water content was reduced further to 0,26 units.

The binder in this material also turned out to be too strong, with only a small scour hole forming after 100 minutes at three gates. Increasing the flow to six gates did not increase the scour significantly. The final depth of the scour hole was only 351,1 m asl (Figure 29).

7. SUMMARY OF FINDINGS AND CONCLUSIONS

7.1 Findings

7.1.1 Prediction formulae

The comparisons of scour predictions with model and actual scour data have shown that none of the formulae put forward to date, work when applied to differing sites. Furthermore, Mason found that the formulae that gave the best results for model studies were not the same ones that gave the best results for prototype studies. It appears that some degree of accuracy may be obtained only in cases where the site conditions are very similar to those from where the proposed formula was derived in the first place. In his work to rationalise the proposed scour prediction formulae, Mason (1985) came up with two distinct formulae, one that best suited the available model data and a general formula based on model and prototype data. The latter of these two formulae gave particularly poor results. This could be accounted for, at least in part, by the fact that the prototype scour data does not necessarily represent the **ultimate scour depth**.

Yildiz and Üzücek (1994), reporting on scour predictions for dams in Turkey, stated that there is a general lack of suitable scour data available in the literature for the necessary research work to be carried out on scour prediction formulae. The writer does not deny the value of such data, but warns that care must be taken when using it to derive prediction formulae. Very rarely, if ever, are we certain that the ultimate scour depth has been achieved at a given site, due mainly to the time dependency of the mechanism of scour. This factor appears to have been neglected in most of the predictive formulae derivations put forward to date. A fact that could lead to under estimation of actual ultimate scour depths if these formulae are later used in practise.

This leads to the question of the usefulness of prototype data. From the facts stated above, it would appear that such data would be meaningless unless;

- it is known that the ultimate scour depth had been reached

or

- the rate of erosion is accounted for in the prediction formulae.

The first of these is in most cases, if not all, not known with any certainty. Therefore, to use available prototype data to produce working scour prediction formulae, the effect of time must be taken into account. This in itself, presents considerable problems, unless very detailed data are available, which is not normally the case.

It seems, therefore, that a reasonably accurate generic scour prediction formula is not possible. The large number of variables at each site, the unavailability of prototype ultimate scour depth data and the lack of time related scour data, make the feasibility of such a formula negligible.

If this is true, then the engineer must turn to other methods of determining the expected scour depths below a dam overflow spillway or flipbucket. Two possibilities are available. The first is to consult some form of rating index for the given site conditions. To the writer's knowledge no working model of such an index is available, however, there has been some work in this direction recently, based on the Kirsten Rippability Index. The Kirsten Index, originally put forward for use in earthworks, gives a relative "strength" value to the bed rock at the site, a factor that should be related to the depth of scour. At least two authors have tried fitting known data to this index, without modifying it at all, with some success. Of course such a system cannot predict an actual depth of scour, only an indication of what scour may occur, for example; no scour, slight, moderate and extensive. However, with some subjective judgement an experienced engineer would be able to derive at least an indication of the expected scour depth. This method is promising, but still requires much work before it can be used reliably in the field.

The second method of determining the scour depth below a dam overflow spillway or flipbucket, is to use a scale model. Despite certain shortcomings, this is still the most reliable method available to date. Further details of using models are discussed in Section 7.1.2.

7.1.2 Scale modelling

This study has highlighted a number of aspects with regard to using scale models for the prediction of scour depths below a dam overflow spillway or flipbucket.

Probably the most important of these is the effect of **jet confinement**. Whilst it has been known for some time that confining the jet may cause increased scour depth, this study has highlighted just how dramatic that effect can be. In this case, when no confinement was inhibiting the jet after entering the plunge pool, the jet was unable to remove a layer of blocks placed at a given level. However, when an artificial confining hole was constructed, reducing the plan area of the downstream river by about 50 %, the same jet was able to remove the blocks with ease. At only half the original flow, the jet removed all the available blocks, leaving a scour hole almost twice as deep as with no confinement. As the bottom of the hole struck the fixed floor of the model, the ultimate scour depth could not be determined in this case.

The packing density, uniformity, regularity of placement and joint fill material between the blocks are important criteria influencing the ultimate scour depth. As these cannot always be replicated correctly in the model environment, the probability of obtaining reliable results from the use of blocks was limited. Thus, although blocks tended to reproduce the mechanism of scour in the model correctly, this study concentrated on using cohesive binders with aggregate as the bed material. This was easier and quicker to place and gave a homogeneous matrix, which in turn meant that repeatability of results was achievable in the model.

Further to this, it was shown that the **local topography** also plays an important role in determining the shape and therefore depth of the scour hole. It would seem that the flow patterns in the vicinity of the scour hole have a major influence on the scour mechanism. Altering the downstream topography slightly, can influence the flow patterns in and around the scour hole, which in turn significantly alters the potential scour depth. It is therefore, extremely important to construct the model to exacting detail, thereby ensuring the correct flow patterns and therefore accurate scour reproduction. Failure to do this could make the model results meaningless and in the worst scenario, lead to an under-designed system.

Related to this problem is that of the formation of a bar from the eroded material being deposited just downstream of the scour hole. In reality much of this material would erode into finer material that could easily be washed further downstream. As this does not occur in a model, the material is dumped closer to the scour hole and the resultant bar is larger than would form in reality. This in turn alters the flow conditions which have been shown to affect the ultimate scour depth. It is, therefore, necessary to stop the model at regular intervals and reduce the size of the bar forming in front of the scour hole, such that the correct flow conditions are maintained throughout the test.

In a model study investigating the build up of the bar at Kariba Dam (SOGREAH 1962), several tests were run to determine a suitable model material for the investigation. It was found that the scour hole formed during these tests was too large and the hole was finally fixed in mortar to the correct size, to ensure that the flow patterns within the scour hole were similar to the prototype. Using this setup, a suitable non-cohesive material was found that gave bar heights similar to those found in prototype. This material was a mix of 50 % 2 to 9 mm aggregate with 50 % 0,5 to 4 mm sand. It was recognised that a shortcoming of this method was that the erosion rate of the scour hole could be taken into account and its effect on the bar formation was unknown. A material that satisfied the scour hole formation as well as the bar formation was not found during the tests.

This study found that the use of cohesive materials gave deeper scour depths than non-cohesive materials in most cases. This appears to be due to the confinement effect on the jet. With a cohesive material it is possible for the forming scour hole to have steep or vertical sides. Thus a confined hole is formed, that concentrates the jet's energy, ultimately causing deeper scour. On the other hand, in the case of loose bed material, the slopes of the sides of the scour hole are limited to the angle of repose of the material (normally close to 30°). Thus there is very little confinement of the jet, little to no concentration of the energy and therefore a considerably shallower scour hole than could be expected in reality.

The shape of the scour hole was shown to have a large influence on the final scour depth, in that it determines the confinement and hence concentration of the jet. In most of the tests, although the upstream end of the scour hole tended to form as required, the downstream end always had a flatter slope than the prototype. This was primarily due to the aggregate from the loose bed material remaining in the hole and collecting on the downstream end, at its natural angle of repose. This is an unnatural situation, as in the normal course of events, this material would have been broken and eroded into pieces small enough to be removed from the scour hole altogether.

It would be, of course, impractical to stop the model tests regularly and remove this build up of material. Two other solutions can be considered, either using a material that will break down after it is loosened, or using an aggregate that is considerably lighter than the material it represents and will be removed from the model scour hole by the action of the jet. The second of these is obviously the most practical solution and was used in this study (Section 6.6).

The choice of cohesive bed materials is discussed in Section 7.1.3.

7.1.3 Bed material

This study investigated a wide range of cohesive materials to represent the bed material in scale models. The criterion against which the materials were tested was their ability to reproduce the scour hole formed at the Kariba Dam. None of the materials tested gave ideal results, however, one material did behave remarkably better than all the others. This material used a cement based binder with "coke nuts" as the aggregate.

It is thought that the main reason that this material outperformed the others was simply that no loose aggregate collected in the scour hole to distort the natural flow patterns of the plunging jet. During the formation of a scour hole in reality, blocks of bed rock that are lifted out of place by the action of the jet, soon break into pieces small enough to be ejected from the scour hole. Thus the hole itself is kept clear of debris and therefore there is only the actual shape of the hole to influence the flow characteristics of the impinging jet. At Kariba Dam, rock fragments found downstream of the scour hole do not exceed, on average, about 300 mm in diameter. This would translate to a stone of 4,3 mm diameter or a mass of only 0,1 g in the model, which would easily be removed by the jet. In the model, however, the 19 mm granite aggregate used for most of the tests was too heavy for all of it to be removed from the hole. The material that remained tended to collect at the downstream end of the hole, forming a shallow slope at this end. Obviously, this totally distorted the flow patterns within the hole and hence the scour potential of the jet. When using coke nuts as the aggregate, no material remained in the hole, giving a situation closer to that experienced in reality and therefore, a more realistic scour hole.

Four materials were tested for use as binders and these are summarised in Table XVI.

Based on their binding capabilities, only the plaster-of-paris and the cement were found to be viable options. However, the plaster-of-paris had a major draw back in that its setting time was extremely quick (less than five minutes). This led to problems throughout the testing, as the material had to be mixed in small batches, which made achieving a homogeneous matrix very difficult. It had to be concluded that, from a modelling point of view, the plaster-of-paris was impractical and could not be considered.

TABLE XVI : COMPARISON OF BINDING MATERIALS

BINDER	COST	NOTES
Bentonite	Moderate	Tended to be too weak unless completely filling the voids between the aggregate.
Cretestone	Moderate	Tended to break down after long periods of saturation in the model.
Plaster-of-paris	Expensive	Very difficult to obtain homogeneous mix due to the extremely quick setting time.
Cement	Cheap	Slow to set, but easy to work with. Water content critical to strength.

Thus, the most practical binder tested was ordinary portland cement. It should be noted that as the setting of cement is a hydration reaction, the quantity of water in the mix and available throughout the initial setting stages, plays a major role in the final strength of the mortar. This had to be controlled very carefully in the model tests to ensure consistent results. The major draw back of using cement is its long setting time. Ideally to ensure the best results, the mixture should have been allowed to set for about twenty-eight days, at which time the cement has gained most of its strength. However, it is not feasible to only be able to carry out one test a month, so a much shorter period had to be used. In this case a setting period of four days was chosen, which allowed a test per week to be carried out. Unfortunately, during the early stages of setting, the ambient temperature also plays a role in the strength of the material as the temperature accelerates or decelerates the rate of hydration of the cement. This is a problem that would have to be monitored in the laboratory.

Although none of the mixes tested reproduced the Kariba Dam scour hole particularly well, the mix that gave the best results was found to be (by volume);

LIME	CEMENT	COKE NUTS	WATER
1	0,04	7,2	0,26

It is of interest to compare this with the mix put forward by Johnson (1977), that is;

CHALK	CEMENT	AGGREGATE	WATER
1	0,02	6,0	1,6

The major difference between the two mixes is the water to cement ratio. This study proposed a ratio of 1:6, while Johnson used a ratio of 1:74,2 which would have given a very fluid binder.

It was found during the testing that if the binder was too fluid, it tended to flow to the bottom of the bed material, leaving a nonhomogeneous matrix. Johnson's binder would also have been considerably weaker than that of this study. Besides having only 50 % of the cement content, the very high water content would have lead to a weaker product. This may have been compensated for, to some extent, by the increased binder to aggregate ratio, that is, more of the voids between the aggregate will have been filled. However, as found in this study, filling the voids tends to change the mechanics of erosion such that the build up of hydrostatic pressure within the matrix does not occur in the model. Thus Johnson's mix would not be suitable for the reproduction of scour by a falling jet.

It was found that the test scouring of the samples was not always representative of what occurred in the model. Although this may have been due to a number of reasons, it is thought that a major contribution was the size of the jet used on the samples, as opposed to that in the model. The smaller jet may have allowed the pore pressures within the matrix to dissipate before they built up enough to cause any damage to the samples. Thus it would have appeared that the material was stronger than it actually was.

7.2 Conclusions

From the results of this study, the following conclusions can be drawn:-

- Recent formulae put forward to predict the depth of scour are no more accurate than those originally proposed in the 1930's. In general, scour prediction formulae tend to be unreliable and should be used with caution.

- It would appear that physical model tests are still the most reliable method of determining the expected depth of scour below a dam overflow spillway or flipbucket.

- The confinement effect of the scour hole has a concentrating effect on the plunging jet that results in a much deeper scour hole. If this confinement is not correctly reproduced in the model, the depth of scour hole produced may be shallower than that occurring in reality, which could lead to an under-designed system.

- The material used to represent the bed rock in the model must be able to reproduce the near vertical sides of the scour hole achieved in nature (for example, by incorporating a cohesive binder). Failure to do so reduces the confinement effect on the jet, resulting in shallower scour holes than will actually occur.

- As the flow patterns in and around the scour hole play a major role in determining the ultimate scour depth, it is imperative that the downstream topography of the river and dam appurtenant structures are modelled in detail.

- Due to the uncertainties involved in trying to represent the natural bedrock in the model, some subjective judgement may be required by the engineer, to assess if the modelled scour hole is realistic.

- The scour depths obtained from model studies should only be regarded as an indication of the probable scour depths, not absolute values.

- The material used to represent the natural bedrock gave the best results when a very light aggregate was used in the mix (in this case coke nuts). This ensured that the loosened aggregate could be ejected from the model scour hole (as it would be in reality) and did not interfere with the flow patterns, and hence scour development, in the hole.

- The most practical binder found, of those tested, was a mix of ordinary portland cement and hydrated lime. The only draw back of using cement was the setting time, for which a period of four days was used as standard in these tests.

- Although no bed material was found that gave identical scour to that found at Kariba, the mix that gave the best results was (by volume);

LIME	CEMENT	COKE NUTS	WATER
1	0,04	7,2	0,26

7.3 Recommendations Regarding Further Research

Although this study has found a potential material for use in models to reproduce the scour hole below a dam overflow spillway or flip bucket, it has also brought to light a number of other aspects. It is recommended that these be investigated in further studies.

- The concept of linking the Kirsten Number (modified or not) with the depth of scour at existing sites may lead to a very simple scour depth indicator for future proposed sites. Although it is likely that adequate suitable data may not be available at this stage to achieve a complete analysis, this approach has merit for the future.

- The material found for model testing in this study needs to be refined further to determine the best possible mix. Consideration could be given to looking into a quicker setting cohesive binder, so that long delays between tests can be avoided. This may also help to alleviate the problem of the actual binder strength at the time of testing, which will ensure similarity between tests.

- A second dam needs to be modelled to check that the test material can be used at differing sites. It is, however, foreseen that finding a suitable site with a significant scour hole and adequate data on the scour hole formation may be a problem.

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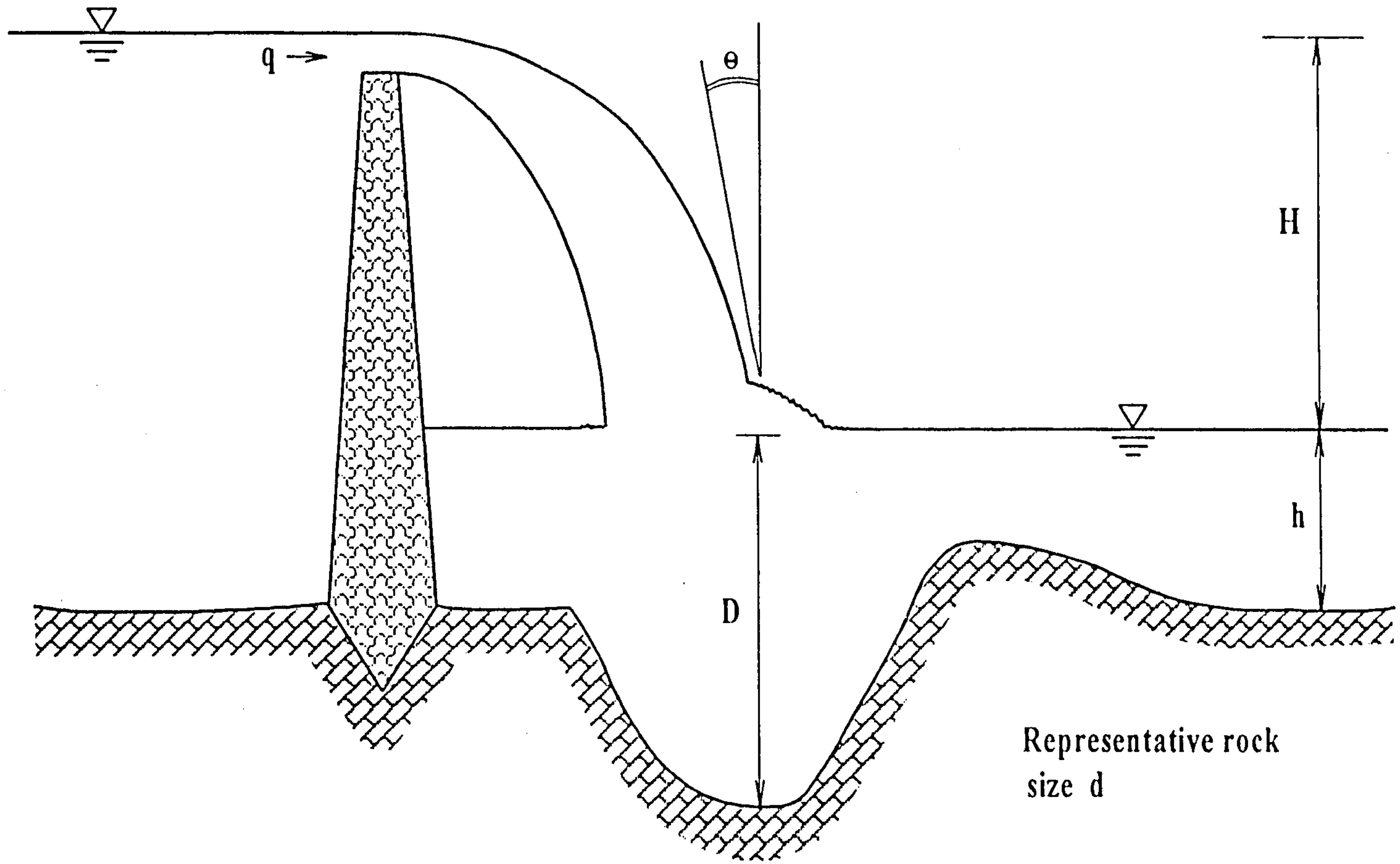
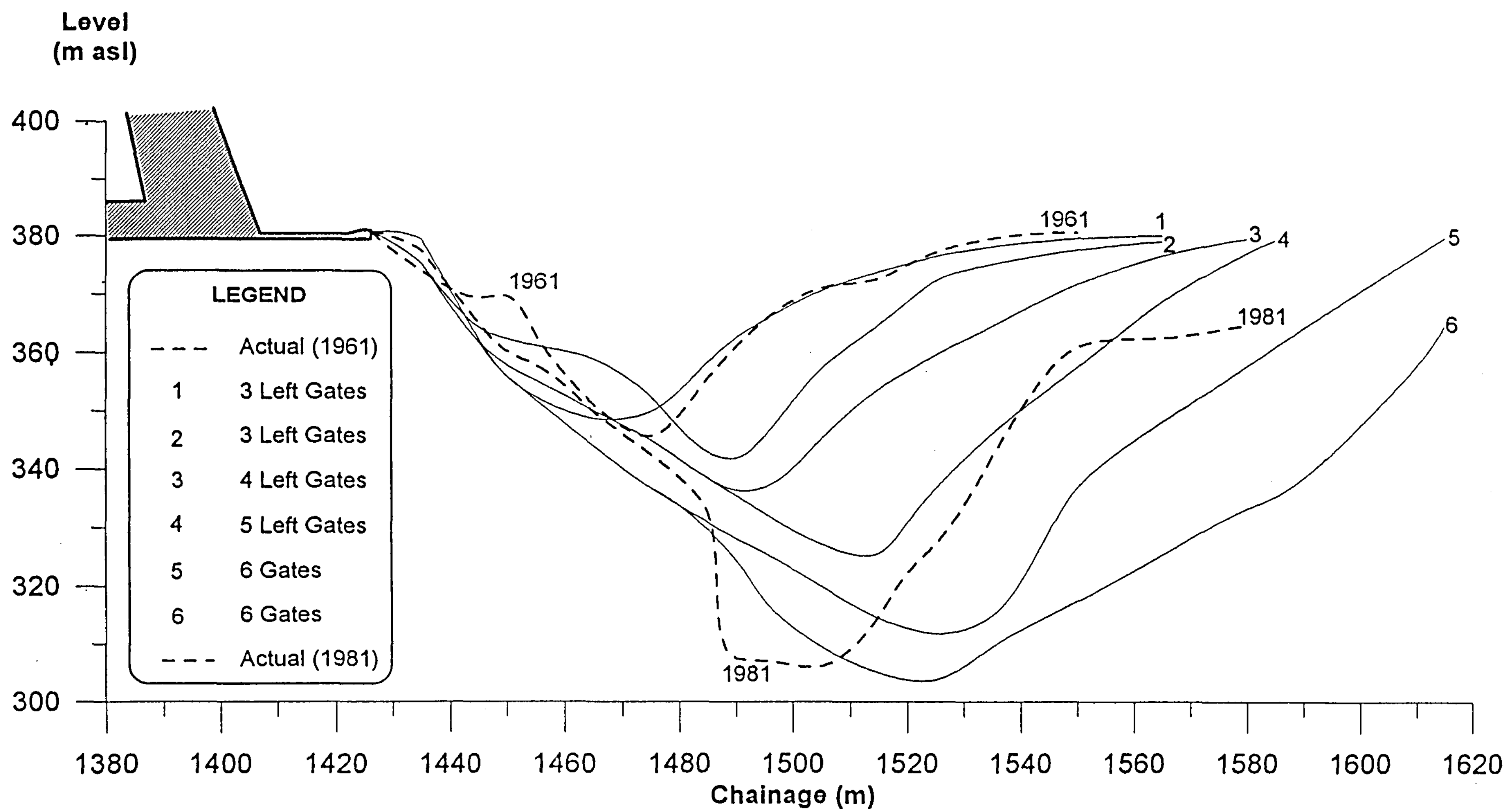


PLUNGE POOL SCOUR

DEFINITION OF SYMBOLS

FIGURE 1

CSIR - EMATEK

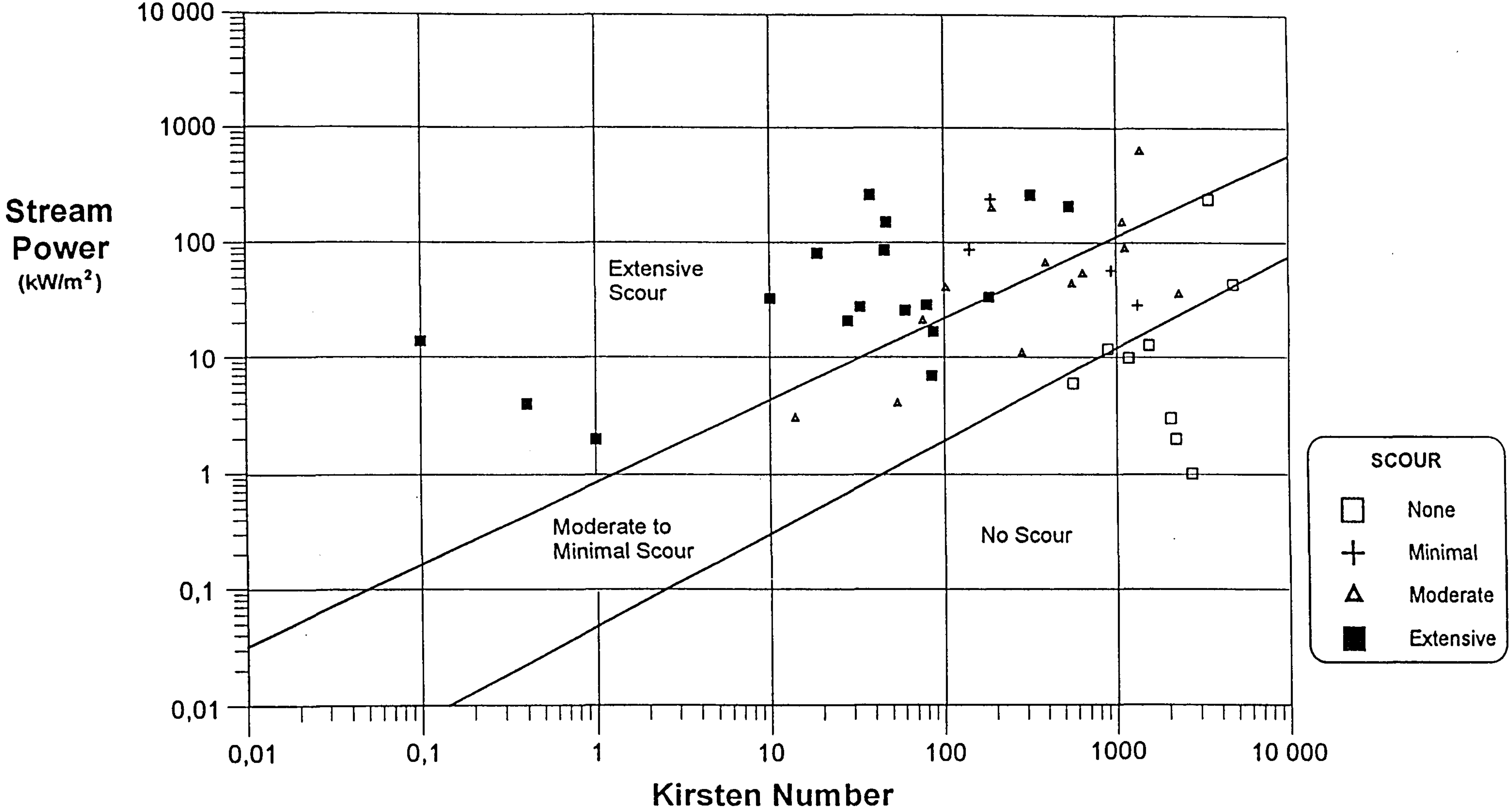


PLUNGE POOL SCOUR

MODEL SCOUR TESTS FOR KARIBA DAM, 1961 (SOGREAH)

FIGURE 2

CSIR - EMATEK

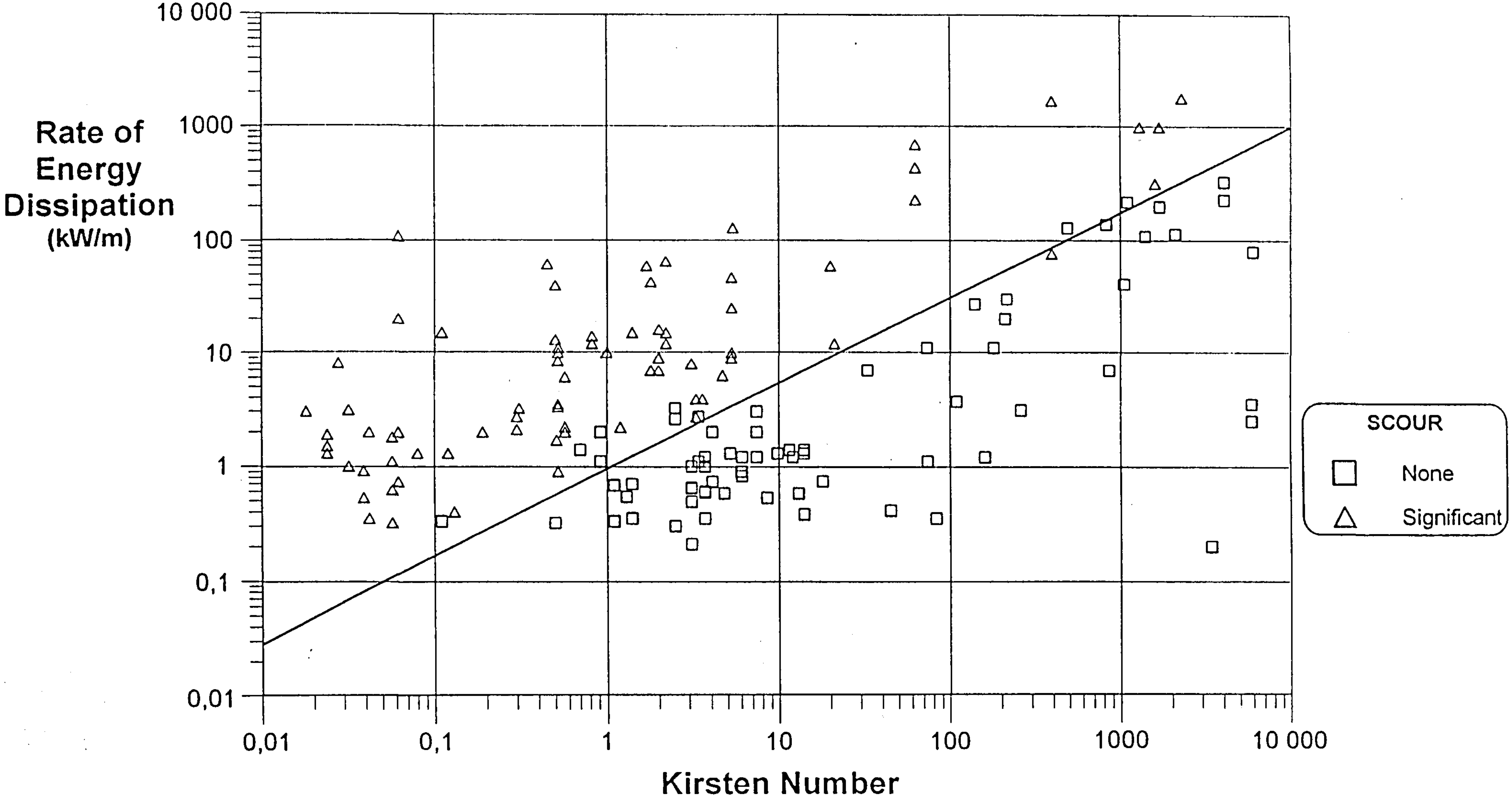


PLUNGE POOL SCOUR

STREAM POWER VERSUS KIRSTEN NUMBER (after van Schalkwyk)

FIGURE 3

CSIR - EMATEK

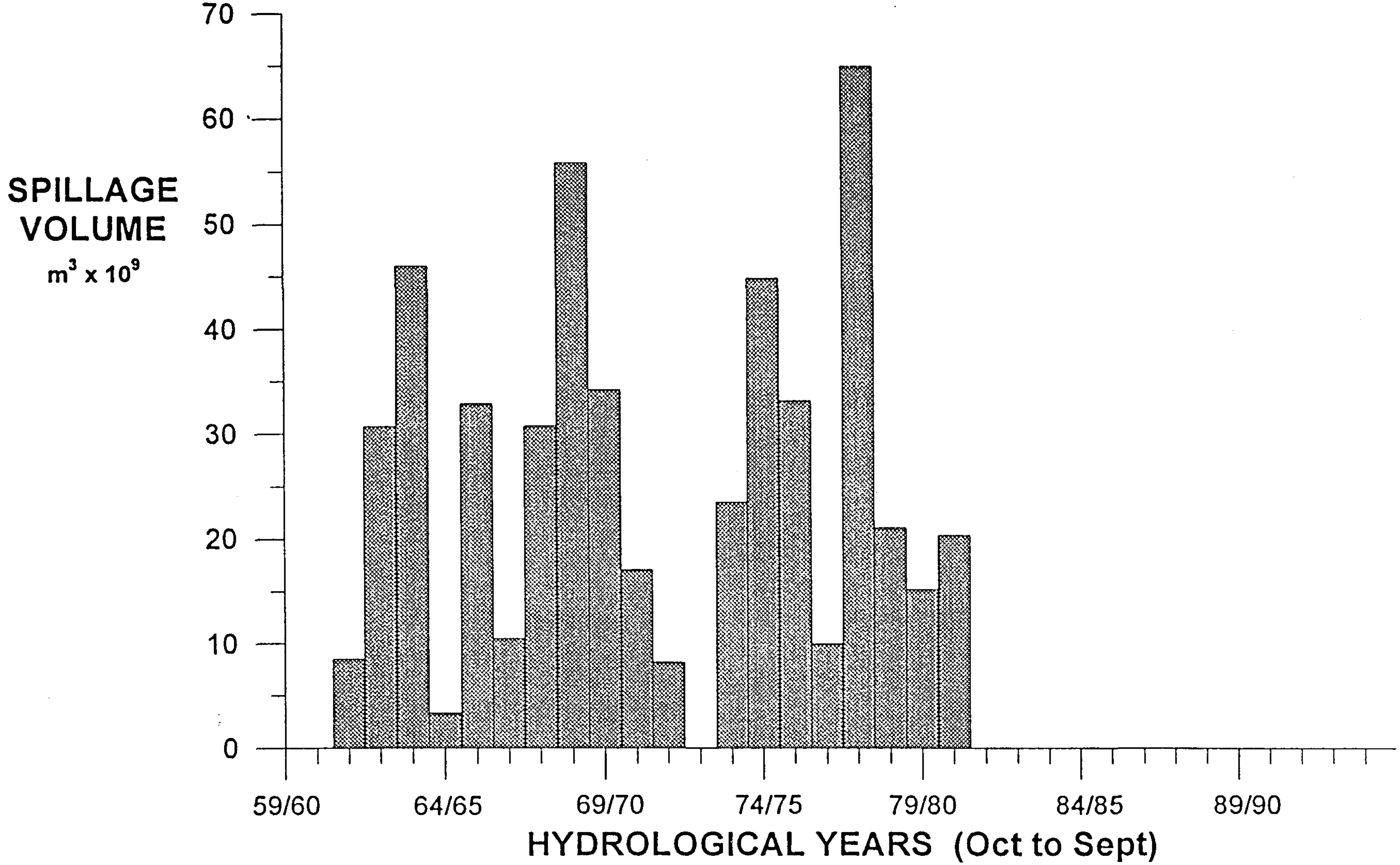


PLUNGE POOL SCOUR

ENERGY DISSIPATION VERSUS KIRSTEN NUMBER (after Annandale)

FIGURE 4

CSIR - EMATEK

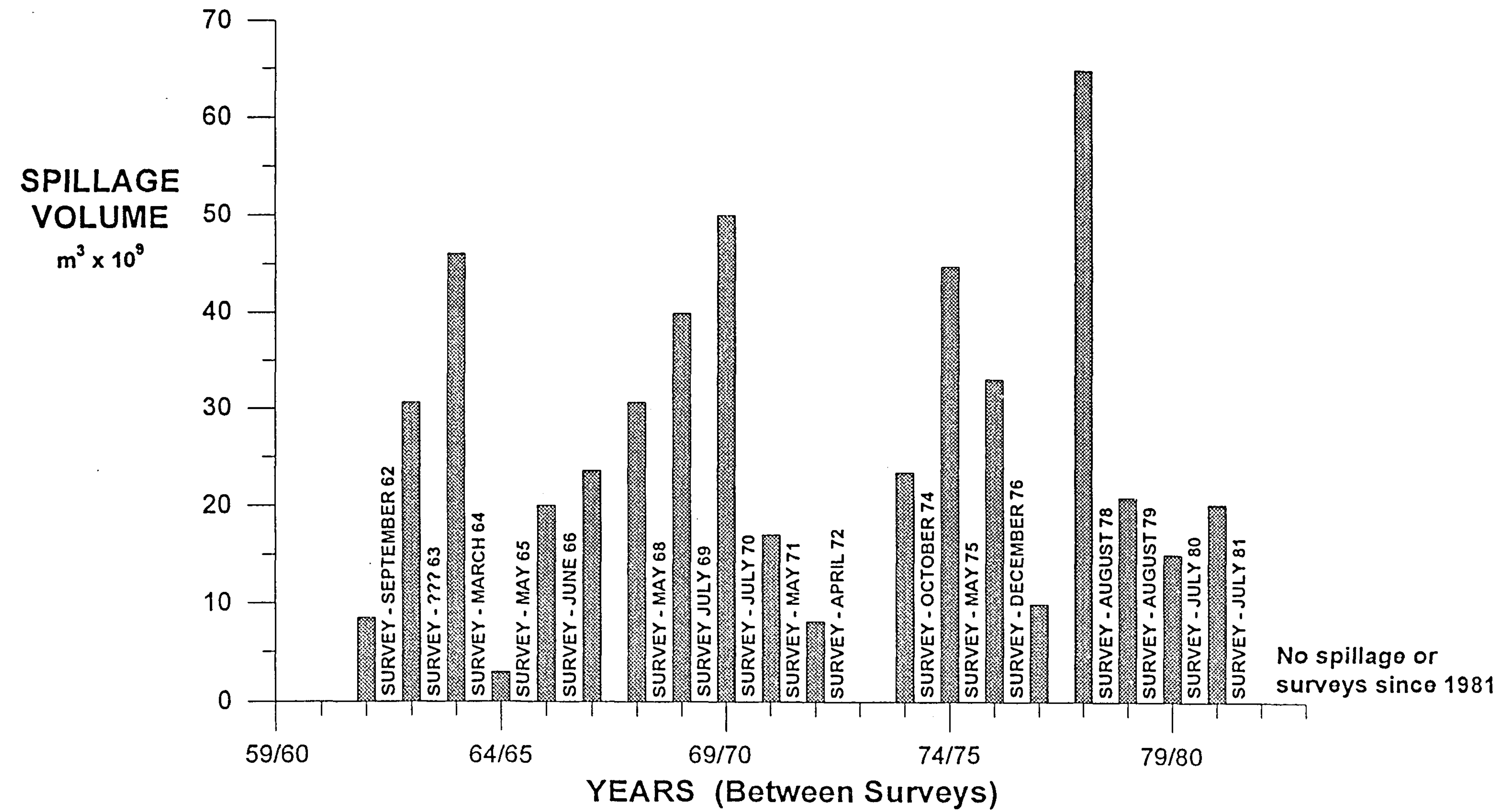


PLUNGE POOL SCOUR

ANNUAL SPILLAGE FROM KARIBA DAM (1961/62 TO 1992/93)

FIGURE
5

CSIR - EMATEK

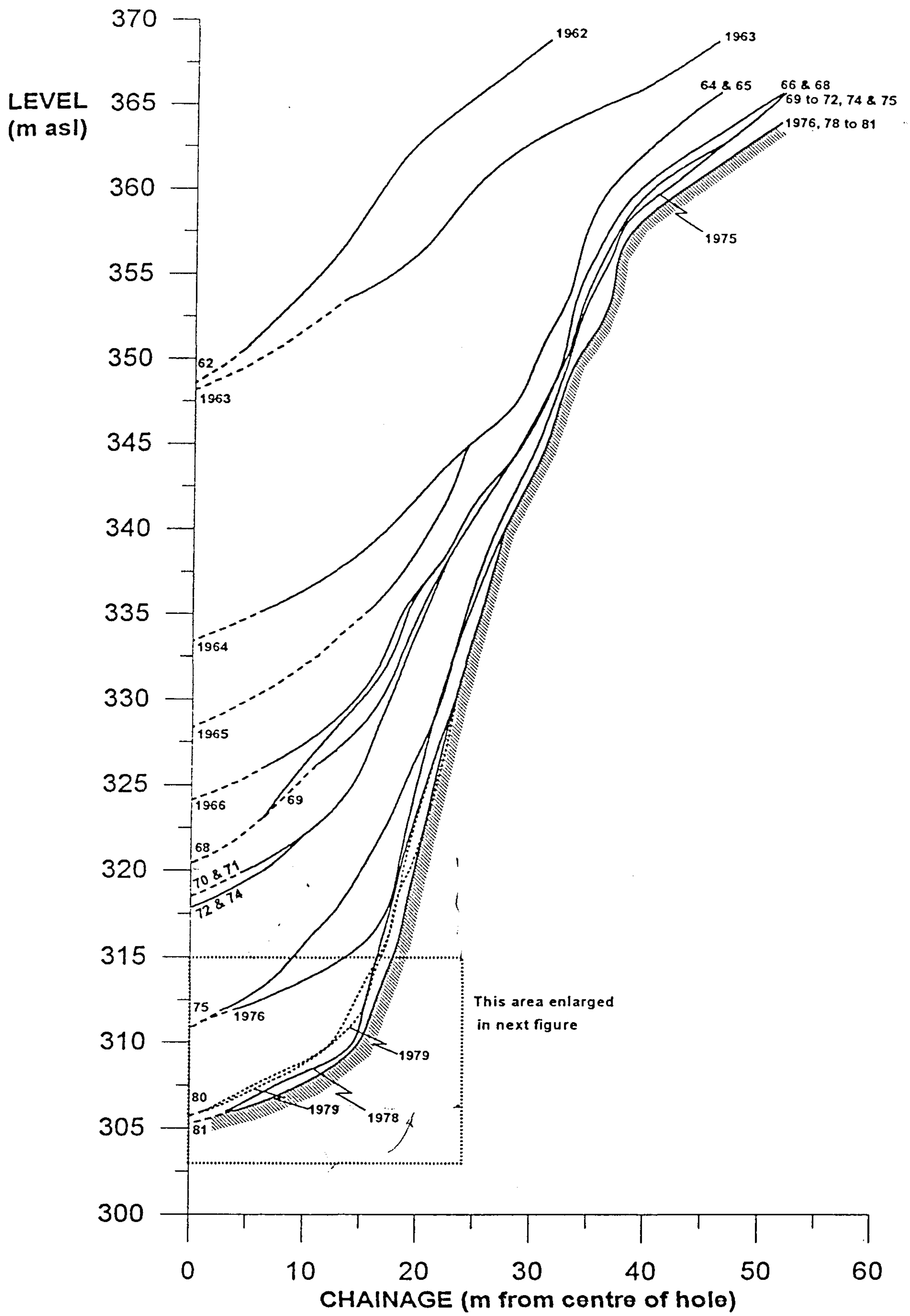


PLUNGE POOL SCOUR

SPILLAGE FROM KARIBA DAM BETWEEN SURVEYS (1961/62 to 1992/93)

FIGURE 6

CSIR - EMATEK

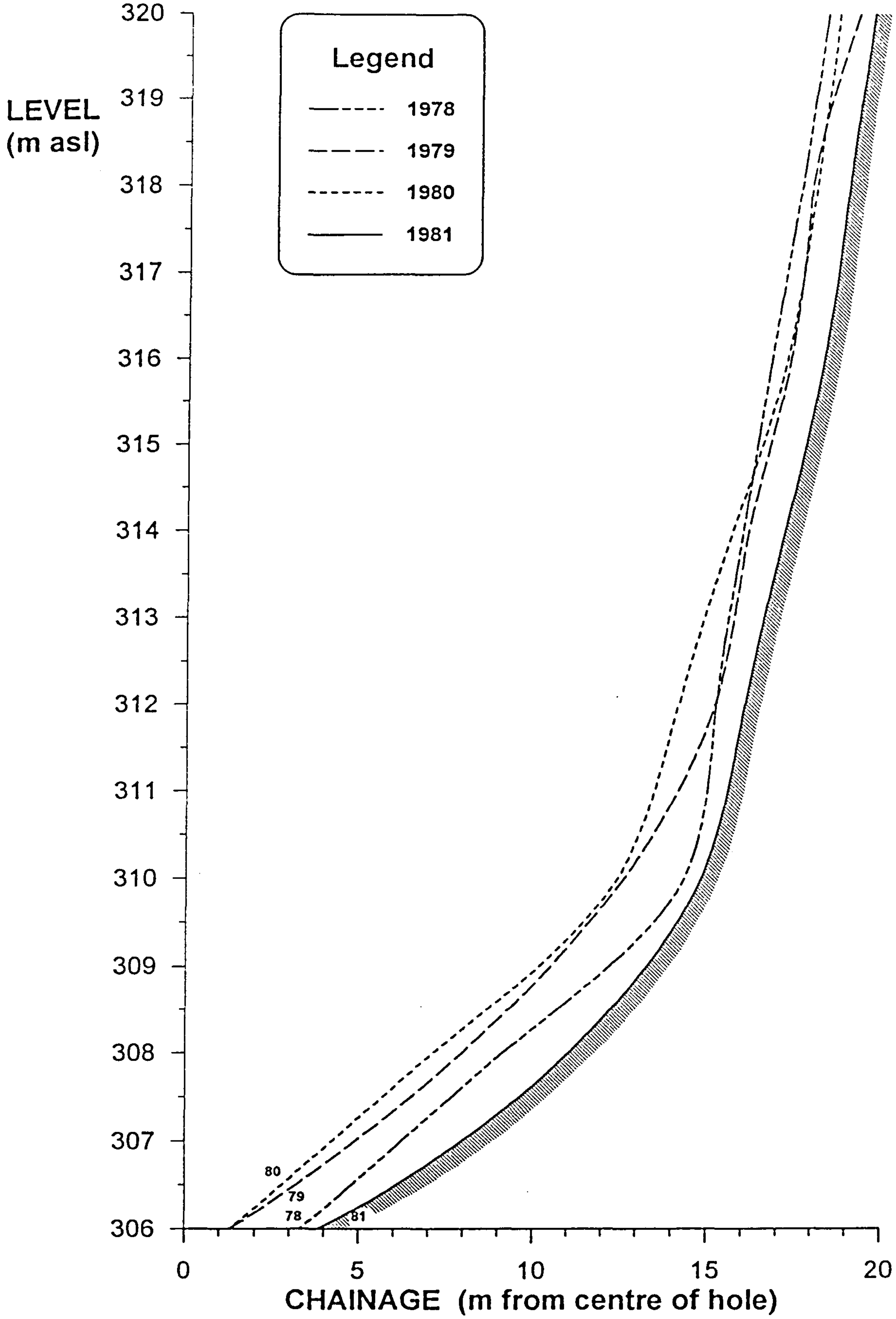


PLUNGE POOL SCOUR

KARIBA SCOUR HOLE PROGRESSION (after Mason)

FIGURE 7

CSIR - EMATEK

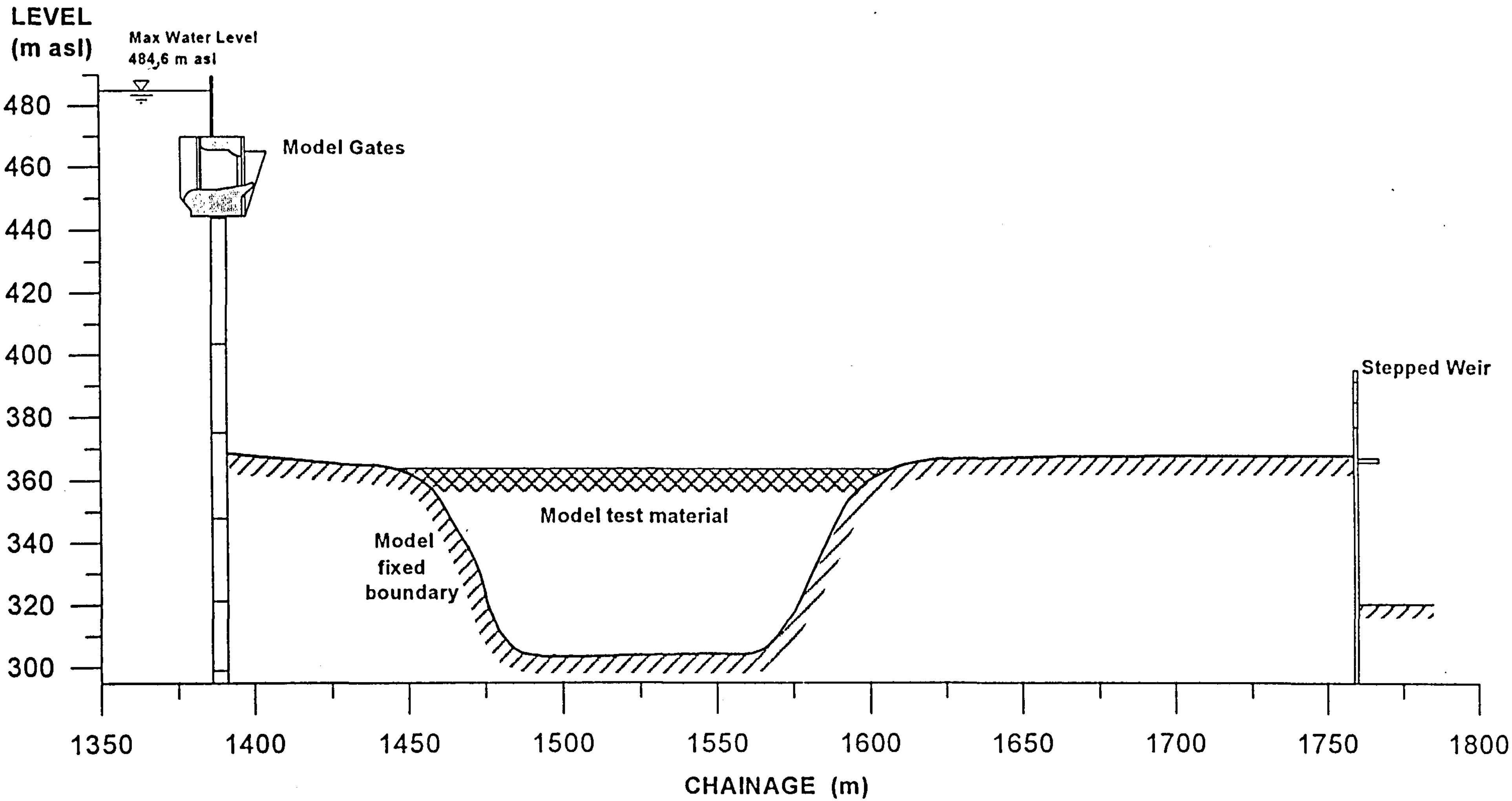


PLUNGE POOL SCOUR

KARIBA SCOUR HOLE PROGRESSION (enlarged section)

FIGURE 8

CSIR - EMATEK

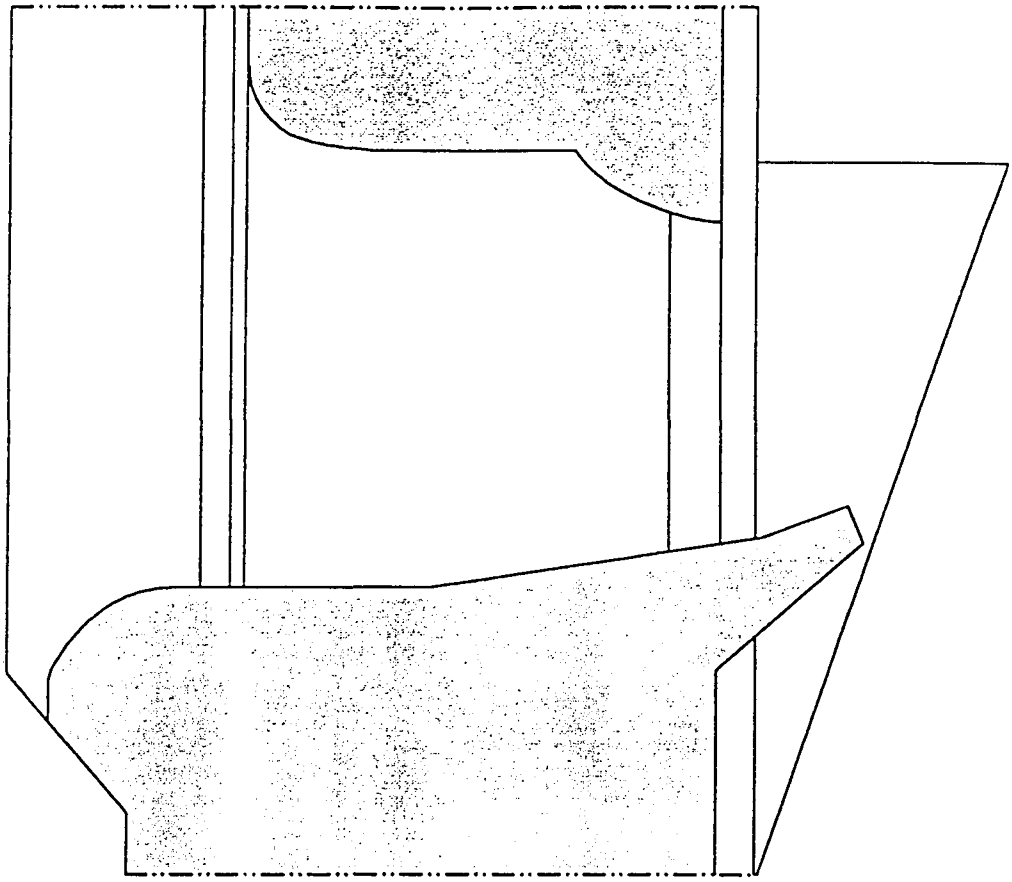


PLUNGE POOL SCOUR

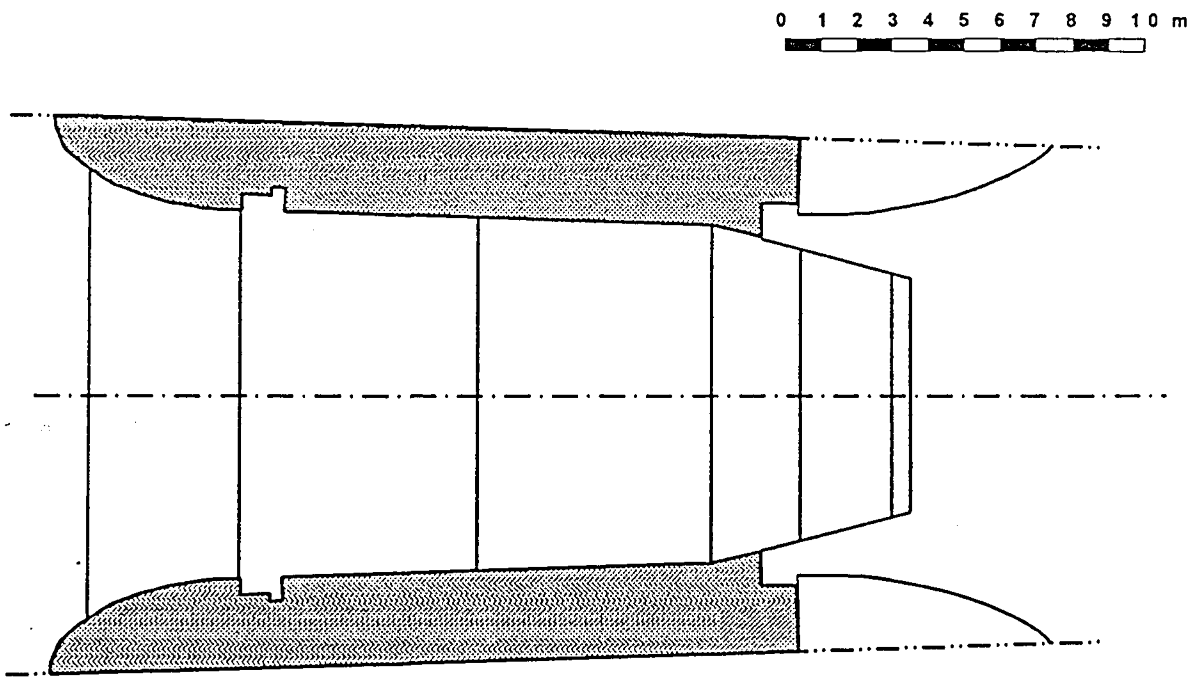
LONGITUDINAL SECTION THROUGH THE KARIBA DAM MODEL

FIGURE 9

CSIR - EMATEK



VERTICAL SECTION



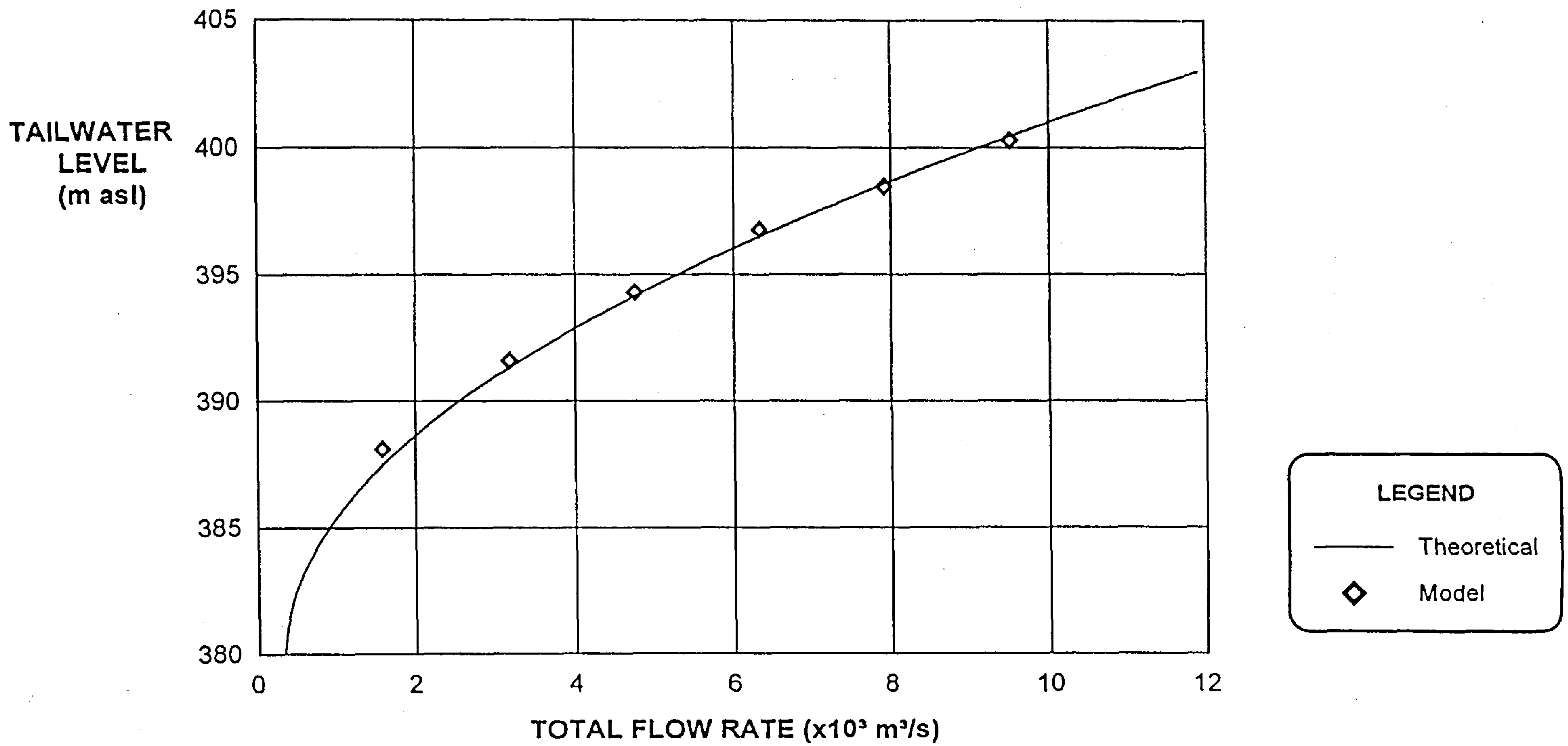
PLAN SECTION

PLUNGE POOL SCOUR

KARIBA DAM - SECTIONS THROUGH ONE GATE

FIGURE 10

CSIR - EMATEK

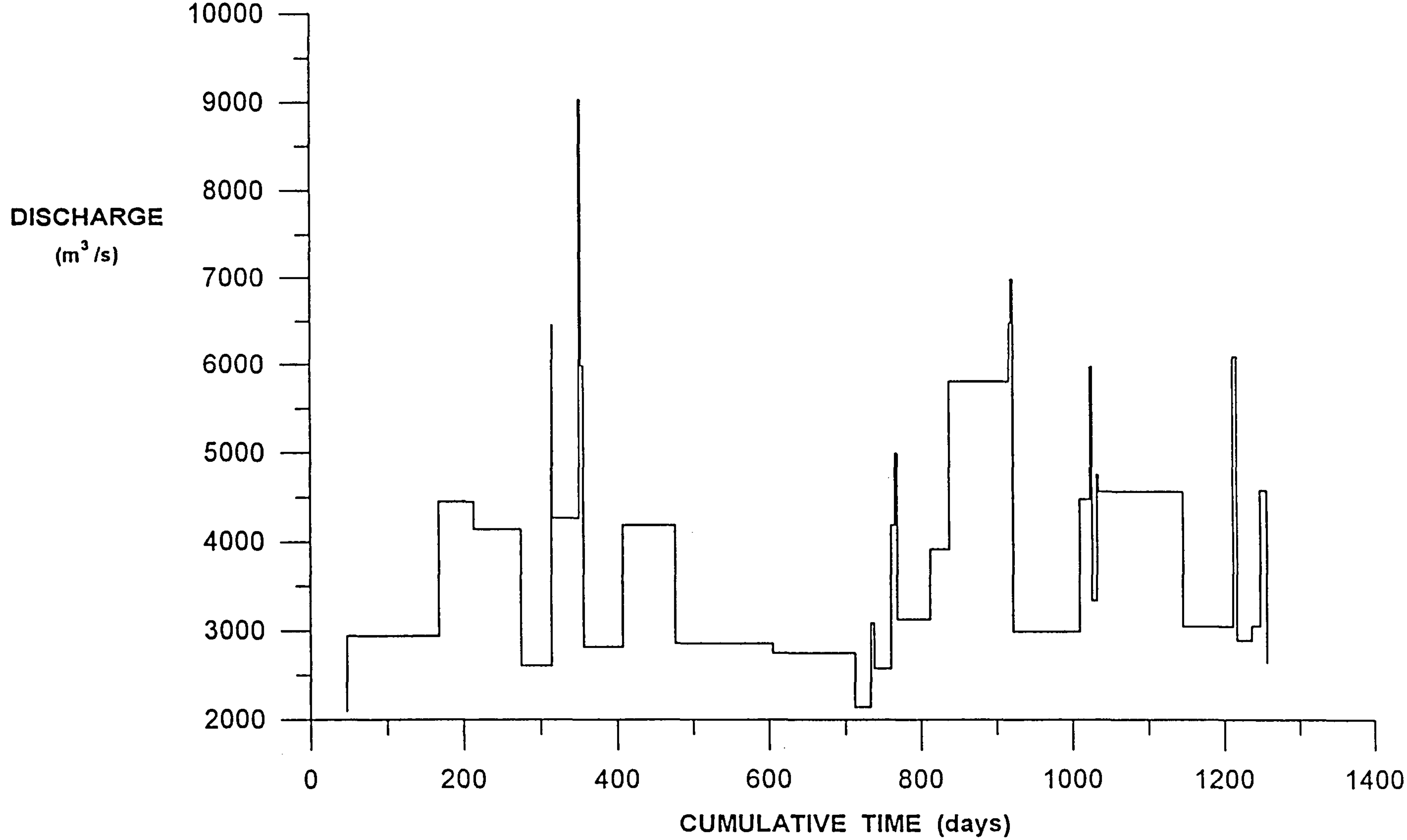


PLUNGE POOL SCOUR

KARIBA DAM TAILWATER RATING CURVE

FIGURE 11

CSIR - EMATEK

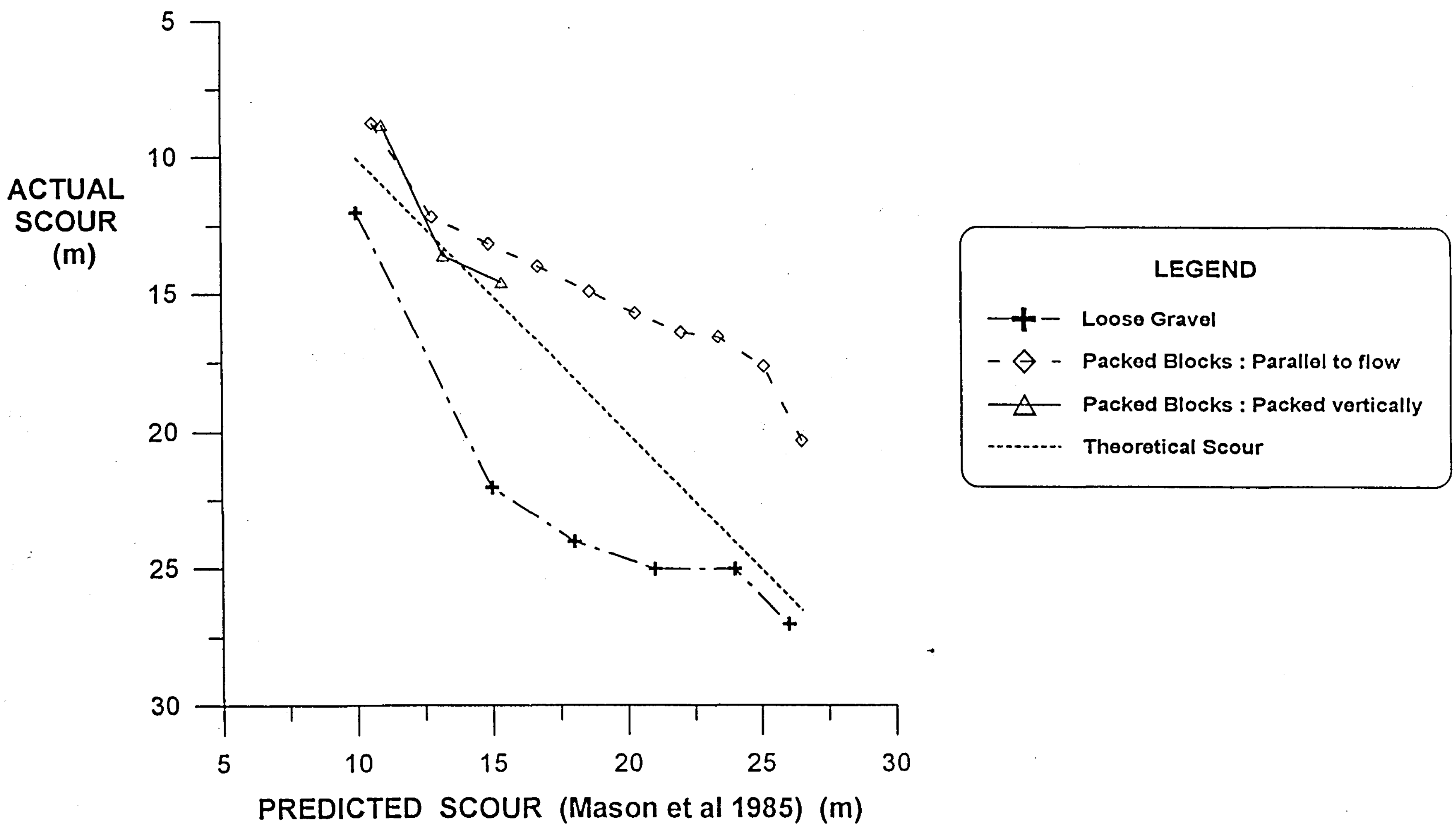


PLUNGE POOL SCOUR

SIMPLIFIED HYDROGRAPH FOR KARIBA DAM

FIGURE 12

CSIR - EMATEK

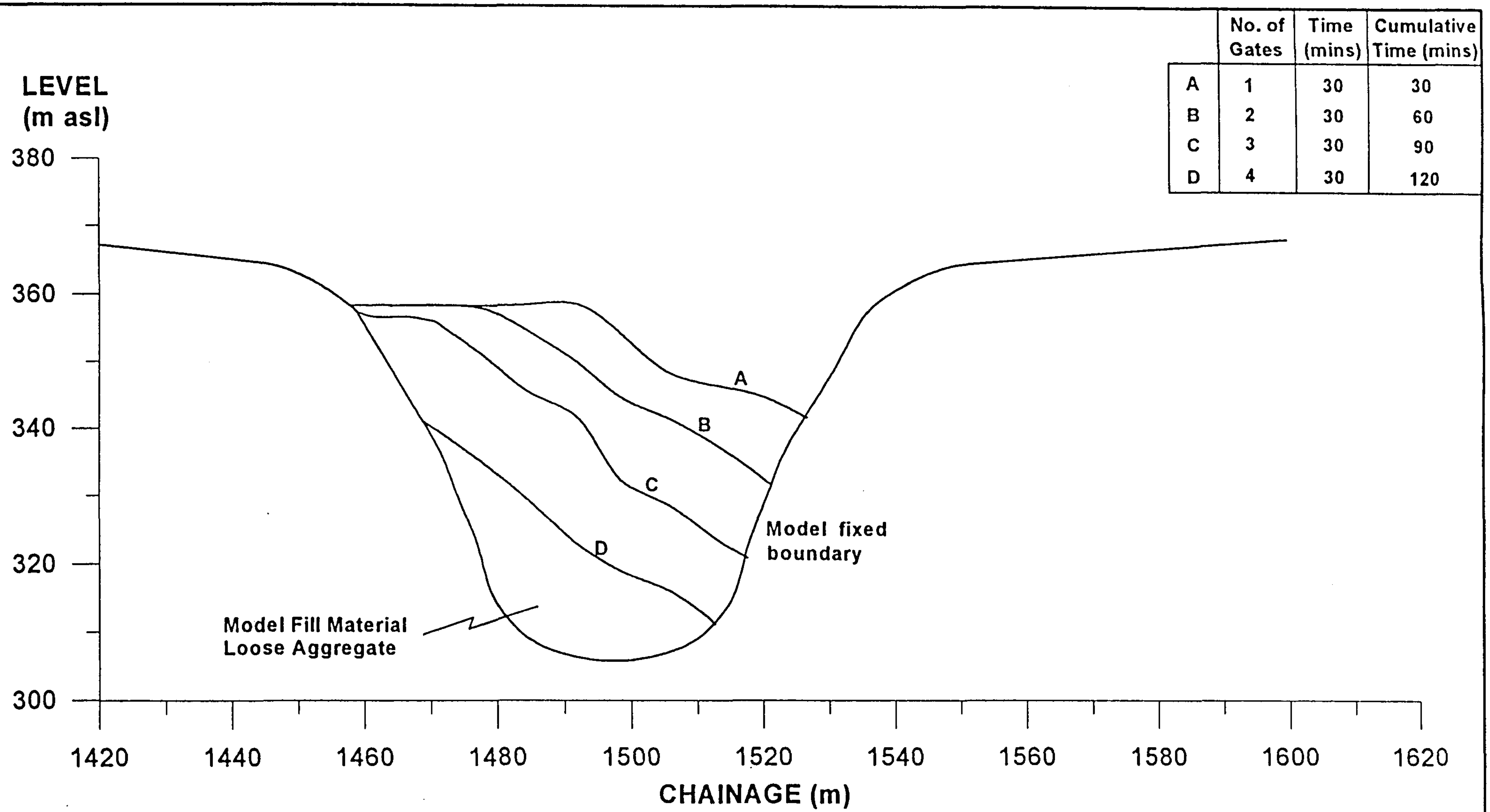


PLUNGE POOL SCOUR

ACTUAL MODEL SCOUR VERSUS PREDICTED SCOUR (Mason et al 1985)

FIGURE 13

CSIR - EMATEK

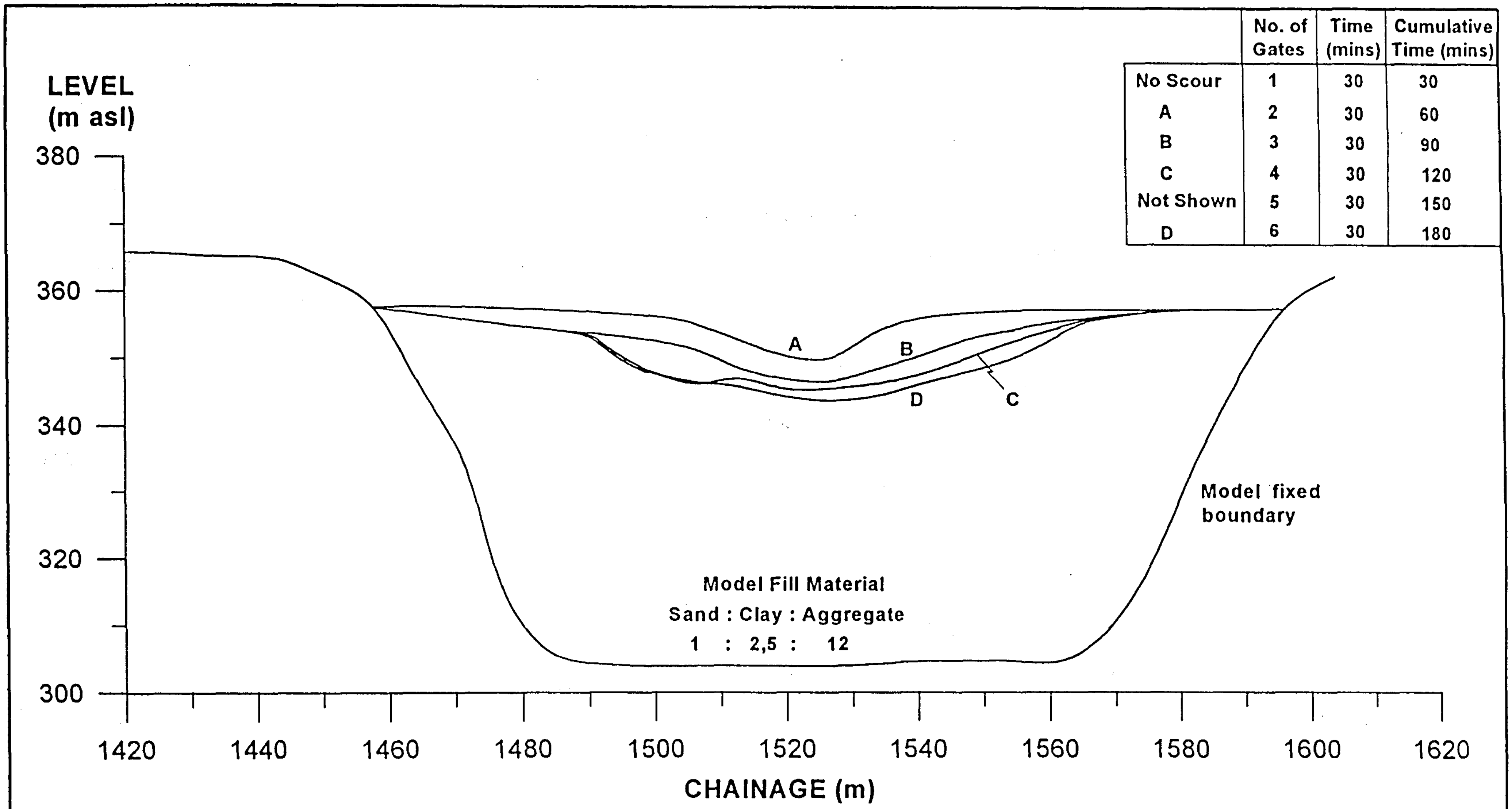


PLUNGE POOL SCOUR

SCOUR TESTS - LOOSE AGGREGATE IN ACTUAL SCOUR HOLE

FIGURE 14

CSIR - EMATEK



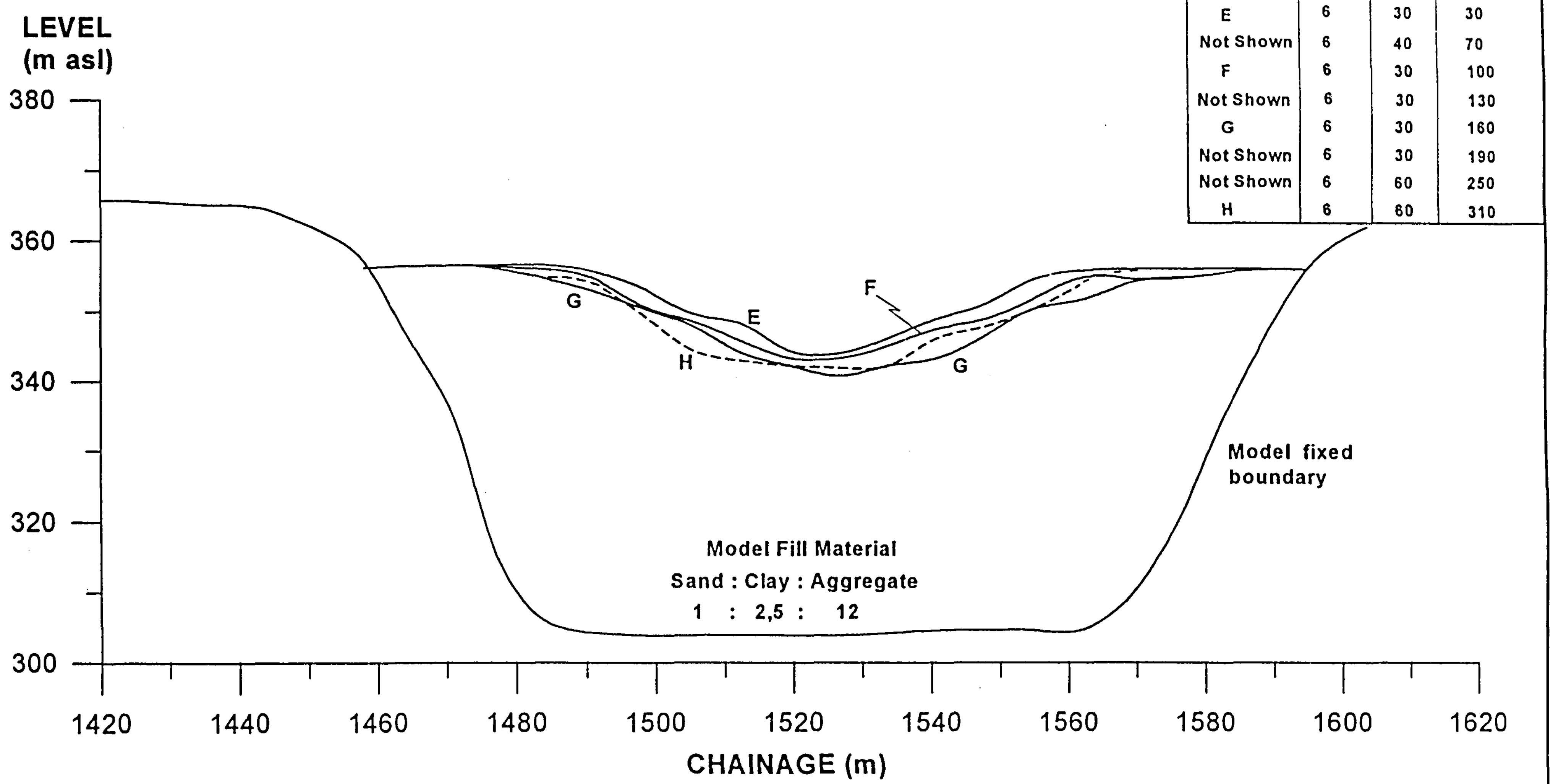
	No. of Gates	Time (mins)	Cumulative Time (mins)
No Scour	1	30	30
A	2	30	60
B	3	30	90
C	4	30	120
Not Shown	5	30	150
D	6	30	180

PLUNGE POOL SCOUR

SCOUR TESTS - SAND : CLAY : AGGREGATE 1 : 2,5 : 12 (Part I)

FIGURE 15

CSIR - EMATEK



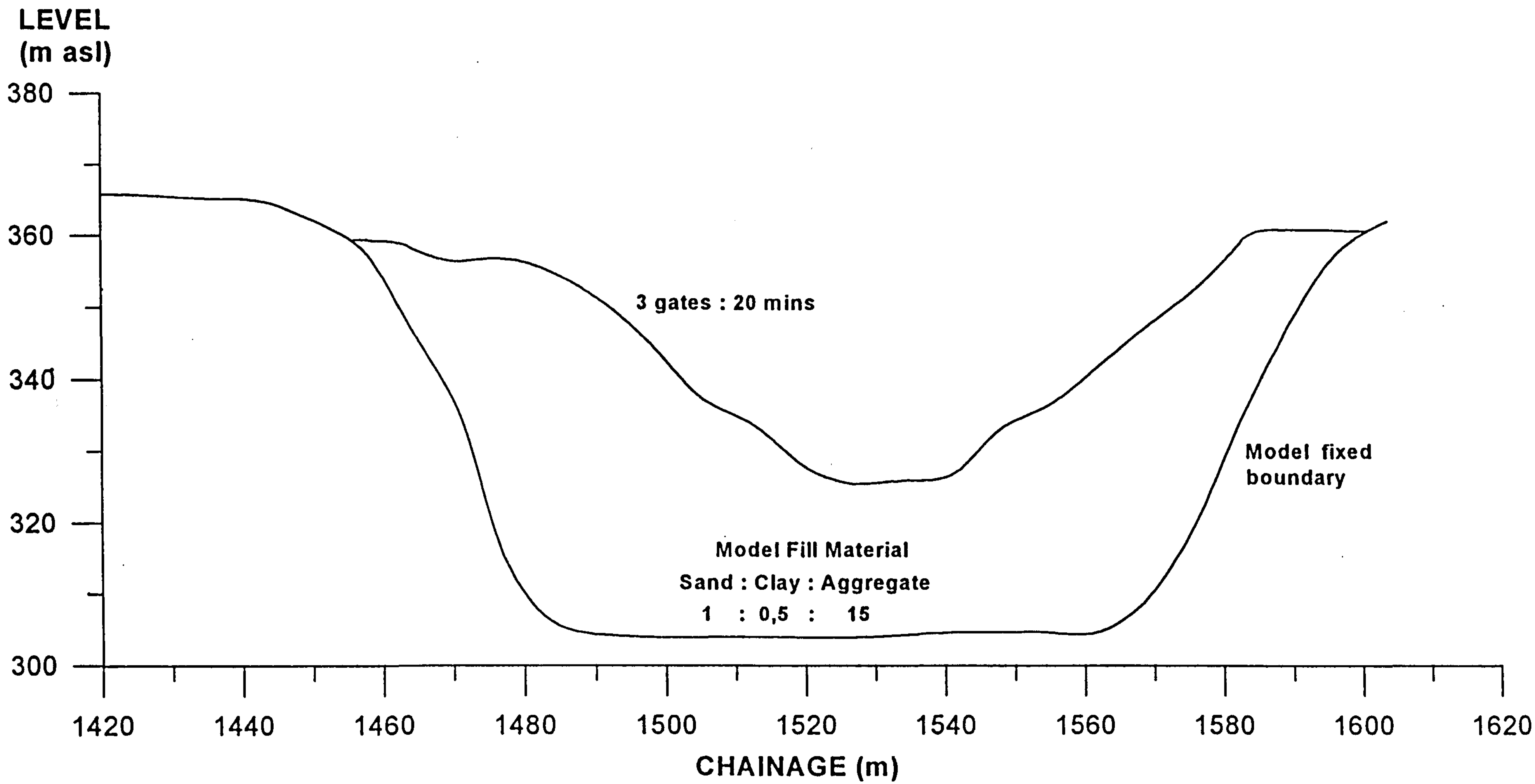
	No. of Gates	Time (mins)	Cumulative Time (mins)
E	6	30	30
Not Shown	6	40	70
F	6	30	100
Not Shown	6	30	130
G	6	30	160
Not Shown	6	30	190
Not Shown	6	60	250
H	6	60	310

PLUNGE POOL SCOUR

SCOUR TESTS - SAND : CLAY : AGGREGATE 1 : 2,5 : 12 (Part II)

FIGURE 16

CSIR - EMATEK



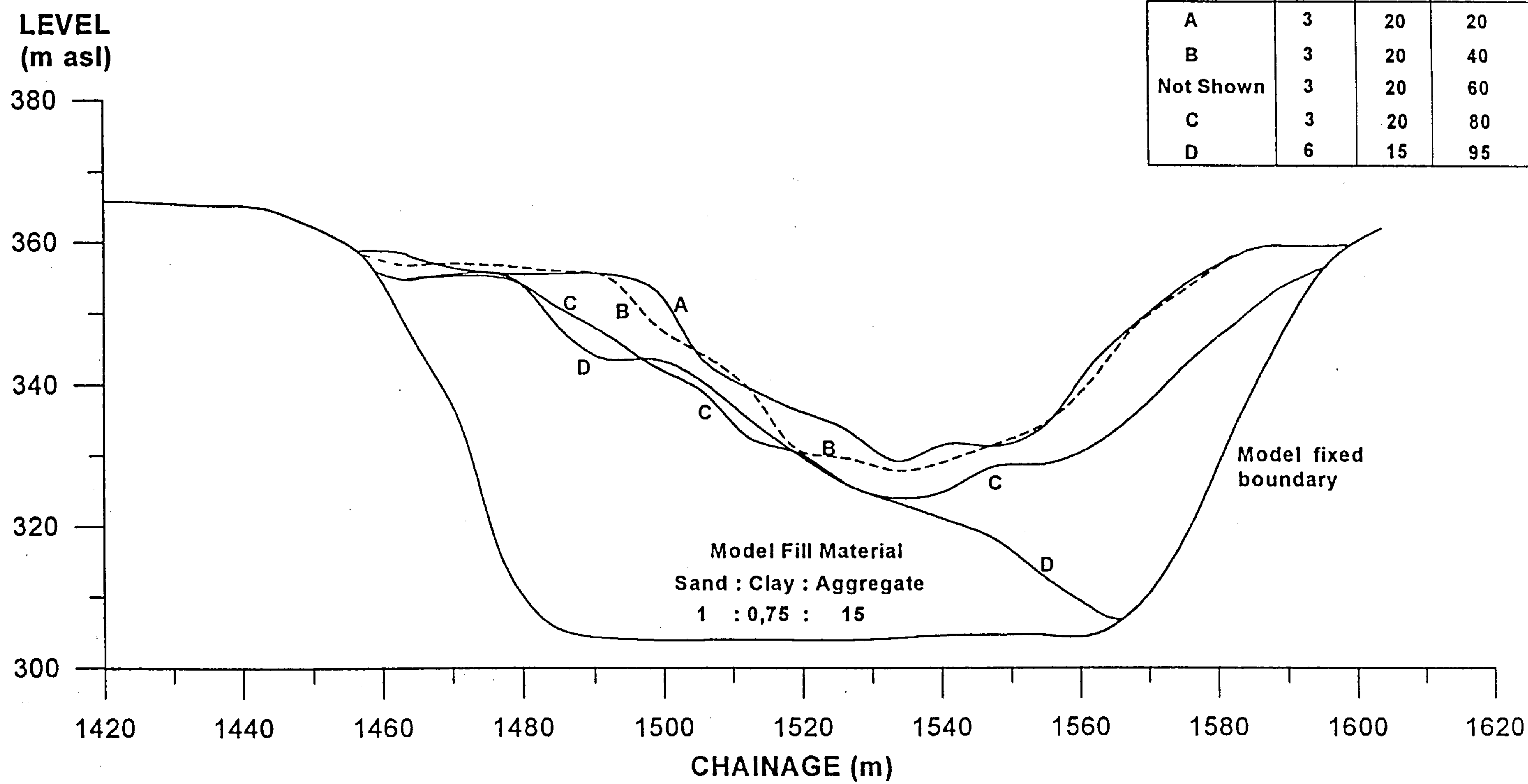
PLUNGE POOL SCOUR

SCOUR TESTS - SAND : CLAY : AGGREGATE 1 : 0,5 : 15

FIGURE 17

CSIR - EMATEK

	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	20	20
B	3	20	40
Not Shown	3	20	60
C	3	20	80
D	6	15	95

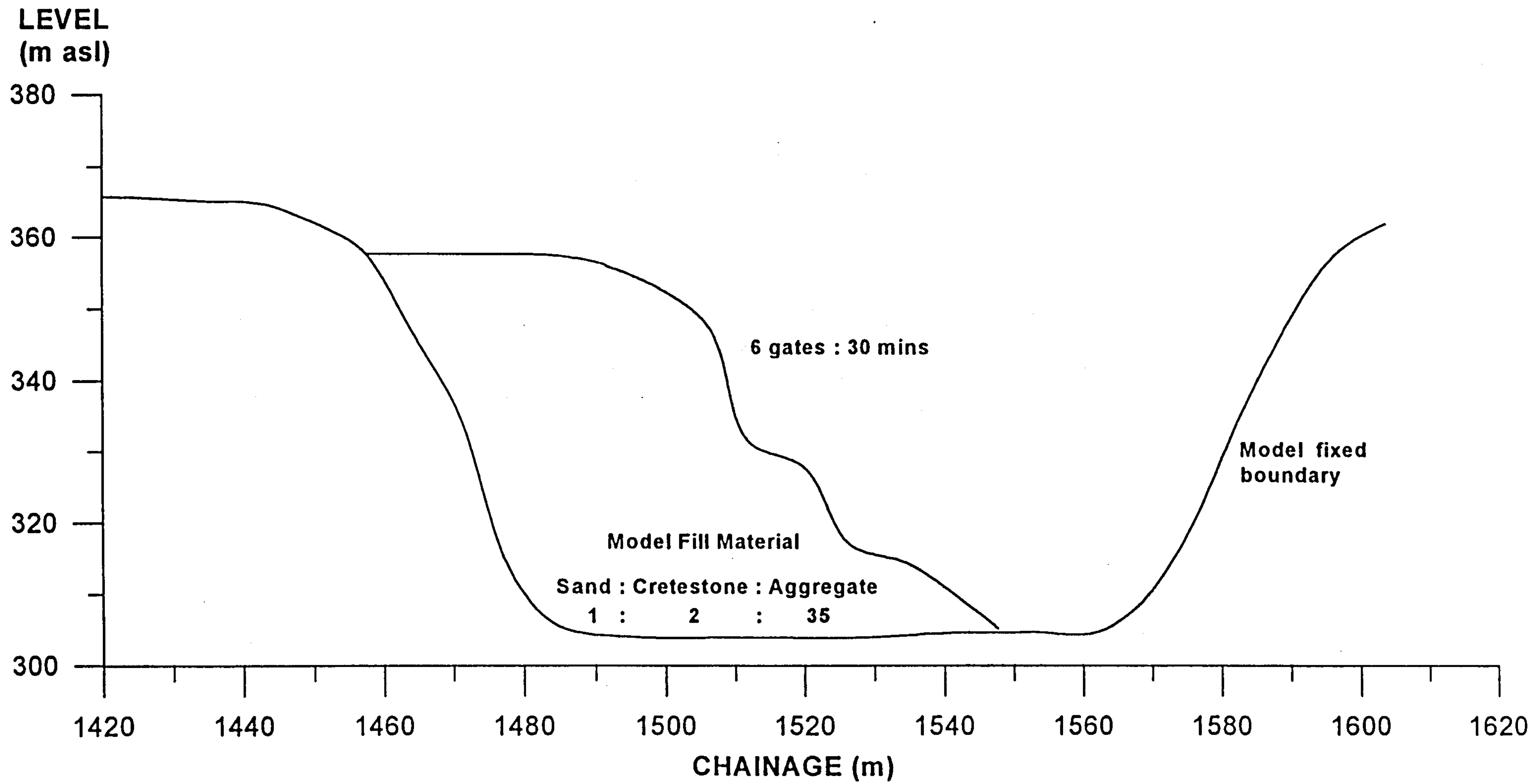


PLUNGE POOL SCOUR

SCOUR TESTS - SAND : CLAY : AGGREGATE 1 : 0,75 : 15

FIGURE 18

CSIR - EMATEK

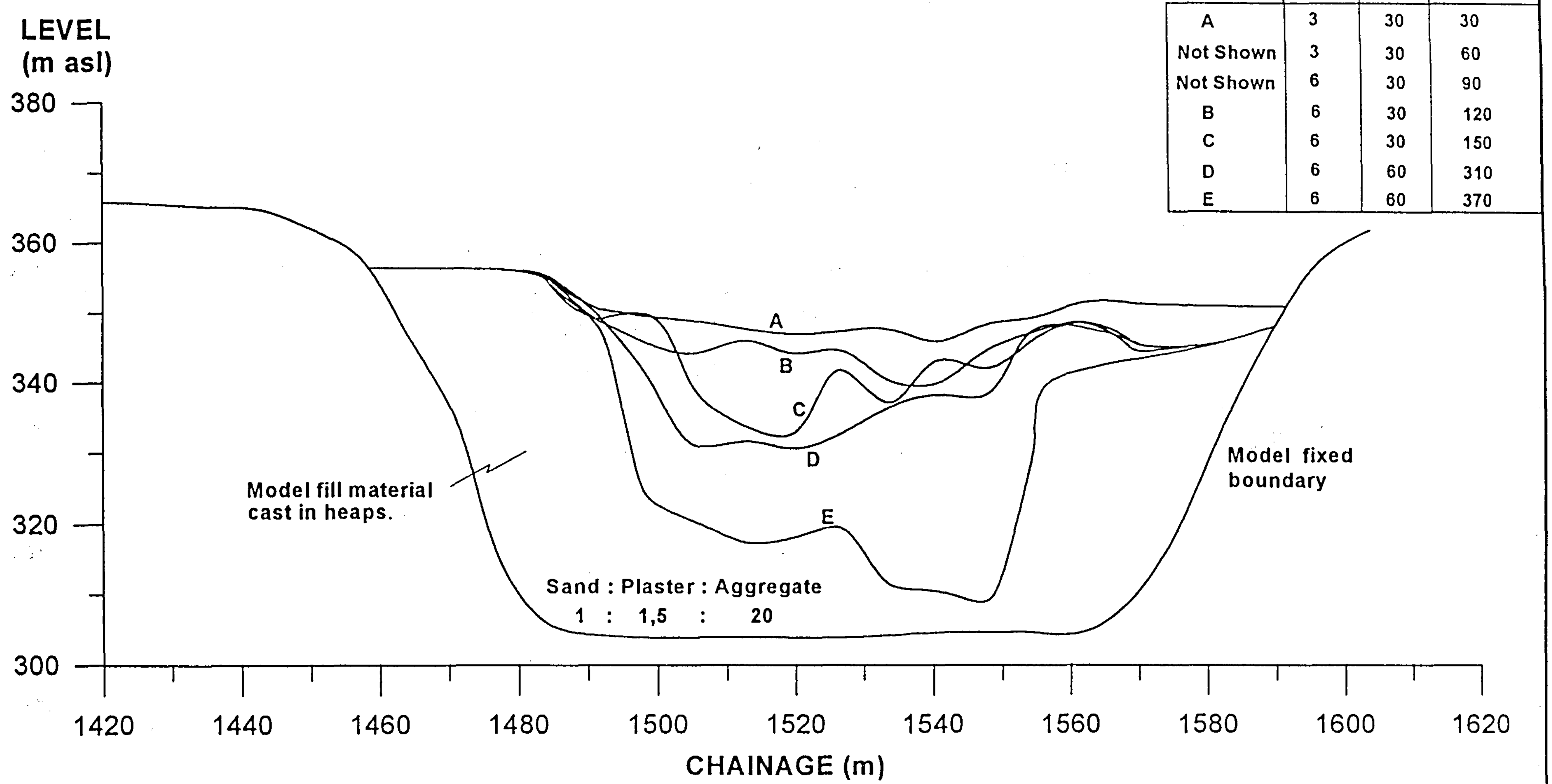


PLUNGE POOL SCOUR

SCOUR TESTS - SAND : CRETESTONE : AGGREGATE 1 : 2 : 35

FIGURE 19

CSIR - EMATEK



	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	30	30
Not Shown	3	30	60
Not Shown	6	30	90
B	6	30	120
C	6	30	150
D	6	60	310
E	6	60	370

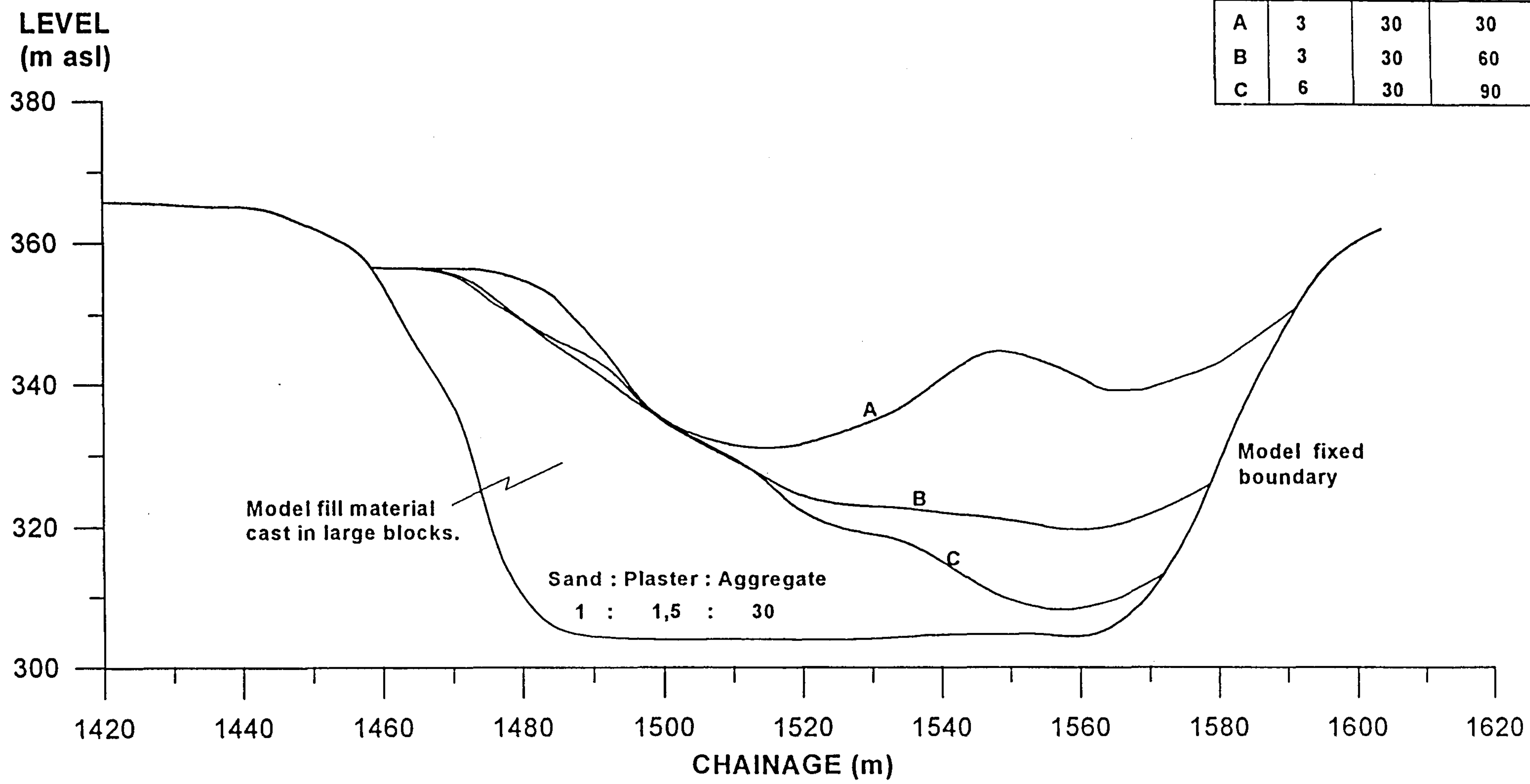
PLUNGE POOL SCOUR

SCOUR TESTS - SAND : PLASTER : AGGREGATE 1 : 1,5 : 20

FIGURE 20

CSIR - EMATEK

	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	30	30
B	3	30	60
C	6	30	90

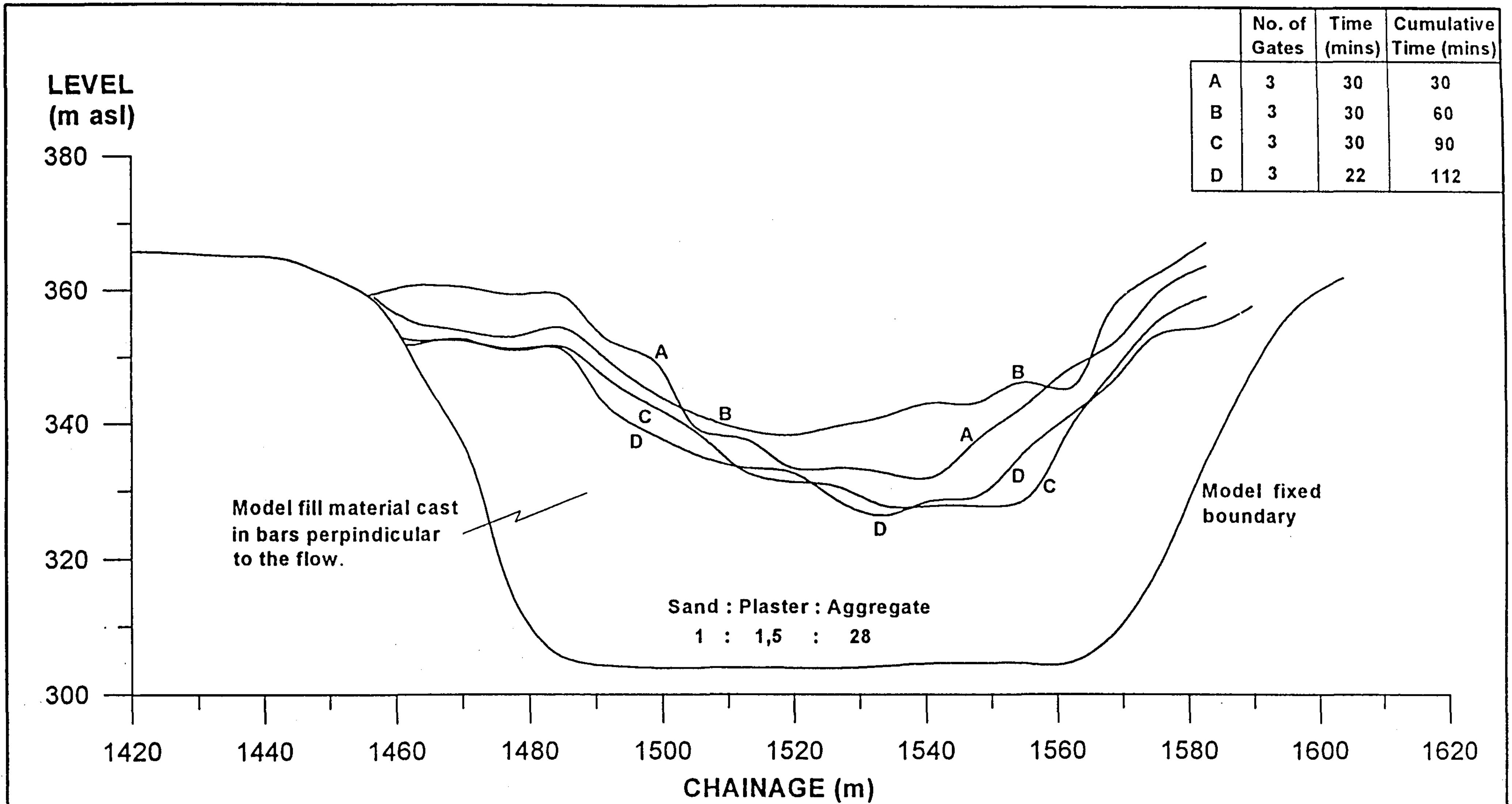


PLUNGE POOL SCOUR

SCOUR TESTS - SAND : PLASTER : AGGREGATE 1 : 1,5 : 30

FIGURE 21

CSIR - EMATEK



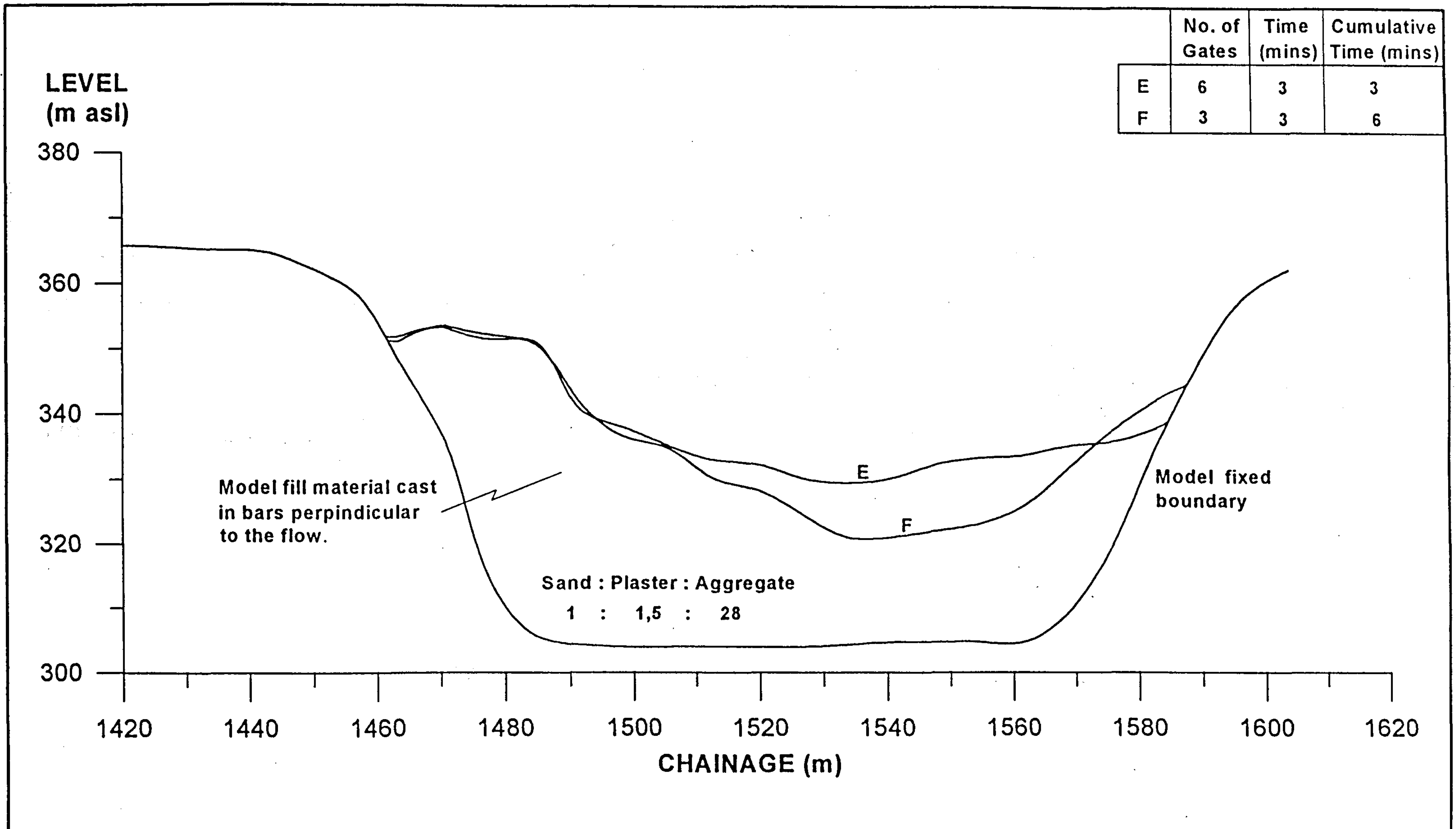
	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	30	30
B	3	30	60
C	3	30	90
D	3	22	112

PLUNGE POOL SCOUR

SCOUR TESTS - SAND : PLASTER : AGGREGATE 1 : 1,5 : 28 (Part I)

FIGURE 22

CSIR - EMATEK



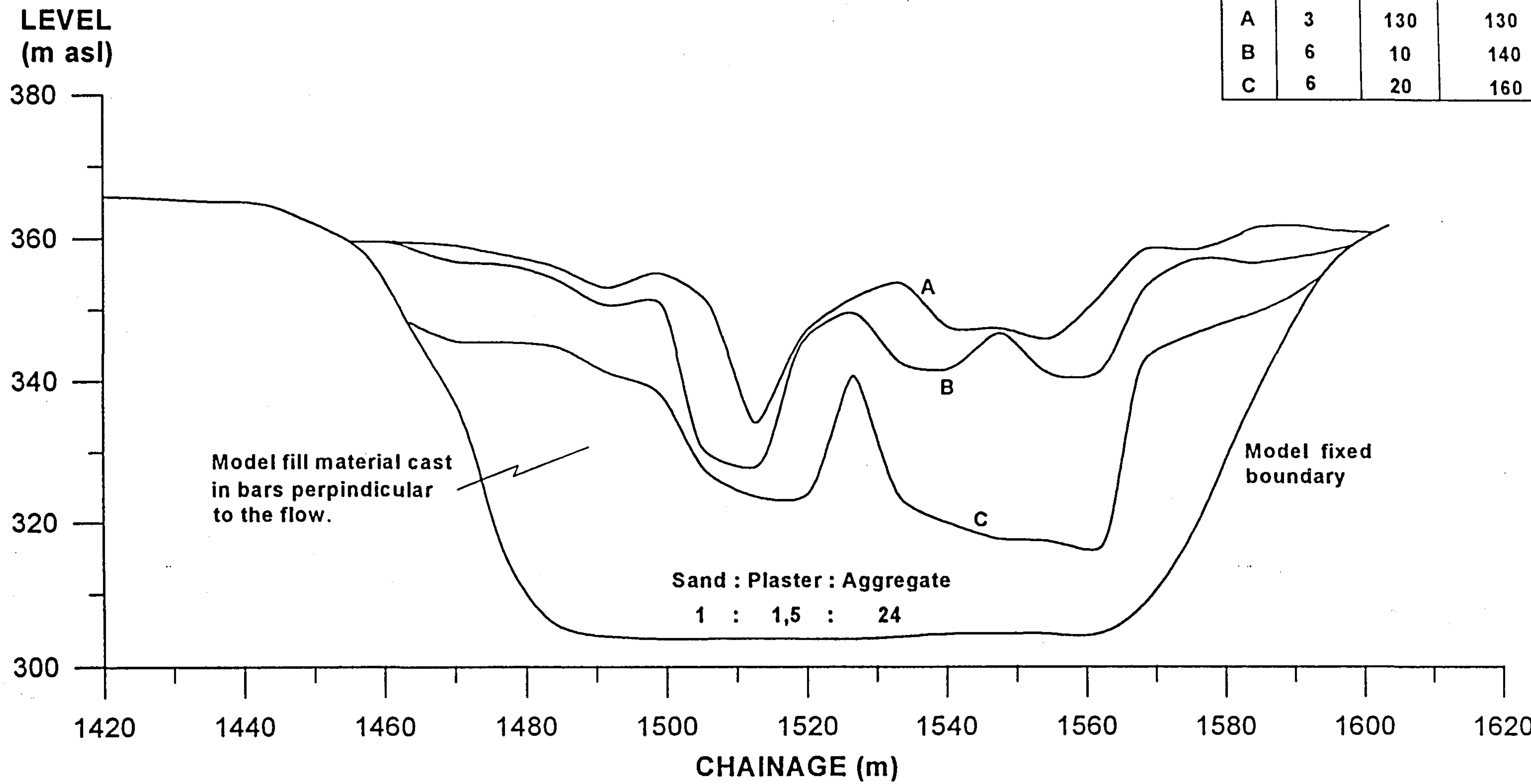
PLUNGE POOL SCOUR

SCOUR TESTS - SAND : PLASTER : AGGREGATE 1 : 1,5 : 28 (Part II)

FIGURE 23

CSIR - EMATEK

	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	130	130
B	6	10	140
C	6	20	160

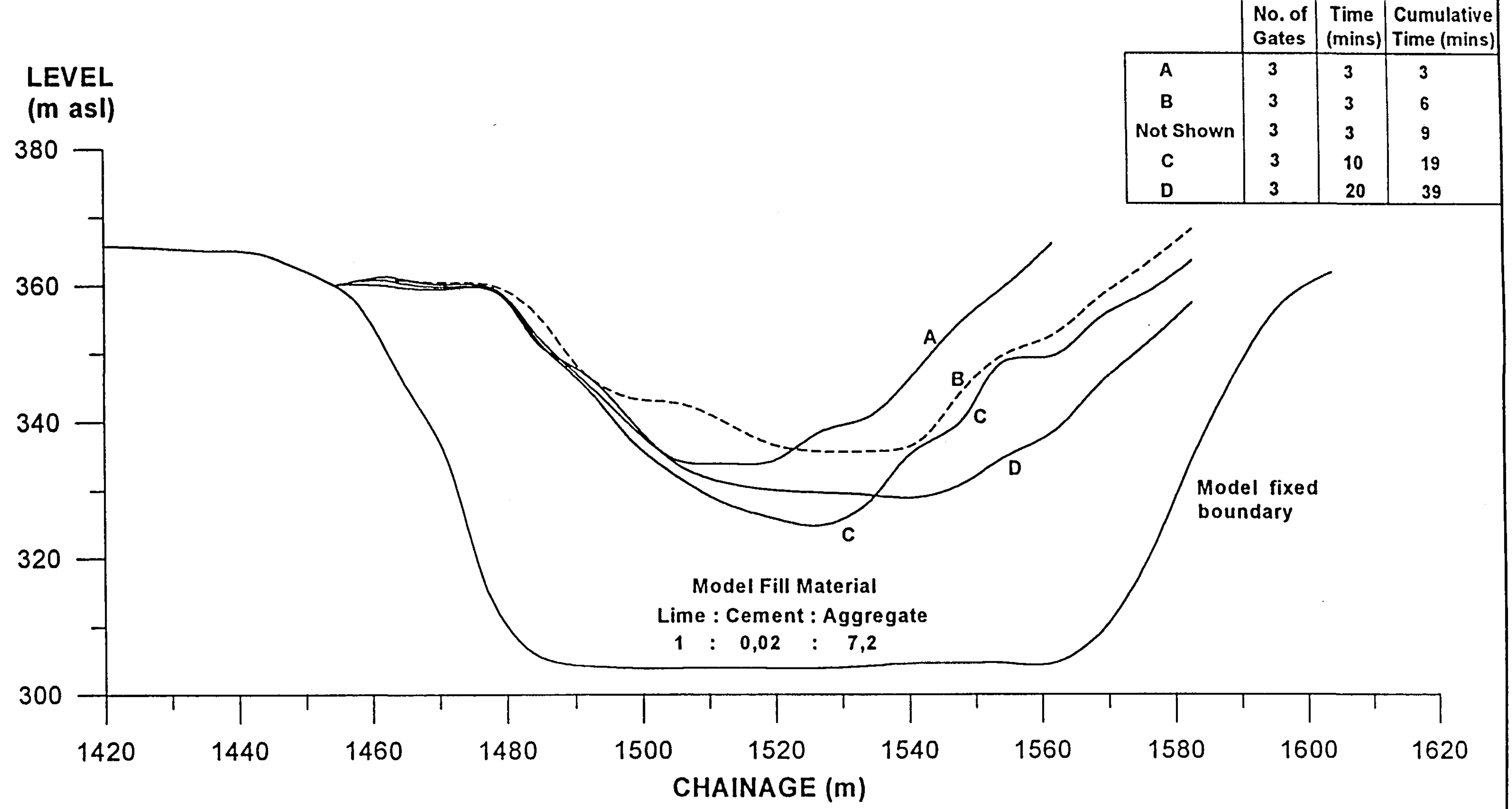


PLUNGE POOL SCOUR

SCOUR TESTS - SAND : PLASTER : AGGREGATE 1 : 1,5 : 24

FIGURE 24

CSIR - EMATEK



	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	3	3
B	3	3	6
Not Shown	3	3	9
C	3	10	19
D	3	20	39

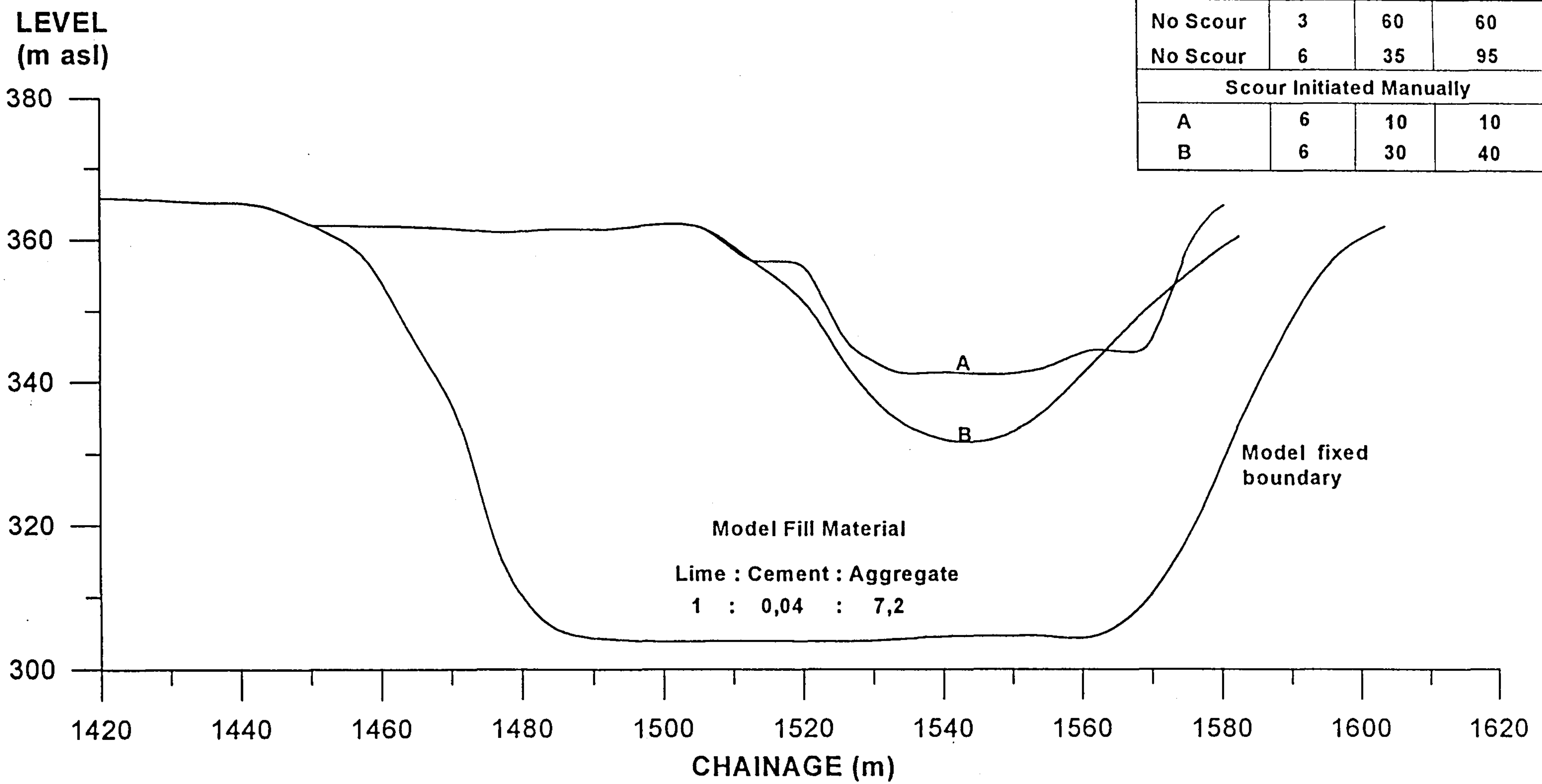
PLUNGE POOL SCOUR

SCOUR TESTS - LIME : CEMENT : AGGREGATE 1 : 0,02 : 7,2

FIGURE 25

CSIR - EMATEK

	No. of Gates	Time (mins)	Cumulative Time (mins)
No Scour	3	60	60
No Scour	6	35	95
Scour Initiated Manually			
A	6	10	10
B	6	30	40

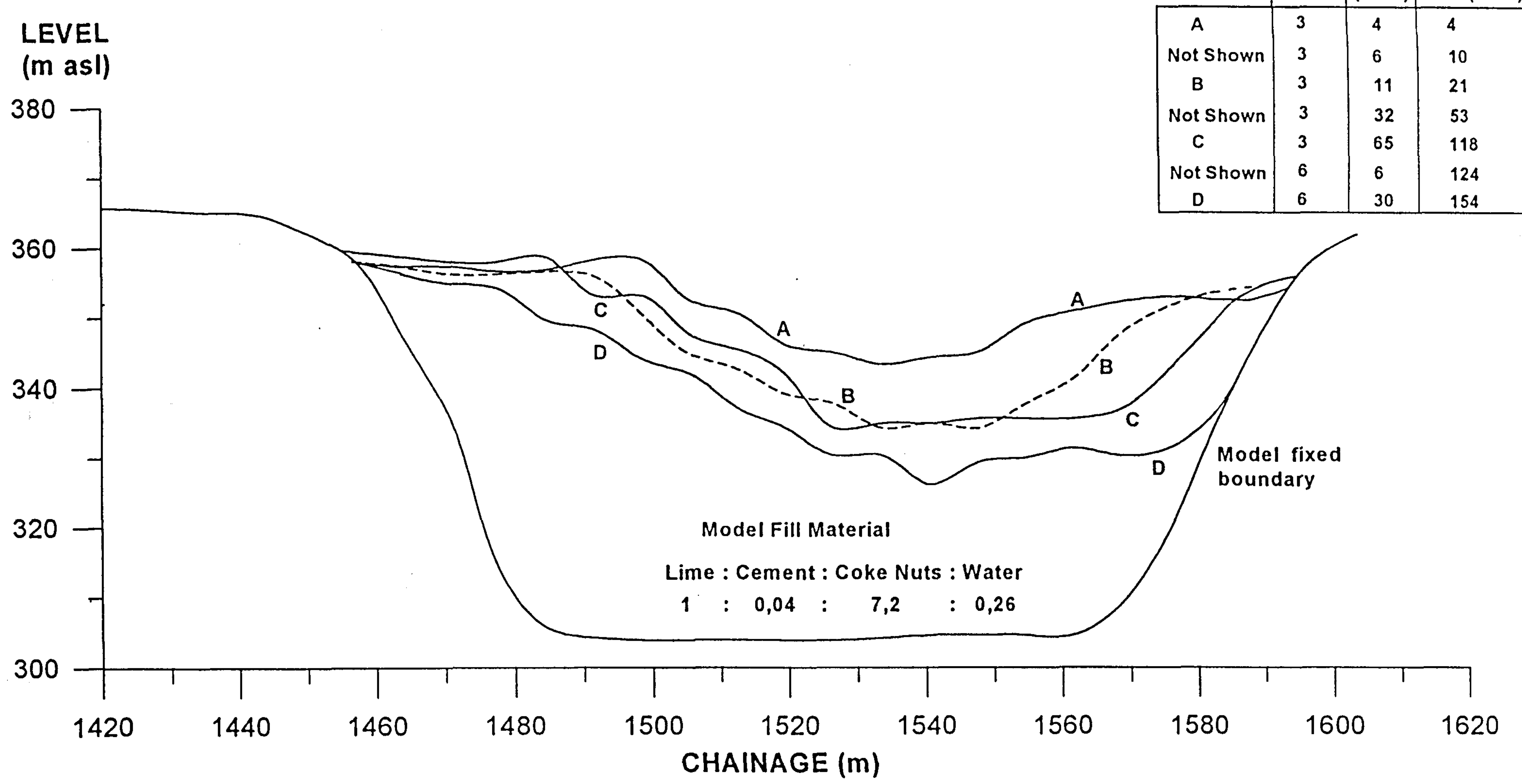


PLUNGE POOL SCOUR

SCOUR TESTS - LIME : CEMENT : AGGREGATE 1 : 0,04 : 7,2

FIGURE 26

CSIR - EMATEK



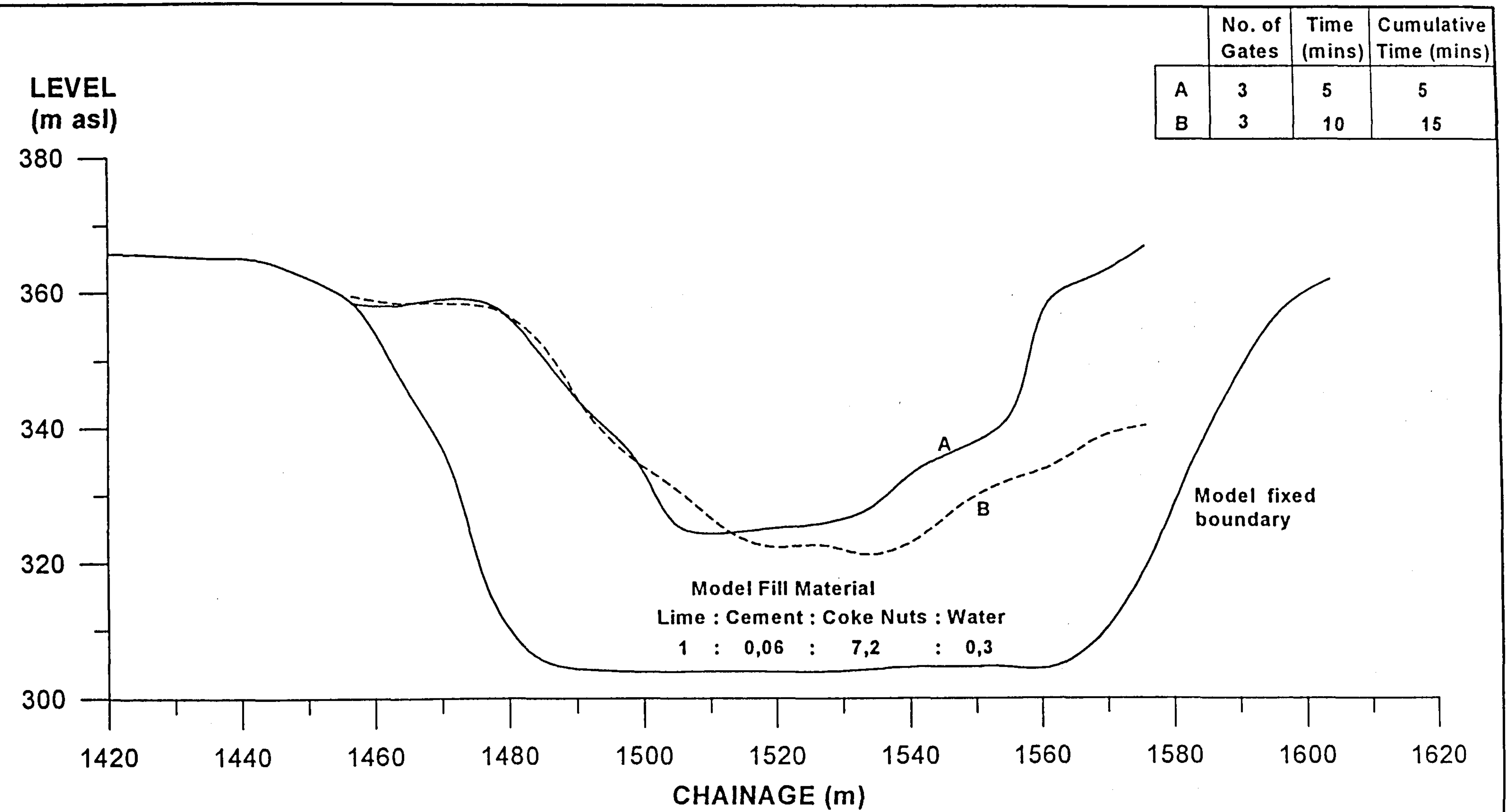
	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	4	4
Not Shown	3	6	10
B	3	11	21
Not Shown	3	32	53
C	3	65	118
Not Shown	6	6	124
D	6	30	154

PLUNGE POOL SCOUR

SCOUR TESTS - LIME : CEMENT : COKE NUTS 1 : 0,04 : 7,2

FIGURE 27

CSIR - EMATEK



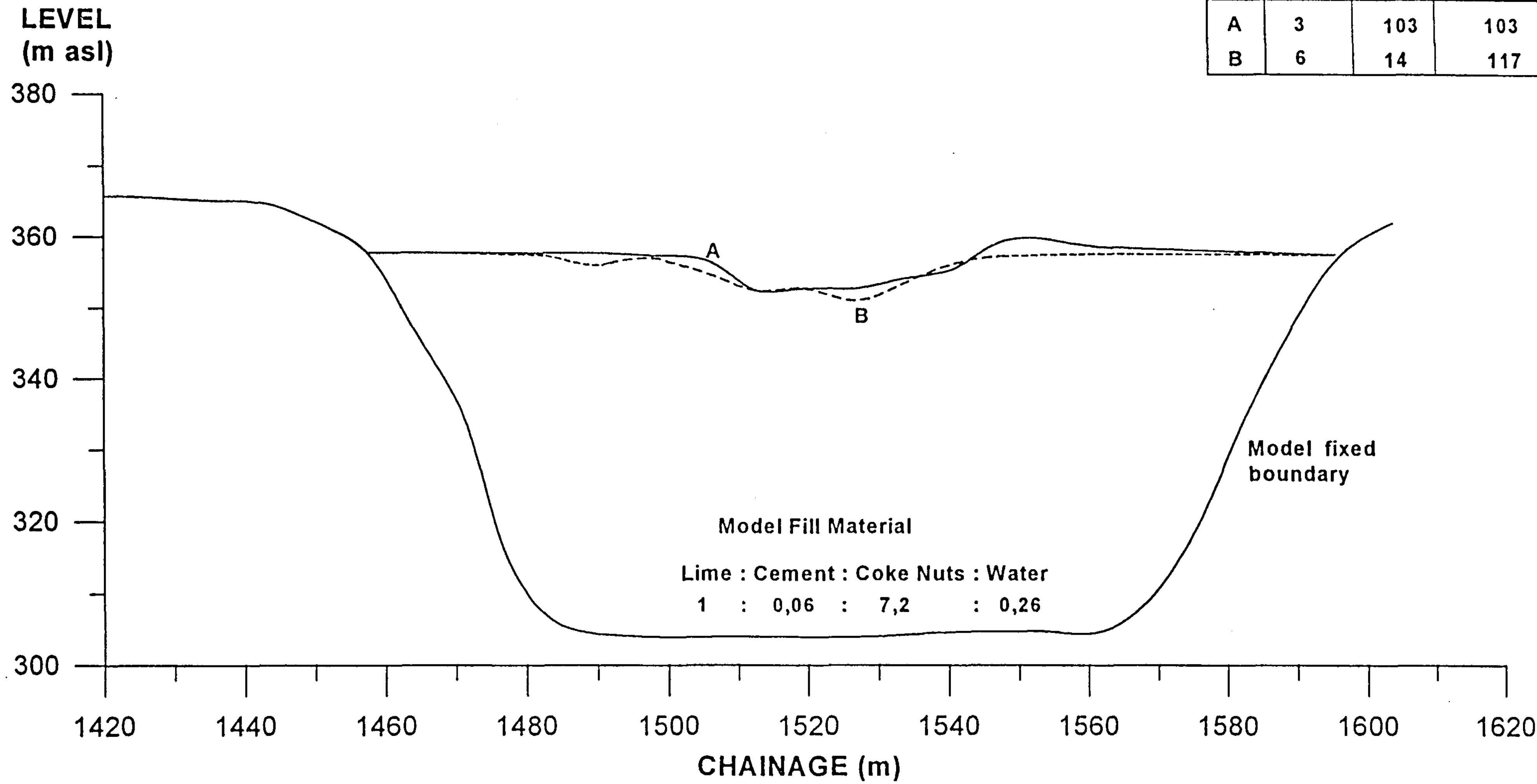
PLUNGE POOL SCOUR

SCOUR TESTS - LIME : CEMENT : COKE NUTS 1 : 0,06 : 7,2 (Test I)

FIGURE 28

CSIR - EMATEK

	No. of Gates	Time (mins)	Cumulative Time (mins)
A	3	103	103
B	6	14	117



PLUNGE POOL SCOUR

SCOUR TESTS - LIME : CEMENT : COKE NUTS 1 : 0,06 : 7,2 (Test II)

FIGURE 29

CSIR - EMATEK

APPENDIX A : TABLES FOR THE CALCULATION OF THE KIRSTEN NUMBER (after Kirsten 1982).

Kirsten Number $$K_n = M_s \cdot \frac{RQD}{Jn} \cdot Js \cdot \frac{Jr}{Ja}$$

TABLE A1 : DERIVATION OF MASS STRENGTH NUMBERS (M_s)

MATERIAL TYPE	CONSISTENCY		SPT BLOW COUNT (No.)	MASS STRENGTH No. M_s
Granular Soil	Very loose	Crumbles easily when scraped with pick	0-4	0,02
	Loose	Small resistance penetration, sharp end pick	4-10	0,04
	Medium dense	Considerable resistance to sharp end pick	10-30	0,09
	Dense	Very high resistance to sharp end pick	30-50	0,19
	Very dense	High resistance to repeated blows, sharp end	50-80	0,41

MATERIAL TYPE	CONSISTENCY		VANE SHEAR STRENGTH (kPa)	MASS STRENGTH No. M_s
Cohesive Soil	Very soft	Pick pushed in up to handle	0-80	0,02
	Soft	Sharp end pick pushed in 30-40 mm	80-140	0,04
	Firm	Sharp end pick pushed in 10 mm	140-210	0,09
	Stiff	Small indentation pushing sharp end pick	210-350	0,19
	Very stiff	Slight indentation from blow sharp end pick	350-750	0,41

MATERIAL TYPE	CONSISTENCY		UNIAXIAL STRENGTH (MPa)	MASS STRENGTH No. M_s
	Very soft	Crumbles firm blows sharp end pick, can be	- 1,7	0,9
		peeled off with knife	1,7 - 3,3	1,9
Rock	Soft	1-3 mm indentations firm blows sharp end pick,	3,3 - 6,6	4,0
		just scraped/peeled with knife	6,6 - 13,2	8,4
	Hard	Hand-held specimen broken with single firm	13,2 - 26,4	17,7
		blow hammer end pick		
	Very hard	Hand-help specimen breaks with several blows	26,4 - 53,0	35,0
		hammer end pick	53,0 - 106	70,0
	Extremely hard .	Specimen requires many blows hammer end	106 - 212	140,0
		pick to break intact material	212 -	280,0

continued /..

TABLE A1 : Continued

MATERIAL TYPE	CONSISTENCY		IN SITU DEFORMATION MODULUS (MPa)	MASS STRENGTH No. M_r
	Very loose	Matrix crumbles very easily when scrapped	0-4	0,02
Detritus	Loose	Small resistance penetration sharp end pick	4-10	0,05
	Medium dense	Considerable resistance penetration	10-30	0,10
	Dense	Requires many blows to dislodge particles	30-80	0,21
	Very dense	High resistance to repeated blows pick	80-200	0,44

TABLE A2 : ROCK QUALITY DESIGNATE (RQD)

JOINTS/m³	RQD
33	5
32	10
30	15
29	20
27	25
26	30
24	35

JOINTS/m³	RQD
23	40
21	45
20	50
18	55
17	60
15	65
14	70

JOINTS/m³	RQD
12	75
11	80
9	85
8	90
6	95
5	100

Based on : $RQD = 115 - 3{,}3\ (\text{Joints/m}^3)$ with limits $5 \leq RQD \leq 100$

TABLE A3 : JOINT SET NUMBER (Jn)

Number of Joint Sets	Joint Set Number Jn
Intact, no or few joints/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

TABLE A4 : RELATIVE GROUND STRUCTURE NUMBER (Js)

Dip direction of closer spaced joint set	Dip angle of closer spaced joint set	Js for Different Ratios of Joint Spacings			
		1:1	1:2	1:4	1:8
Vertical	90	1,00	1,00	1,00	1,00
Downstream	85	0,72	0,67	0,62	0,56
Downstream	80	0,63	0,57	0,50	0,45
Downstream	70	0,52	0,45	0,41	0,38
Downstream	60	0,49	0,44	0,41	0,37
Downstream	50	0,49	0,46	0,43	0,40
Downstream	40	0,53	0,49	0,46	0,44
Downstream	30	0,63	0,59	0,55	0,53
Downstream	20	0,84	0,77	0,71	0,68
Downstream	10	1,22	1,10	0,99	0,93
Downstream	5	1,33	1,20	1,09	1,03
Vertical	0	1,00	1,00	1,00	1,00
Upstream	5	0,72	0,81	0,86	0,90
Upstream	10	0,63	0,70	0,76	0,81
Upstream	20	0,52	0,57	0,63	0,67
Upstream	30	0,49	0,53	0,57	0,59
Upstream	40	0,49	0,52	0,54	0,56
Upstream	50	0,53	0,56	0,58	0,60
Upstream	60	0,63	0,67	0,71	0,73
Upstream	70	0,84	0,91	0,97	1,01
Upstream	80	1,22	1,32	1,40	1,46
Upstream	85	1,33	1,39	1,45	1,50
Vertical	90	1,00	1,00	1,00	1,00

NOTES : For ratios of joint spacings smaller than 1:8, use the values given for 1:8.
For intact material take Js = 1,0

TABLE A5 : JOINT ROUGHNESS NUMBER (Jr)

Joint Seperation	Condition of Joint	Jr
Joints/fissures tight or closing on movement	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 remaining open on movement	Containing joint fill, sufficient to prevent joint wall contact.	1,0
	Shattered or micro-shattered clays	1,0

TABLE A6 : JOINT ALTERATION NUMBER (Ja)

A4

Description of joint fill material	Ja for joint seperation (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*	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 filling, with or without crushed rock.	5,0*	10,0**	18,0

Notes:
1. Joint walls effectively in contact.
2. Joint walls come into contact after approximately 100 mm shear.
3. Joint walls do not come into contact at all upon shear.
4. * Values added to Barton *et al*'s data.
5. ** Also applies when crushed rock occurs in clay fill material without rock wall contact.

TABLE A7 : SOME TYPICAL KIRSTEN NUMBER VALUES

Description	Ms	RQD	Jn	Js	Jr	Ja	N
Dry,soft rock, laminated (25 mm apart, smooth, planar 1-5mm wide filled with a clayey sand), residual quartzitic sandstone.	8,39	5,0	1,5	1,0	1,0	6,0	4,7
Dry, very soft rock, tightly jointed (spacing 125 mm in all directions), residual shale.	1,88	35	2,7	1,0	1,0	3,0	8,0
Dry, hard rock, tightly laminated, residual quartzitic sandstone, joints 30 mm apart (rough planar filled with a clayey sand and dip 30° downstream).	17,7	5	1,5	0,6	1,5	3,0	16,8
Dry, very stiff, intact, clay-sand with abundant iron concretions.	0,41	100	1,0	1,0	4,0	4,0	41,0
Dry, soft rock, laminated, residual quartzitic sandstone, joints spaced 40 mm, 2 mm wide, planar and filled with clayey sand.	8,39	30	1,5	1,0	1,5	6,0	42,0
Dry soft rock, laminated residual quartzitic sandstone, joints spaced 55 mm, 5 mm wide, rough undulating and filled with clayey sand.	8,39	55	1,5	1,0	3,0	10,0	92
Dry, soft rock, intact, well cemented ferricrete.	3,95	100	1,0	1,0	4,0	4,0	395
Dry, hard rock, laminated, quartzitic sandstone, laminations spaced 83 mm, 1 mm wide, rough undulating and filled with clayey sand.	17,7	72	1,0	1,0	3,0	3,0	850
Dry, hard rock, jointed quartzitic sandstone, two orthogonal joint sets, (primary set spaced 143 mm, tight, rough and undualting, secondary is random, spaced 340 mm, 2 mm wide, irregular undulating filled with clayey sand.	17,7	77	1,5	1,0	3,0/3,0	1,0/6,0	2044
Dry, extremely hard rock, fresh jointed quartzite, three joint sets, all tight with a thin mylonite filling.	280	65	2,7	1,0	1,5	2,0	5000
Dry, very hard rock, intact, ferrugenized detritus, containing boulders of chert and quartzite up to 500 mm, densely packed in well cemented silty sand matrix.	35,0	100	1,0	1,0	4,0	1,0	14000

PHASE 2

EXTENDED TESTS USING WEAK CEMENT / SAND MIX, PLUS A REPEAT OF COKE NUT TESTS

by

E Dunkley

PLUNGE POOL SCOUR REPRODUCTION
IN PHYSICAL HYDRAULIC MODELS :
EXTENDED TESTS

SCOPE OF THIS REPORT

The information gathered in this report is a continuation of the work carried out by Mr K R K Blake (CSIR, Environmentek) concerning physical modelling aspects of plunge pool scour reproduction.

The research project directed by Mr K R K Blake set out to investigate the most appropriate materials to be used to model the bed rock below a dam wall, for use in scale physical models. The study investigated a wide range of possible materials to represent the bed material in scale models. To test the proposed model bed materials, an existing dam had to be modelled in the laboratory: the Kariba Dam on the Zambezi river was chosen. The reasons for this choice are explained in the report written by Mr K R K Blake. The criterion against which the materials were tested was their ability to reproduce the scour hole formed at the Kariba Dam. As explained by Mr K R K Blake, none of the materials tested gave ideal results. However one of the materials tested, consisting of a cement based binder with coke nuts (degassed anthracite) as the aggregate, gave much better results than the others.

The aim was therefore to pursue in the direction set out by Mr K R K Blake and carry out further investigations into this material by performing additional tests. However, the coke nuts became temporarily unavailable. It was thus decided to test another mix consisting of a cement based binder with sand and water. Tests with the coke nuts were also performed once the material became available.

This work was carried out by Miss E C Dunkley with the guidance of Mr K R K Blake. The model tests were performed by Mr A Gregorio , Mr F Jack and Miss J Erasmus, under the supervision of Miss E C Dunkley. This report was compiled by Miss E C Dunkley.

L BARWELL
PROGRAMME MANAGER

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1. INTRODUCTION

The aim of the research project carried out by Mr Blake was to develop and test physical model materials or techniques that would more accurately reproduce scour in plunge pools. One material tested, consisting of a cement based binder with coke nuts as the aggregate, proved to reproduce the scour remarkably better than all the other materials tested. With Mr Blake leaving South Africa, the study needed to be completed. It was therefore decided to pursue in the direction set out by Mr Blake and carry out further investigations into that particular material by performing additional tests.

However, the coke nuts became locally unavailable at the time. The various firms approached that dealt in coke nuts would have had to import the material from Zimbabwe and would also not deliver in small quantities. It therefore appeared that the coke nuts could be obtained, however the costs associated would have been very expensive, and the certainty of obtaining more coke nuts in the future could not be established. The coke nuts thus appeared to become a " non readily available" material.

Therefore, it was decided to continue the study using another material which hadn't been tested by Mr Blake. The mix used consisted of a cement based binder with sand and water. Various mixes with different cement strengths were tested. At that time coke nuts became available, therefore two additional tests were performed. The aim was to reproduce the results obtained by Mr Blake.

This report gives a description of the tests conducted and the results obtained.

2. MODEL TESTS

2.1 General

Two different types of mixes were tested upon. The first one consisted of a cement-sand-water mix: five mixes consisting of ordinary portland cement, sand (Malmesbury quarry) and water (ordinary tap water) were tested. Each time, the sand to cement ratio was changed. The mixes all appeared to be fairly dry. The proportions for all mixes are indicated in Section 2.2.

The second one consisted of a cement-lime-coke nuts-water mix, using the proportions indicated by Mr Blake in his report.

The tests were undertaken in the Kariba Dam model. The description of the Kariba Dam model and flow rates are given in the report written by Mr Blake. No alteration was made to the existing model and the same flow rates were used.

Six tests were carried out in the Kariba Dam model. A description of these tests and the results obtained for each of them is given hereafter.

2.2 Description of Tests

2.2.1 Cement-sand-water mixes

Four tests were performed in the Kariba Dam model. Each mix, except for the first one, once put in the hole below the dam wall, was allowed to rest there for three days before running the test. For the first test, the mix rested for a period of four days. For each test carried out, the cement strength, the temperature in the laboratory, the sand water contents and grain size were carefully measured.

For each mix, five or six moulds (dimensions: 150x150x150 mm) were filled and after four days for the first test and three days for the second, third and fourth tests, the moulds were taken to the University of Stellenbosch so that the cubes could be crushed as the test began on the mix in the laboratory. The tests on the cubes were done according to SABS method 0863:1994. The results are given for each mix hereafter.

The temperature was monitored from the start of the mixing until the test was performed with the use of a Mike Cotton Data Logger MCS-120 with a MCS-151 temperature sensor; the accuracy is ± 0,2°C. The results for each test are given in terms of an average, minimum and maximum temperature recorded during the four or three day period.

The water contents of the sand were determined by a gravimetric moisture determination technique.

The sand used was the same for all the mixes. A sample was analysed in the CSIR settling tube. The median grain size (D_{50}) was found to be approximately 440 μm.

Test 1 (19.02.98-23.02.98)

For the first test, the hole below the dam wall was filled with the following mix ratio:

SAND	CEMENT	WATER
1	0,06	0,14

The hole was filled by tipping buckets of the mix into it. Each layer, approximately 2 cm thick, was then tamped with a wooden stick 10 to 15 times. Once the last layer was tipped into the hole it was carefully levelled out and smoothed out. Six moulds were also filled with the same mix and the same method of compaction was applied as the one in the hole.

The results of the testing of the concrete cubes, after a period of 4 days, are given in the following table.

TESTING OF CONCRETE CUBES: COMPRESSION STRENGTH				
	Mass (kg)	**Dimensions (mmxmmxmm)**	**Force (kN)**	**Strength (MPa)**
Cube 1	5.22	149x151x150	6.4	0.284
Cube 2	5.78	150x151x150	11.29	0.498
Cube 3	5.36	150x150x150	8.35	0.371
Cube 4	5.44	149x150x150	5.98	0.268
Cube 5	5.9	150x151x150	9.61	0.424
Cube 6	5.2	148x150x150	6.79	0.306

The average strength was therefore: 0.358 MPa.

During the four day period, the following temperatures were recorded: the average temperature was 24,8°C, the minimum temperature was 20,4°C and the maximum temperature was 28,5°C. The sand moisture for this first mix was not determined. As shown in section 2.3, it appeared that this mix was too strong, as hardly any scouring occurred. It also appeared that filling the entire hole was very time consuming and waiting for four days for the mix to set only permitted one test per week.

It was therefore decided to let the mix set for three days instead. In order to test more mixes, it was also decided to test two mixes at the same time as explained hereafter.

Test 2 (06.03.98-09.03.98)

In order to get an idea of the appropriate mix to use, two different mixes were used to fill the hole. However, as filling the entire hole with the mixes was too time consuming, it was decided to fill two thirds of the hole with sand, cover it up with a cement floor and fill the remaining third with the mixes. A thin zinc plate was used as a separator between the two different mixes; the zinc plate was placed vertically along the longest axis of the hole, thus bisecting the hole. If one of the mixes gave a significant scouring depth (i.e. the scouring reached the cement floor), that mix would then be fully tested upon by filling the entire hole with it.

According to the results of Test 1, two weaker mixes were tested. One half was filled with the following mix ratio (where the cement content of the mix was halved):

MIX 1		
SAND	**CEMENT**	**WATER**
1	0,03	0,127

The other half was filled with the mix ratio:

MIX 2		
SAND	**CEMENT**	**WATER**
1	0,04	0,099

A different method of compaction was used than the one for Test 1. The same method was applied for both halves of the hole as well as for the moulds. The method consisted of sprinkling the mix over the hole from a height of 1,3 m. Once the last layer was sprinkled over the hole, it was gently levelled out with a flat piece of wood. Three moulds were filled for the first mix (Mix 1) and three were filled for the second mix (Mix 2). The results of the testing of the concrete cubes, after a period of 3 days, are given in the following table.

TESTING OF CONCRETE CUBES: COMPRESSION STRENGTH					
		Mass (kg)	**Dimensions (mm^3)**	**Force (kN)**	**Strength (MPa)**
MIX 1	**Cube 1**	5.70	150x150x150	2.74	0.122
	Cube 2	6.06	150x150x150	1.94	0.086
	Cube 3	6.10	150x150x150	3.89	0.173
MIX 2	**Cube 1**	6.22	150x150x150	11.52	0.512
	Cube 2	5.86	150x150x150	7.66	0.340
	Cube 3	6.06	150x150x150	9.76	0.434

The average strength for Mix 1 was therefore: 0,127 MPa. The average strength for Mix 2 was: 0,428 MPa.

During the three day period, the following temperatures were recorded: the average temperature was 20,9°C, the minimum temperature was 20,5°C and the maximum temperature was 21,4°C. The sand moisture analysis revealed that the sand water contents were of 1,6%.

The mix ratios were therefore:

MIX 1

SAND	CEMENT	WATER
1	0,03	0,144

MIX 2

SAND	CEMENT	WATER
1	0,04	0,115

As explained in section 2.3, the scouring was not significant enough to test one of those mixes fully. Therefore, it was decided to test two more weaker mixes.

Test 3 (13.03.98-16.03.98)

The same filling and separation technique as in Test 2 was used for Test 3. One half was filled with the following mix ratio:

MIX 1

SAND	CEMENT	WATER
1	0,024	0,113

The other half was filled with the mix ratio:

MIX 2

SAND	CEMENT	WATER
1	0,021	0,109

The same method of compaction was applied as the one used in Test 2. Figure 1 shows photographs of the hole filled with both mixes. Three moulds were filled for the first mix (Mix 1) and three were filled for the second mix (Mix 2). The results of the testing of the concrete cubes, after a period of 3 days, are given in the following table:

TESTING OF CONCRETE CUBES: COMPRESSION STRENGTH					
		Mass (kg)	**Dimensions (mm^3)**	**Force (kN)**	**Strength (MPa)**
MIX 1	**Cube 1**	6.22	150x150x150	5.95	0.264
	Cube 2	6.00	150x150x150	3.47	0.154
	Cube 3	5.74	150x150x150	2.82	0.125
MIX 2	**Cube 1**	5.88	150x150x150	1.67	0.074
	Cube 2	5.52	150x150x150	4.00	0.177
	Cube 3	5.62	150x150x150	1.25	0.056

The average strength for Mix 1 was therefore: 0,181 MPa. The average strength for Mix 2 was: 0,102 MPa. During the three day period, the following temperatures were recorded: the average temperature was 21,6°C, the minimum temperature was 20,9°C and the maximum temperature was 22,2°C.

The sand moisture analysis revealed that the sand water contents were of 1,83%. The mix ratios were therefore:

MIX 1

SAND	**CEMENT**	**WATER**
1	0,024	0,134

MIX 2

SAND	**CEMENT**	**WATER**
1	0,021	0,13

Both mixes gave a full scour (down to the cement floor). However, as explained in section 2.3, it appeared that the scouring was most probably being influenced by the zinc plate. To verify this, it was decided to remove the zinc plate from the hole and test on one mix only, the weaker one of the above, i.e. Mix 2.

Test 4 (17.03.98-20.03.98)

Two thirds of the hole were still filled with sand covered by a cement floor. The remaining third of the hole was then filled with Mix 2:

MIX 2		
SAND	**CEMENT**	**WATER**
1	0,021	0,109

The same method of compaction was applied as the one used in Test 2. Figure 2 shows photographs of the way the mix was sprinkled over the hole and of the hole filled with the mix. Five moulds were filled. However, the results of the testing of the concrete cubes, after a period of 3 days, revealed that the mix was too weak to even crush the cubes. The cubes were apparently not able to resist any pressure.

During the three day period, the following temperatures were recorded: the average temperature was 20,6°C, the minimum temperature was 20,1°C and the maximum temperature was 21,2°C. The sand moisture analysis revealed that the sand water contents were of 1,35%. The mix ratios were therefore:

MIX 2		
SAND	**CEMENT**	**WATER**
1	0,021	0,124

As expected, this mix did not give a full scour (down to the cement floor).

2.2.2 Cement-lime-coke nuts-water mixes

The initial scope of this study, as explained previously, was to further the investigations carried out by Mr Blake in trying to accurately predict the depth of scour holes below dams. In his conclusions, Mr Blake recommended that a particular material, consisting of a cement based binder with coke nuts (degassed anthracite) as the aggregate be investigated further since it gave remarkably better results than any other material that had been tested. The proportions for this material are (by volume):

MIX			
LIME	**CEMENT**	**COKE NUTS**	**WATER**
1	0,04	7,2	0,26

Once the coke nuts became available, additional tests could be performed. The first objective was to reproduce the results obtained by Mr Blake. The aim was then to investigate how the mix could be altered to give better results. The first step (reproduction of results) was not very successful. The last step was therefore not achieved.

The coke nuts were supplied by the local company KAZ Engineering, Elsies River. These were then sieved to keep only the pieces within the range 20 mm to 25 mm for the tests (as specified by Mr Blake). This in itself proved to be a very long process because of limited resources. Ordinary portland cement mixed with hydrated lime was used as the binder. Two tests were performed in the Kariba Dam model: Test 5 and Test 6.

Test 5 (04.06.98-08.06.98)

For this test, it was also decided to use a third of the hole, to ensure the test was performing as described by Mr Blake's experiments. The mix ratio of lime, cement and coke nuts was : 1:0,04:7,2 with a water content of 0,26 units. This was placed in the model and allowed to set for four days before testing.

Test 6 (29.06.98-03.07.98)

Test 6 is a repetition of Test 5 except that the entire hole was filled with the mix and this was also allowed to set for four days.

The following section gives a more detailed description of the results obtained for each test.

2.3 Tests in the Kariba Dam Model

2.3.1 General

The tests were run in the same way as the tests carried out by Mr Blake. In his report, Mr Blake explains why the model materials were tested with constant flows instead of a hydrograph simulating the actual spillage experienced at Kariba Dam since its commissioning. Indeed, testing with a hydrograph would have meant that the testing time would have been too impractical because too long. A constant flow was thus applied, using three or four gates, cutting the testing time down to two hours.

The table below gives the prototype and model flows for three and four gates:

GATES OPEN No.	FLOW (PROTOTYPE) m^3/s	FLOW (MODEL) l/s
3	4 750	115,9
4	6 333	154,4

Tests 1, 2, 4, 5 and 6 were run with opening three gates, while Test 3 was run with opening four gates for symmetry reasons as explained below.

During Tests 1, 2 and 3, the depth of scour was measured 10 minutes after opening the gates and then every 15 minutes after that. For Test 4, the depth of scour was measured 10 minutes after opening the gates and then every 30 minutes after that. The depth of scour was measured with the use of a staff.

The results for each test are given hereafter.

2.3.2 Results

Test 1

After four days of allowing the mix to set in the hole, a flow from three gates was passed through the model for a period of two hours. After ten minutes the scour hole had deepened by 0,7 m (actual measurements as opposed to model-scale measurements). After forty-five minutes, the scour had deepened by 1,4 m but thereafter remained constant until the end of the testing period. It was therefore clear that the binder used in this mix was too strong.

Test 2

A flow from three gates was passed through the model for a period of two hours, after both mixes had settled in the hole for three days. It immediately became clear that the jet coming out of the gates was not evenly distributed on the two mixes as two gates were directed onto Mix 2 and one onto Mix 1. During the testing it was also not possible to maintain a normal three gate flow since there appeared to be a shortage of water in the system.

After forty minutes both mixes appeared to be scouring well, the objective being, as described in section 2.2, that the scour reached the cement floor, i.e. a total of 30 cm. The weaker mix (Mix 1) had scoured by 9 cm whereas Mix 2 had scoured by 4,5 cm. However, after running the test for two hours, the scour in Mix 1 had reached only 15 cm, and the scour in Mix 2 only 5 cm.

(The measurements are given in model-scale dimensions.)

Test 3

For symmetry reasons a flow of four gates was passed through the model instead of three. However, problems with the water in the system were also experience and the four gate flow could not be maintained either.

Figure 3 shows photographs of the test being run and of measurements that were made with the use of a staff.

Both mixes appeared to be scouring well. The table below gives the various measurements that were made during the test:

Time after start (mn)	Scour (cm): Mix 1	Scour (cm): Mix 2
10	2	6
25	8,5	11,5
40	10	12
55	15,5	22
70	24	22
85	27	25,5
100	27	28
115	29	29
120	30	30

Mix 2, which was the weaker mix, appeared to be scouring faster than Mix 1. However the final result gave the same scouring depth for both mixes. Both mixes gave a full scour (i.e. down to the cement floor). Figure 4 shows a photograph of the result of the test. It was very noticeable that the scouring occurred in the vicinity of the zinc plate, with the scour hole forming along the plate. The zinc plate therefore most probably highly influenced the scour pattern, as well as the scouring depth. Indeed, once the scour is initiated, the zinc plate by standing out of the mix is influencing the direction of the flow and it is also suspected that, after initiation of the scour, the zinc plate experienced vibrations creating movement in both mixes and thus helping the scour to deepen further. Furthermore, it was thought that the measuring instrument used to assess the scour depth, the staff, also helped the scour to develop.

Therefore, it was decided for the next test to remove the zinc plate and also measure every thirty minutes instead of every fifteen minutes.

Test 4

For this test only one mix was used, therefore a flow of three gates was passed through the model. No problems were experienced with the flow during the testing period. The test confirmed the suspicions that aroused during Test 3: after two hours the scour depth had only reached 17 cm (in model-scale measurements). The table below shows the progression of the scour:

Time after start (mn)	Depth of scour (cm)
10	2,5
40	4,5
70	11
100	14
120	17

The full scour (down to the cement floor) had therefore not been reproduced as in Test 3.

It is to be noted however that the formation of the shape of the scour hole was good. The larger diameter of the scour hole had reached 70 cm and the smaller diameter had reached 56 cm (model-scale measurements). Figure 5 shows photographs of the result of the test.

The results of the testing of the concrete cubes seemed to indicate that the mix used for this test was too weak to test upon. However, the results of Test 4 seem to indicate that it would be appropriate to pursue with an even weaker sand-cement ratio. Therefore, other methods of testing the strength of the mix will need to be investigated.

Test 5

After four days of setting in the model, the final mix matrix appeared fairly porous and also still fairly 'loose', as if the binder had not properly adhered to the aggregate. A flow from three gates was passed through the model after the four day period. After twenty-five minutes, all the coke nuts had been 'washed' out of the hole, with no proper scour pattern that seemed to be forming. As only a third of the hole was filled with the mix and since all the coke nuts had been washed out, it was decided to test that same mix filling the entire hole (Test 6).

Test 6

After four days of setting in the model, the mix had a similar appearance to that of Test 5. A flow from three gates was passed through the model for a period of two hours, after which the coke nuts had nearly all been washed out of the hole. There was also no indication of a proper scour pattern developing. The results of the test are shown in Figure 6 (before and after the model run).

Test 5 and Test 6 therefore could not reproduce the results obtained by Mr Blake, although the proportions of the mix were identical to those used by Mr Blake. This is however not totally unexpected. Indeed, the major drawback with using cement as a binder is its inconsistency. Cement has a very long setting time (28 days). To ensure the best results, the mix should have been allowed to set for 28 days, as also noted by Mr Blake. However, it is not feasible to only carry out one test per month, so a much shorter period had to be used. But whilst the 28-day strength can be reasonably well reproduced, the initial setting rate is extremely susceptible to the water content, the ambient temperature and to the quality of the actual cement (which can vary dramatically). Therefore, one cannot expect, after a four-day setting period, to reach a uniform strength for a mix submitted to two different sets of ambient conditions.

2.3.3 Summary

This section gives a summary of all the tests performed during the period from 19.02.98 until 03.07.98. The results and test conditions are summarized in the following table.

SUMMARY OF TEST CONDITIONS AND RESULTS						
Tests	**Mix proportions sand:cement:water (by volume)**	**Strength (MPa)**	**Temperature: average value (°C)**	**Sand moisture**	**D_{50} (mm)**	**Scour (after 2 hours of test) (model-scale: cm)**
Test 1	1: 0,06: 0,14	0,358	24,8	-	0,44	2
Test 2						
Mix 1	1: 0,03: 0,144	0,127	20,9	1,6	0,44	15
Mix 2	1: 0,04: 0,115	0,428	20,9	1,6	0,44	5
Test 3						
Mix 1	1: 0,024: 0,134	0,181	21,6	1,83	0,44	30
Mix 2	1: 0,021: 0,13	0,102	21,6	1,83	0,44	30
Test 4	1: 0,021: 0,124	-	20,6	1,35	0,44	17

SUMMARY OF TEST CONDITIONS AND RESULTS		
Tests	**Mix proportions lime:cement:coke nuts:water (by volume)**	**Scour**
Test 5	1: 0,04:7,2:0,26	full, but no scour shape
Test 6	1: 0,04: 7,2:0,26	full, but no scour shape

3. CONCLUSIONS AND RECOMMENDATIONS

This study should be seen as a continuation of the research work undertaken by Mr K R K Blake on plunge pool scour reproduction in physical hydraulic models. The initial aim was to carry out further investigations into a particular material tested by Mr Blake which consisted of a cement based binder with coke nuts (degassed anthracite) as the aggregate. However, the aggregate needed for this material, the coke nuts, became temporarily unavailable. It was therefore concluded that a different approach would be undertaken: it was decided to carry out other tests using a different material: a cement based binder with sand and water.

Only the mix tested in Test 4 seemed to give a good enough reproduction of the shape of the scour hole formed below the Kariba Dam, which was one of the criteria that had to be met so that the mix would prove satisfactory. The depth of scour was however still insufficient, indicating that a weaker sand:cement mix should be tested upon.

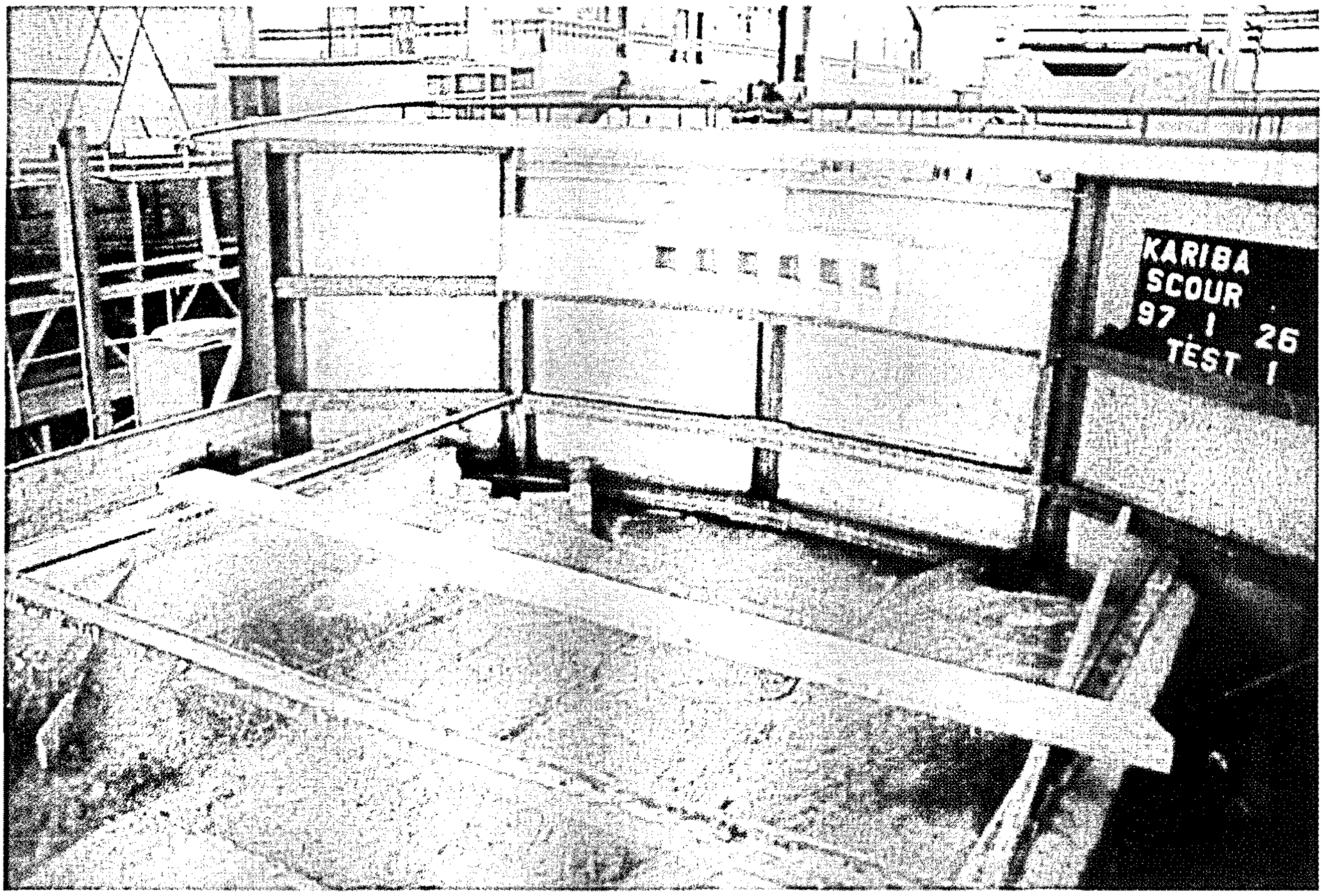
Both sand and cement are easily obtainable materials, relatively cheap and easy to work with. The sand should be fine (in the order of 200 μm) so that it would be easily transported after scour and not form a berm wall downstream and influence the flow pattern. However, the problem arising with testing weaker mixes than the one in Test 4 is that the traditional method of testing the strength of the mix (by crushing cubes) is not suitable. Indeed, because the mix is very weak, the cubes tend to disintegrate before they can be crushed. If that direction of testing weaker sand:cement mixes is to be followed, other strength test methods of the mixes, such as soil sampling techniques, would need to investigated. Contact has been made with the University of Stellenbosch in this regard: confined compressive tests were suggested. Those tests would be done on cylindrical samples and carried out in triaxial cells, with the samples put in a membrane with a confining pressure around the samples. The samples would then be compressed to obtain their compressive strength. The cost of this method is however expensive.

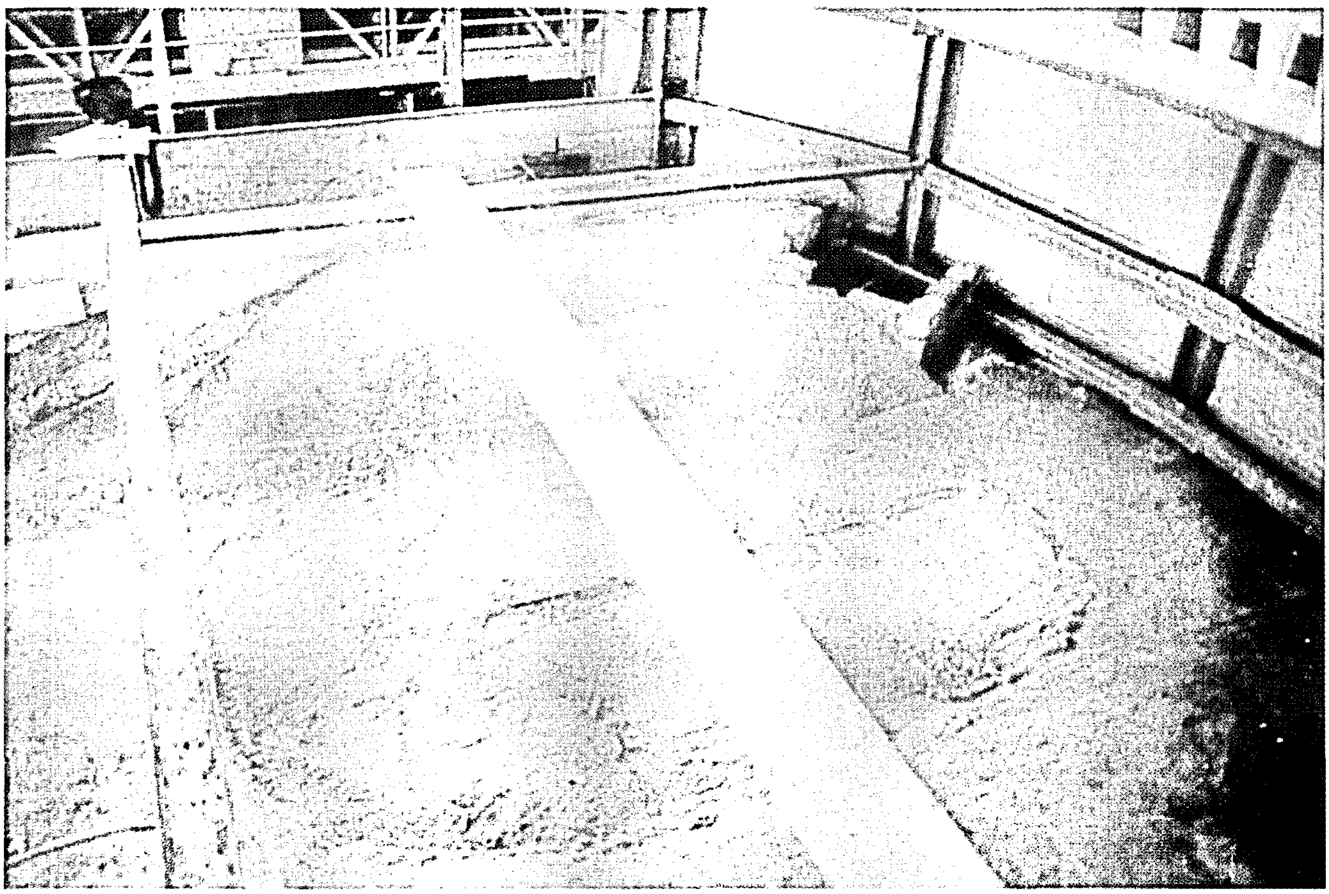
When coke nuts became available it was decided to pursue in the direction indicated by Mr Blake's promising results. The results, however, proved to be very different from those obtained by Mr Blake, due to the inconsistency of the cement used as the binder. This inconsistency is the main factor responsible for the non-repeatability of the tests using coke nuts. It would therefore be tempting to try and use a more reliable binder, however, Mr Blake's research in that direction indicated little success. It should also be concluded that although the coke nuts seemed to give good results during the experiments conducted by Mr Blake, this material seems to be difficult to obtain and could therefore prove unsuitable for further research.

The future?

- With regard to the technical aspect of this study, we recommend that a promising direction would be to continue research on cohesive materials or fine sands:cement mixes. This research would be suitable for a post-graduate student.

- With regard to the organisational aspect, we recommend a more intensive cooperation between universities and CSIR to mobilize maximum expertise level in this field of research. In this regard, advice from the Plunge Pool Scour Steering Committee is essential.



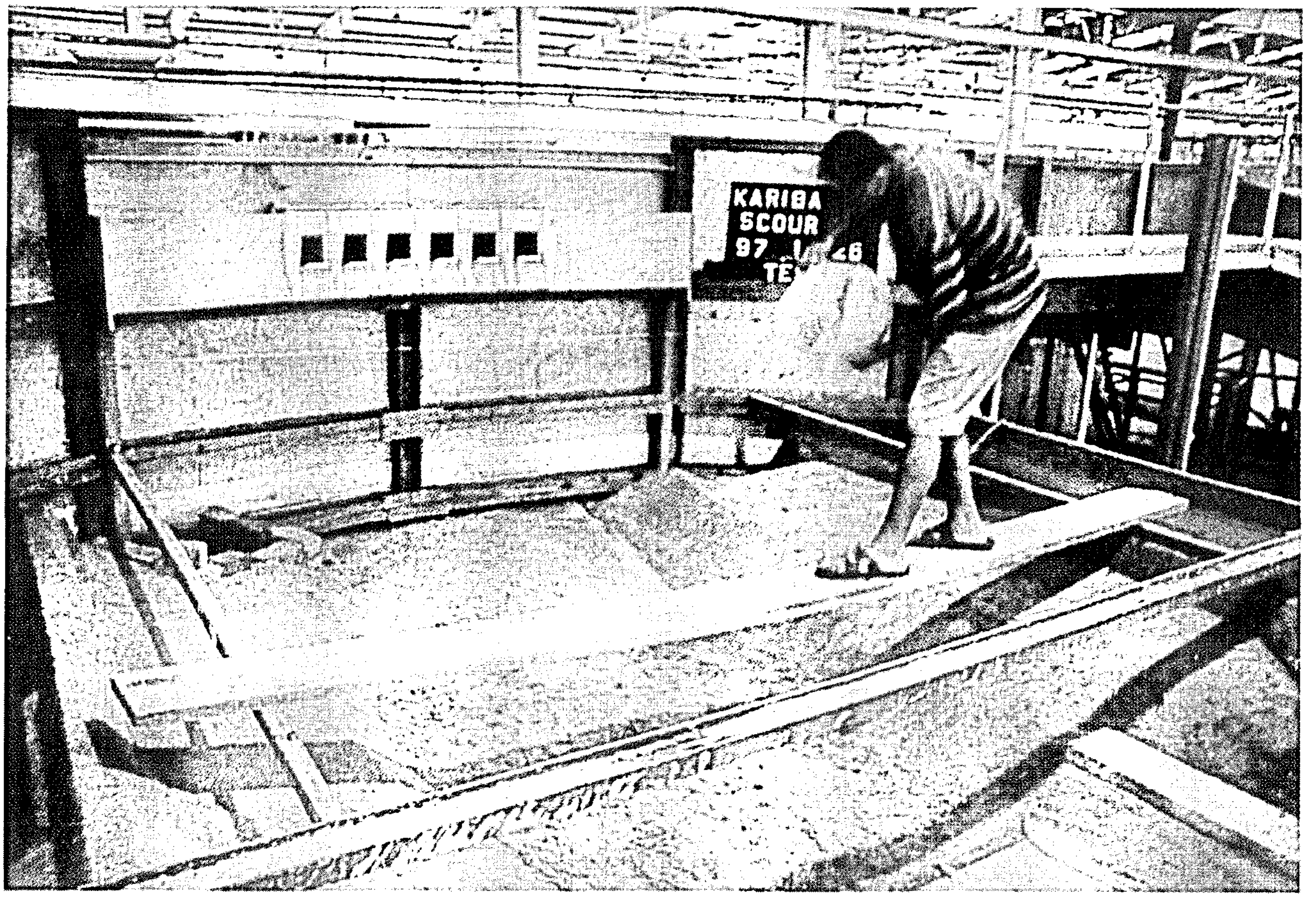


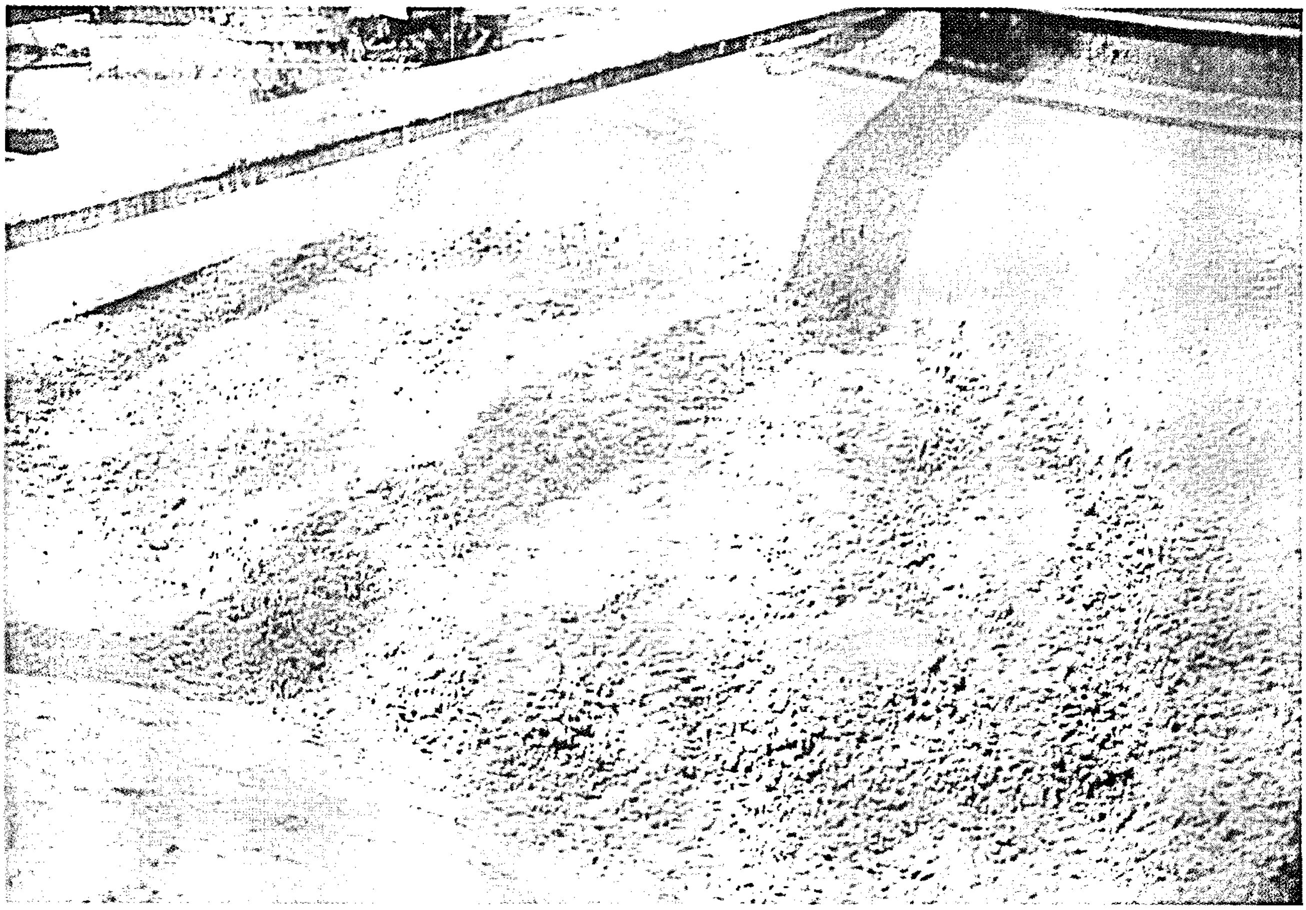
CSIR
ENVIRONMENTEK

KARIBA DAM MODEL

PLUNGE POOL SCOUR:
TEST 3

FIGURE
1



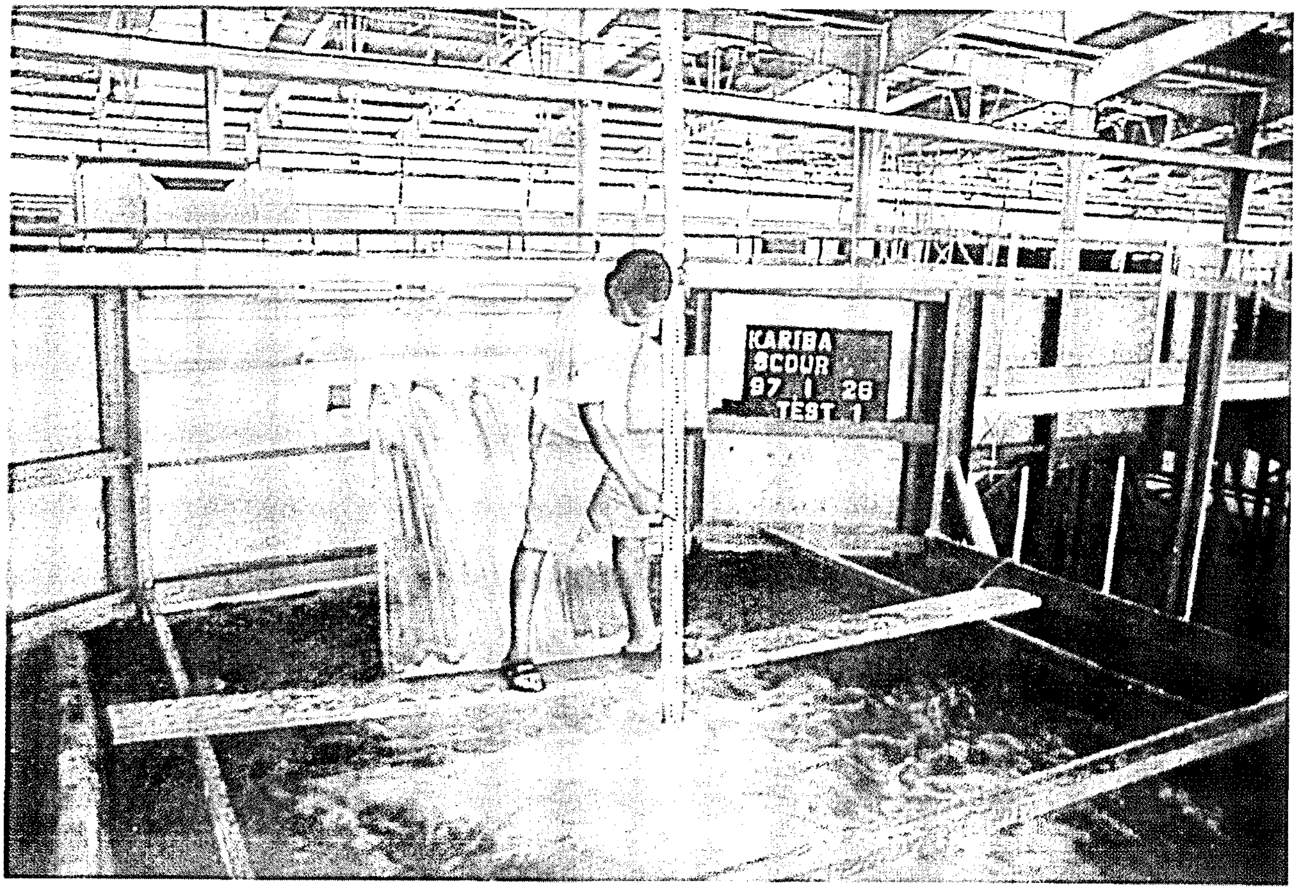


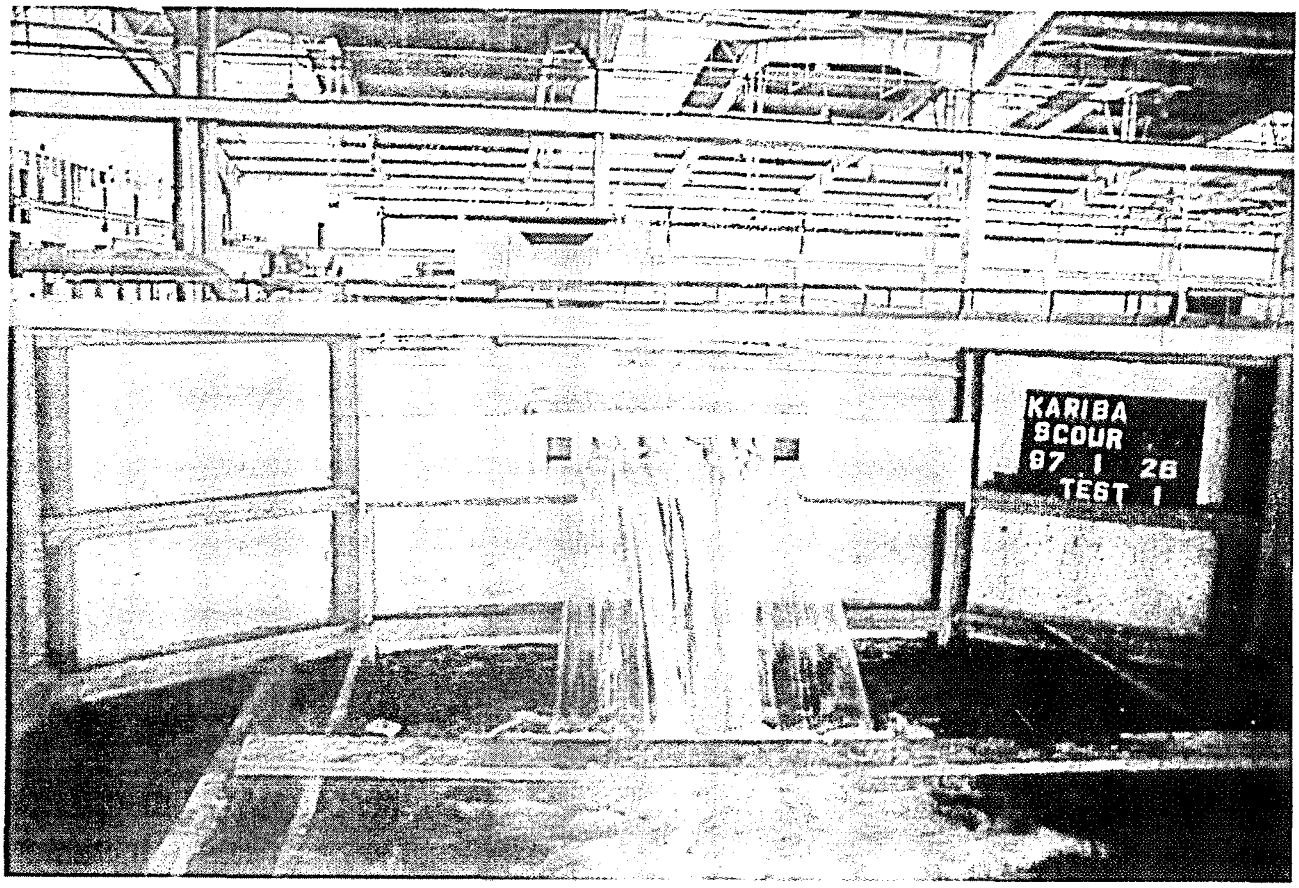
KARIBA DAM MODEL

PLUNGE POOL SCOUR:
TEST 4

FIGURE

2





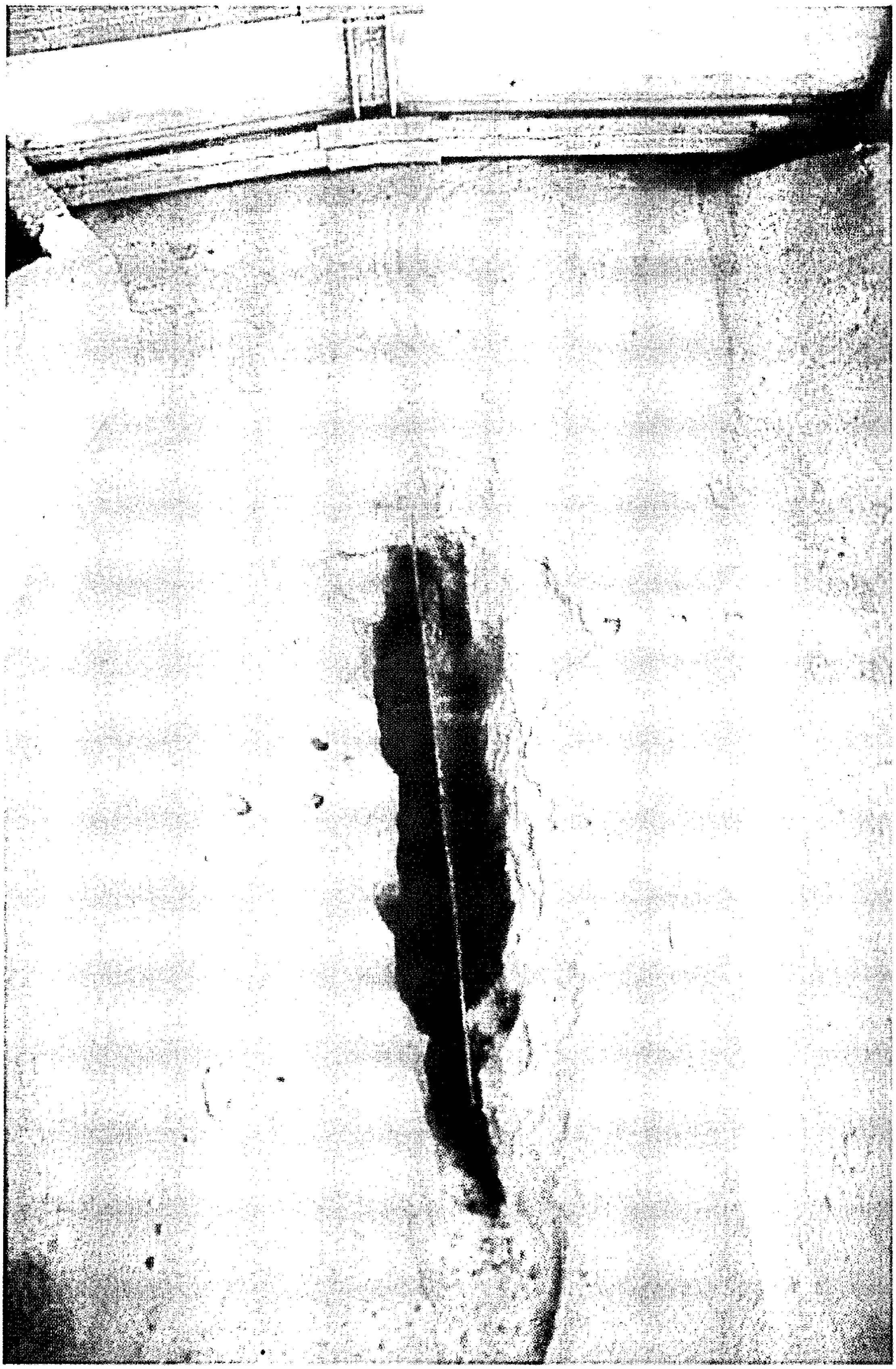
ENVIRONMENTEK

KARIBA DAM MODEL

PLUNGE POOL SCOUR:
TEST 3: MEASUREMENTS

FIGURE

3

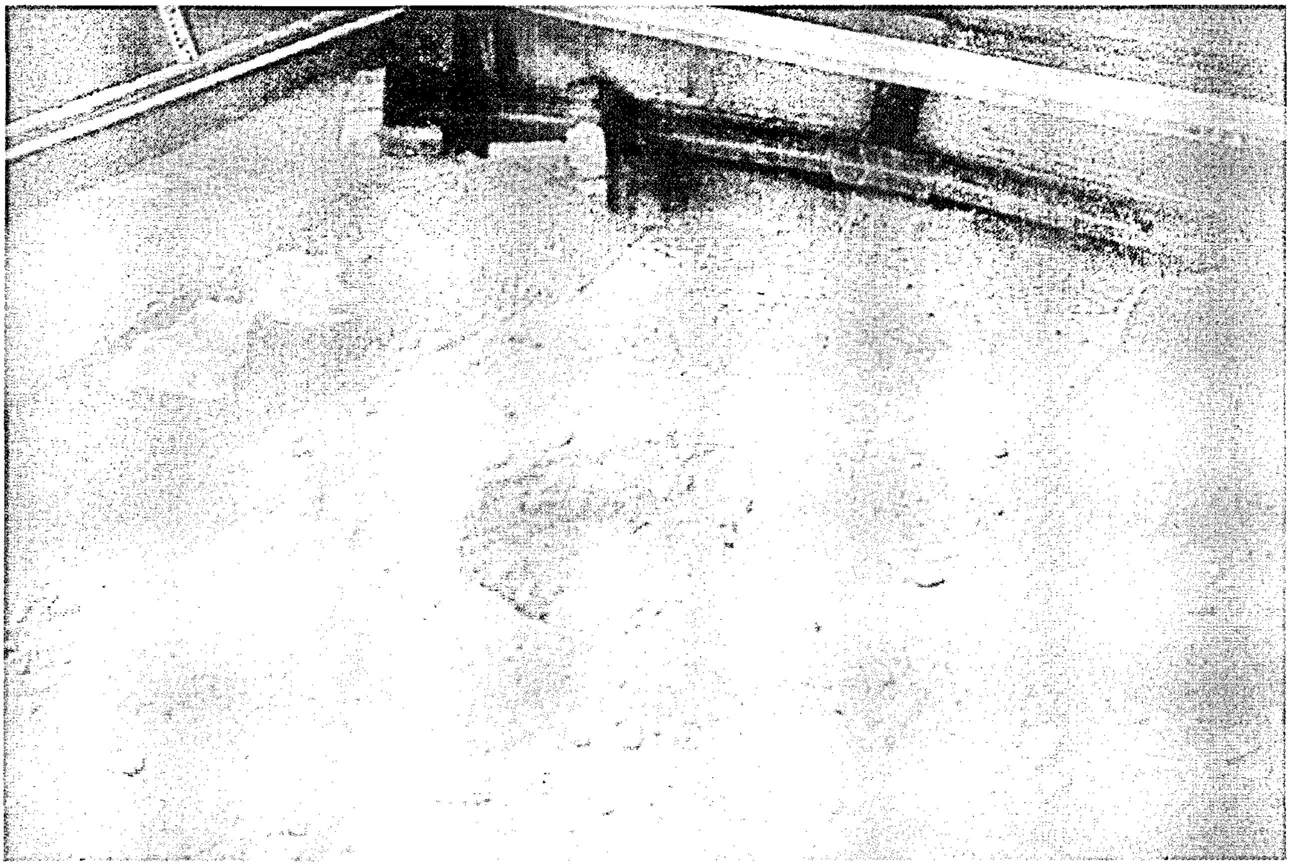


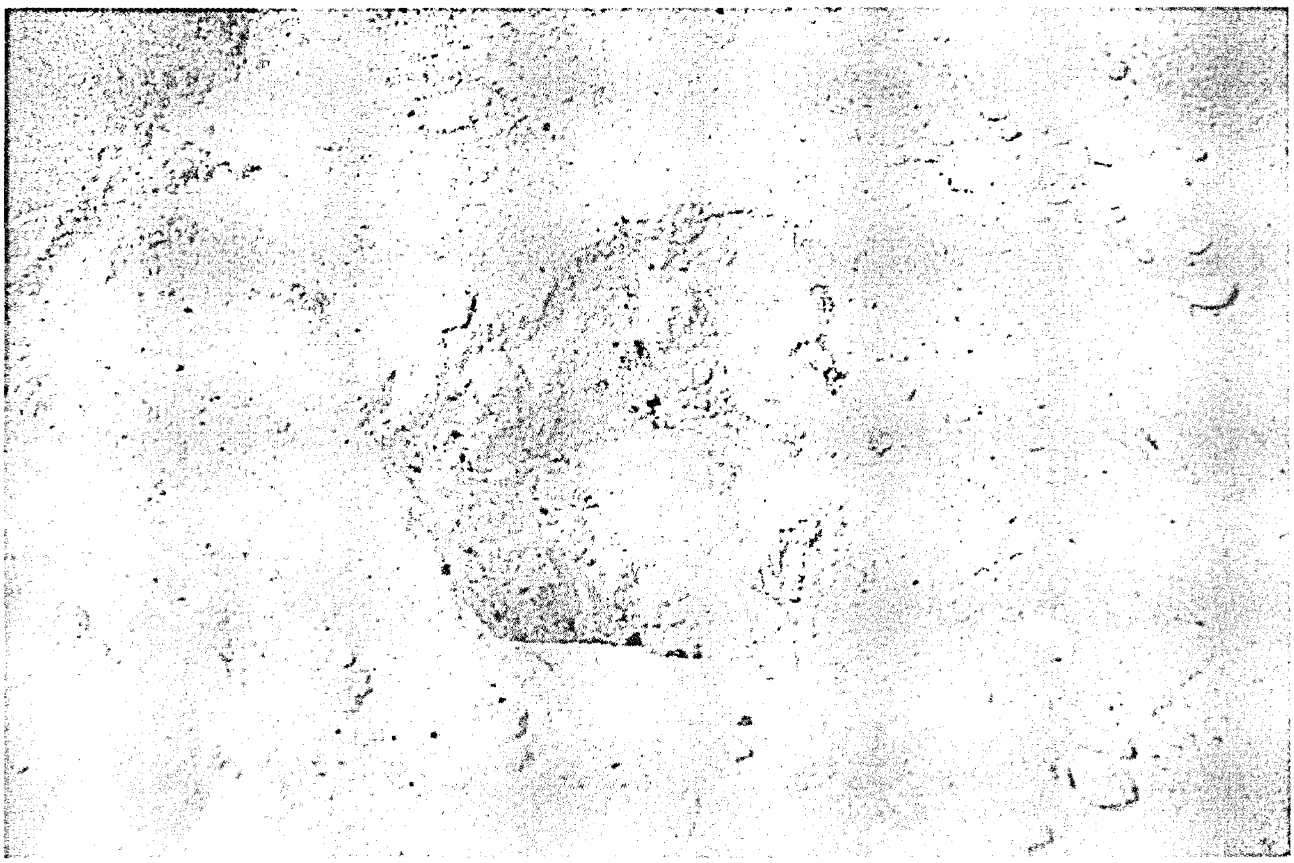
CSIR ENVIRONMENTEK

KARIBA DAM MODEL

PLUNGE POOL SCOUR:
TEST 3: RESULTS

FIGURE 4



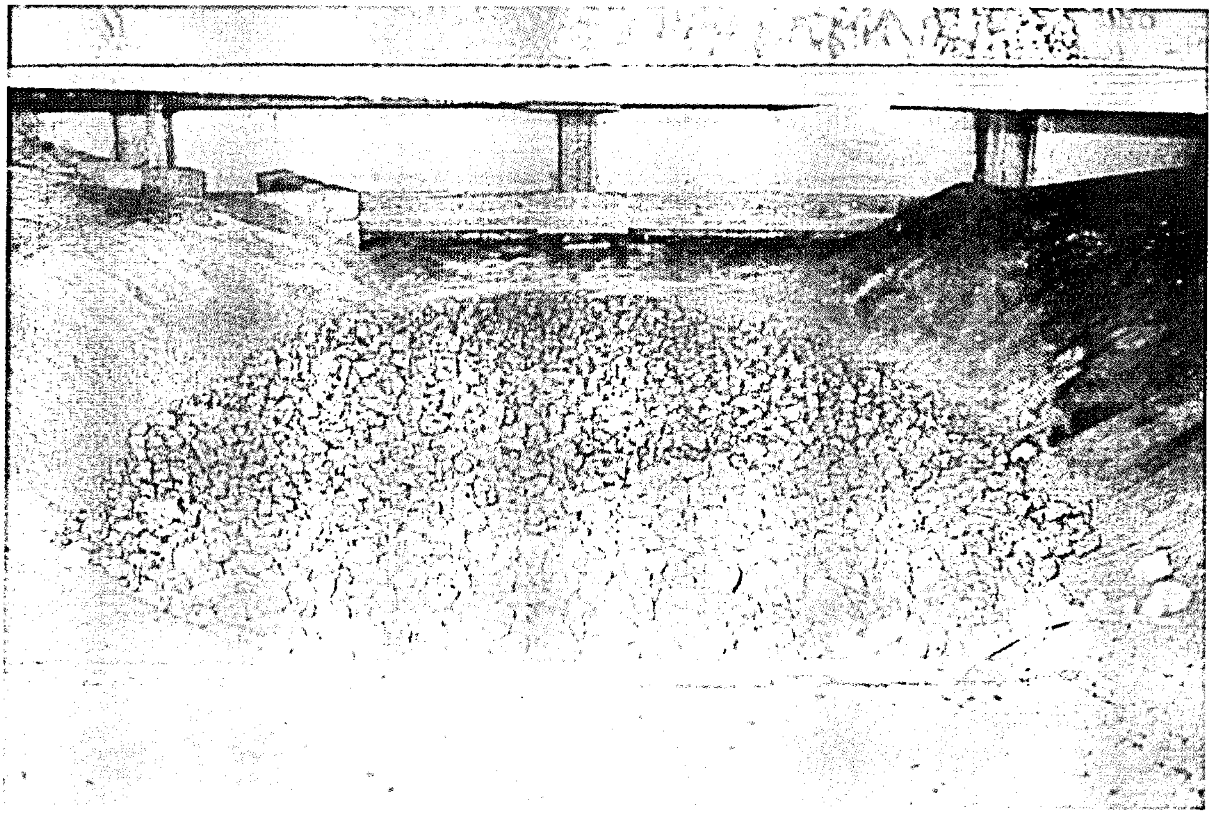


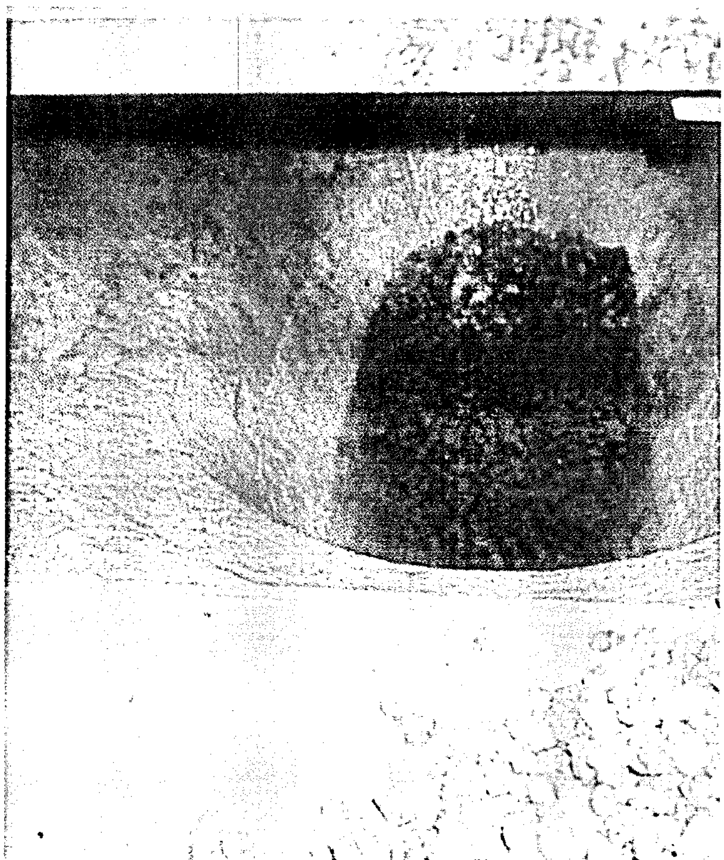
CSIR ENVIRONMENTEK

KARIBA DAM MODEL

PLUNGE POOL SCOUR:
TEST 4: RESULTS

FIGURE
5





CSIR ENVIRONMENTEK

KARIBA DAM MODEL

PLUNGE POOL SCOUR:
TEST 6

FIGURE

6

PLUNGE POOL SCOUR REPRODUCTION IN PHYSICAL HYDRAULIC MODELS

(K. Blake)

EXECUTIVE SUMMARY

Free trajectory jets are often used as a means to pass excess flood waters from a dam to the river below. This inevitably results in the formation of a scour hole downstream of the dam wall. If this scour hole is extensive, as is often the case, it may endanger the dam's ancillary structures and even the stability of the dam wall itself. It is, therefore, imperative that the ultimate size of the scour hole is known at the design stage of a dam.

The mechanism of scour, however, is an extremely complex process due to the influence of various hydraulic, hydrological and geological factors. Some of these factors are difficult to assess theoretically and limited success has been had when trying to reproduce them experimentally. It is not surprising, therefore, that a reliable method of predicting the size and depth of scour holes is still not available. Although some empirical formulae have been proposed to predict such scour holes, in most cases, scale physical models are constructed to try to reproduce what will ultimately happen in the prototype. This research project set out to investigate the most appropriate materials to be used to model the bed rock below a dam wall, for use in such physical models. At the same time it also looked at the reliability of the available empirical formulae proposed to date.

Nineteen scour prediction formulae were used to predict the scour depths at various known sites and the results compared with the actual scour data. Analysis of these results showed that while certain formulae worked with some degree of success at certain given sites, problems always arose when using the formulae at other sites. It would appear that the proposed formulae do not take into account all the factors influencing the formation of the hole, or that the data used to derive the formulae did not represent the ultimate scour depth. In hindsight, it would seem that a reasonably accurate generic scour prediction formula is not possible. The engineer is thus forced to turn to scale modelling to obtain a more accurate prediction of the depth of the scour hole.

To test the proposed model bed materials, an existing dam had to be modelled in the laboratory, such that its scour hole could be used as a criterion for the tests. However, due to the lack of appreciable depth of scour below suitable dams within South Africa, no sites could be found for this purpose. Therefore, the search had to be extended further north and the Kariba Dam on the Zambezi River was chosen. The scour hole below this dam has extended almost 75 m below the river bed and annual surveys have been kept of the progress of the hole since the dam's construction.

During the extensive model testing phase of this study, a number of aspects were highlighted concerning the accurate reproduction of scour holes. The most important of these was the sensitivity of the formation of the scour hole to the flow patterns in, and around, the eroding hole. The final scour depth was found to vary greatly if the local topography was not exactly reproduced in the model, or if the material being eroded from the hole was not ejected as it would be in reality. Coupled to the latter was the effect of the confinement of the developing hole on the plunging jet. This confinement tended to enhance the erosive power of the jet by concentrating its energy onto a smaller area, resulting in a much deeper scour hole. Thus, the use of cohesive materials to represent the bed rock in the model, which could sustain the steep sides expected in reality, gave deeper, more realistic scour holes. Testing with non-cohesive materials lead to shallower scour holes that, if taken as correct, may result in an under designed system.

This study investigated a wide range of possible materials to represent the bed material in scale models. The criterion against which the materials were tested was their ability to reproduce the scour hole formed at the Kariba Dam. It was found that the loose material collecting in the scour hole adversely affected the ultimate depth of the hole. In reality this material would erode into pieces small enough to be ejected from the hole and transported downstream. To resolve the problem, a very light aggregate (coke nuts) was used, that could easily be ejected from the scour hole once it was freed from the binding matrix. Thus, the scour hole maintained a shape similar to that expected in prototype, thereby ensuring that the flow patterns in and around the hole were correct.

None of the materials tested gave ideal results, however, the one material that did behave remarkably better than all the others consisted of a cement based binder with coke nuts (degassed anthracite) as the aggregate, in the ratio (by mass);

LIME	CEMENT	COKE NUTS	WATER
1	0,04	7,2	0,26

Due to financial limitations, further investigations into this material could not be carried out. It is, therefore, recommended that this be taken up at a later stage to further the ability to accurately predict the depth of scour holes below dams.

SUMMARY OF FINDINGS AND CONCLUSIONS

(K Blake)

B.1 Findings

B.1.1 Prediction formulae

The comparisons of scour predictions with model and actual scour data have shown that none of the formulae put forward to date, work when applied to differing sites. Furthermore, Mason found that the formulae that gave the best results for model studies were not the same ones that gave the best results for prototype studies. It appears that some degree of accuracy may be obtained only in cases where the site conditions are very similar to those from where the proposed formula was derived in the first place. In his work to rationalise the proposed scour prediction formulae, Mason (1985) came up with two distinct formulae, one that best suited the available model data and a general formula based on model and prototype data. The latter of these two formulae gave particularly poor results. This could be accounted for, at least in part, by the fact that the prototype scour data does not necessarily represent the **ultimate scour depth**.

Yildiz and Üzücek (1994), reporting on scour predictions for dams in Turkey, stated that there is a general lack of suitable scour data available in the literature for the necessary research work to be carried out on scour prediction formulae. The writer does not deny the value of such data, but warns that care must be taken when using it to derive prediction formulae. Very rarely, if ever, are we certain that the ultimate scour depth has been achieved at a given site, due mainly to the time dependency of the mechanism of scour. This factor appears to have been neglected in most of the predictive formulae derivations put forward to date. A fact that could lead to under estimation of actual ultimate scour depths if these formulae are later used in practise.

This leads to the question of the usefulness of prototype data. From the facts stated above, it would appear that such data would be meaningless unless;

- it is known that the ultimate scour depth had been reached

or

- the rate of erosion is accounted for in the prediction formulae.

The first of these is in most cases, if not all, not known with any certainty. Therefore, to use available prototype data to produce working scour prediction formulae, the effect of time must be taken into account. This in itself, presents considerable problems, unless very detailed data are available, which is not normally the case.

It seems, therefore, that a reasonably accurate generic scour prediction formula is not possible. The large number of variables at each site, the unavailability of prototype ultimate scour depth data and the lack of time related scour data, make the feasibility of such a formula negligible.

If this is true, then the engineer must turn to other methods of determining the expected scour depths below a dam overflow spillway or flipbucket. Two possibilities are available. The first is to consult some form of rating index for the given site conditions. To the writer's knowledge no working model of such an index is available, however, there has been some work in this direction recently, based on the Kirsten Rippability Index. The Kirsten Index, originally put forward for use in earthworks, gives a relative "strength" value to the bed rock at the site, a factor that should be related to the depth of scour. At least two authors have tried fitting known data to this index, without modifying it at all, with some success. Of course such a system cannot predict an actual depth of scour, only an indication of what scour may occur, for example; no scour, slight, moderate and extensive. However, with some subjective judgement an experienced engineer would be able to derive at least an indication of the expected scour depth. This method is promising, but still requires much work before it can be used reliably in the field.

The second method of determining the scour depth below a dam overflow spillway or flipbucket, is to use a scale model. Despite certain shortcomings, this is still the most reliable method available to date. Further details of using models are discussed in Section 7.1.2.

B.1.2 Scale modelling

This study has highlighted a number of aspects with regard to using scale models for the prediction of scour depths below a dam overflow spillway or flipbucket.

Probably the most important of these is the effect of **jet confinement**. Whilst it has been known for some time that confining the jet may cause increased scour depth, this study has highlighted just how dramatic that effect can be. In this case, when no confinement was inhibiting the jet after entering the plunge pool, the jet was unable to remove a layer of blocks placed at a given level. However, when an artificial confining hole was constructed, reducing the plan area of the downstream river by about 50 %, the same jet was able to remove the blocks with ease. At only half the original flow, the jet removed all the available blocks, leaving a scour hole almost twice as deep as with no confinement. As the bottom of the hole struck the fixed floor of the model, the ultimate scour depth could not be determined in this case.

The packing density, uniformity, regularity of placement and joint fill material between the blocks are important criteria influencing the ultimate scour depth. As these cannot always be replicated correctly in the model environment, the probability of obtaining reliable results from the use of blocks was limited. Thus, although blocks tended to reproduce the mechanism of scour in the model correctly, this study concentrated on using cohesive binders with aggregate as the bed material. This was easier and quicker to place and gave a homogeneous matrix, which in turn meant that repeatability of results was achievable in the model.

Further to this, it was shown that the **local topography** also plays an important role in determining the shape and therefore depth of the scour hole. It would seem that the flow patterns in the vicinity of the scour hole have a major influence on the scour mechanism. Altering the downstream topography slightly, can influence the flow patterns in and around the scour hole, which in turn significantly alters the potential scour depth. It is therefore, extremely important to construct the model to exacting detail, thereby ensuring the correct flow patterns and therefore accurate scour reproduction. Failure to do this could make the model results meaningless and in the worst scenario, lead to an under-designed system.

Related to this problem is that of the formation of a bar from the eroded material being deposited just downstream of the scour hole. In reality much of this material would erode into finer material that could easily be washed further downstream. As this does not occur in a model, the material is dumped closer to the scour hole and the resultant bar is larger than would form in reality. This in turn alters the flow conditions which have been shown to affect the ultimate scour depth. It is, therefore, necessary to stop the model at regular intervals and reduce the size of the bar forming in front of the scour hole, such that the correct flow conditions are maintained throughout the test.

In a model study investigating the build up of the bar at Kariba Dam (SOGREAH 1962), several tests were run to determine a suitable model material for the investigation. It was found that the scour hole formed during these tests was too large and the hole was finally fixed in mortar to the correct size, to ensure that the flow patterns within the scour hole were similar to the prototype. Using this setup, a suitable non-cohesive material was found that gave bar heights similar to those found in prototype. This material was a mix of 50 % 2 to 9 mm aggregate with 50 % 0,5 to 4 mm sand. It was recognised that a shortcoming of this method was that the erosion rate of the scour hole could be taken into account and its effect on the bar formation was unknown. A material that satisfied the scour hole formation as well as the bar formation was not found during the tests.

This study found that the use of cohesive materials gave deeper scour depths than non-cohesive materials in most cases. This appears to be due to the confinement effect on the jet. With a cohesive material it is possible for the forming scour hole to have steep or vertical sides. Thus a confined hole is formed, that concentrates the jet's energy, ultimately causing deeper scour. On the other hand, in the case of loose bed material, the slopes of the sides of the scour hole are limited to the angle of repose of the material (normally close to 30°). Thus there is very little confinement of the jet, little to no concentration of the energy and therefore a considerably shallower scour hole than could be expected in reality.

The shape of the scour hole was shown to have a large influence on the final scour depth, in that it determines the confinement and hence concentration of the jet. In most of the tests, although the upstream end of the scour hole tended to form as required, the downstream end always had a flatter slope than the prototype. This was primarily due to the aggregate from the loose bed material remaining in the hole and collecting on the downstream end, at its natural angle of repose. This is an unnatural situation, as in the normal course of events, this material would have been broken and eroded into pieces small enough to be removed from the scour hole altogether.

It would be, of course, impractical to stop the model tests regularly and remove this build up of material. Two other solutions can be considered, either using a material that will break down after it is loosened, or using an aggregate that is considerably lighter than the material it represents and will be removed from the model scour hole by the action of the jet. The second of these is obviously the most practical solution and was used in this study (Section 6.6).

The choice of cohesive bed materials is discussed in Section 7.1.3.

B.1.3 Bed material

This study investigated a wide range of cohesive materials to represent the bed material in scale models. The criterion against which the materials were tested was their ability to reproduce the scour hole formed at the Kariba Dam. None of the materials tested gave ideal results, however, one material did behave remarkably better than all the others. This material used a cement based binder with "coke nuts" as the aggregate.

It is thought that the main reason that this material outperformed the others was simply that no loose aggregate collected in the scour hole to distort the natural flow patterns of the plunging jet. During the formation of a scour hole in reality, blocks of bed rock that are lifted out of place by the action of the jet, soon break into pieces small enough to be ejected from the scour hole. Thus the hole itself is kept clear of debris and therefore there is only the actual shape of the hole to influence the flow characteristics of the impinging jet. At Kariba Dam, rock fragments found downstream of the scour hole do not exceed, on average, about 300 mm in diameter. This would translate to a stone of 4,3 mm diameter or a mass of only 0,1 g in the model, which would easily be removed by the jet. In the model, however, the 19 mm granite aggregate used for most of the tests was too heavy for all of it to be removed from the hole. The material that remained tended to collect at the downstream end of the hole, forming a shallow slope at this end. Obviously, this totally distorted the flow patterns within the hole and hence the scour potential of the jet. When using coke nuts as the aggregate, no material remained in the hole, giving a situation closer to that experienced in reality and therefore, a more realistic scour hole.

Four materials were tested for use as binders and these are summarised in Table B.1.

Based on their binding capabilities, only the plaster-of-paris and the cement were found to be viable options. However, the plaster-of-paris had a major draw back in that its setting time was extremely quick (less than five minutes). This led to problems throughout the testing, as the material had to be mixed in small batches, which made achieving a homogeneous matrix very difficult. It had to be concluded that, from a modelling point of view, the plaster-of-paris was impractical and could not be considered.

TABLE B.1 : COMPARISON OF BINDING MATERIALS

BINDER	COST	NOTES
Bentonite	Moderate	Tended to be too weak unless completely filling the voids between the aggregate.
Cretestone	Moderate	Tended to break down after long periods of saturation in the model.
Plaster-of-paris	Expensive	Very difficult to obtain homogeneous mix due to the extremely quick setting time.
Cement	Cheap	Slow to set, but easy to work with. Water content critical to strength.

Thus, the most practical binder tested was ordinary portland cement. It should be noted that as the setting of cement is a hydration reaction, the quantity of water in the mix and available throughout the initial setting stages, plays a major role in the final strength of the mortar. This had to be controlled very carefully in the model tests to ensure consistent results. The major draw back of using cement is its long setting time. Ideally to ensure the best results, the mixture should have been allowed to set for about twenty-eight days, at which time the cement has gained most of its strength. However, it is not feasible to only be able to carry out one test a month, so a much shorter period had to be used. In this case a setting period of four days was chosen, which allowed a test per week to be carried out. Unfortunately, during the early stages of setting, the ambient temperature also plays a role in the strength of the material as the temperature accelerates or decelerates the rate of hydration of the cement. This is a problem that would have to be monitored in the laboratory.

Although none of the mixes tested reproduced the Kariba Dam scour hole particularly well, the mix that gave the best results was found to be (by mass);

LIME	CEMENT	COKE NUTS	WATER
1	0,04	7,2	0,26

It is of interest to compare this with the mix put forward by Johnson (1977), that is;

CHALK	CEMENT	AGGREGATE	WATER
1	0,02	6,0	1,6

The major difference between the two mixes is the water to cement ratio. This study proposed a ratio of 1:6, while Johnson used a ratio of 1:74,2 which would have given a very fluid binder. It was found during the testing that if the binder was too fluid, it tended to flow to the bottom of the bed material, leaving a nonhomogeneous matrix. Johnson's binder would also have been considerably weaker than that of this study. Besides having only 50 % of the cement content, the very high water content would have lead to a weaker product. This may have been compensated for, to some extent, by the increased binder to aggregate ratio, that is, more of the voids between the aggregate will have been filled. However, as found in this study, filling the voids tends to change the mechanics of erosion such that the build up of hydrostatic pressure within the matrix does not occur in the model. Thus Johnson's mix would not be suitable for the reproduction of scour by a falling jet.

It was found that the test scouring of the samples was not always representative of what occurred in the model. Although this may have been due to a number of reasons, it is thought that a major contribution was the size of the jet used on the samples, as opposed to that in the model. The smaller jet may have allowed the pore pressures within the matrix to dissipate before they built up enough to cause any damage to the samples. Thus it would have appeared that the material was stronger than it actually was.

B.2 Conclusions

From the results of this study, the following conclusions can be drawn:-

- Recent formulae put forward to predict the depth of scour are no more accurate than those originally proposed in the 1930's. In general, scour prediction formulae tend to be unreliable and should be used with caution.

- It would appear that physical model tests are still the most reliable method of determining the expected depth of scour below a dam overflow spillway or flipbucket.

- The confinement effect of the scour hole has a concentrating effect on the plunging jet that results in a much deeper scour hole. If this confinement is not correctly reproduced in the model, the depth of scour hole produced may be shallower than that occurring in reality, which could lead to an under-designed system.

- The material used to represent the bed rock in the model must be able to reproduce the near vertical sides of the scour hole achieved in nature (for example, by incorporating a cohesive binder). Failure to do so reduces the confinement effect on the jet, resulting in shallower scour holes than will actually occur.

- As the flow patterns in and around the scour hole play a major role in determining the ultimate scour depth, it is imperative that the downstream topography of the river and dam appurtenant structures are modelled in detail.

- Due to the uncertainties involved in trying to represent the natural bedrock in the model, some subjective judgement may be required by the engineer, to assess if the modelled scour hole is realistic.

- The scour depths obtained from model studies should only be regarded as an indication of the probable scour depths, not absolute values.

- The material used to represent the natural bedrock gave the best results when a very light aggregate was used in the mix (in this case coke nuts). This ensured that the loosened aggregate could be ejected from the model scour hole (as it would be in reality) and did not interfere with the flow patterns, and hence scour development, in the hole.

- The most practical binder found, of those tested, was a mix of ordinary portland cement and hydrated lime. The only draw back of using cement was the setting time, for which a period of four days was used as standard in these tests.

- Although no bed material was found that gave identical scour to that found at Kariba, the mix that gave the best results was (by mass);

LIME	CEMENT	COKE NUTS	WATER
1	0,04	7,2	0,26

B.3 Recommendations Regarding Further Research

Although this study has found a potential material for use in models to reproduce the scour hole below a dam overflow spillway or flip bucket, it has also brought to light a number of other aspects. It is recommended that these be investigated in further studies.

- The concept of linking the Kirsten Number (modified or not) with the depth of scour at existing sites may lead to a very simple scour depth indicator for future proposed sites. Although it is likely that adequate suitable data may not be available at this stage to achieve a complete analysis, this approach has merit for the future.

- The material found for model testing in this study needs to be refined further to determine the best possible mix. Consideration could be given to looking into a quicker setting cohesive binder, so that long delays between tests can be avoided. This may also help to alleviate the problem of the actual binder strength at the time of testing, which will ensure similarity between tests.

- A second dam needs to be modelled to check that the test material can be used at differing sites. It is, however, foreseen that finding a suitable site with a significant scour hole and adequate data on the scour hole formation may be a problem.

PHASE 3

LOOSE AGGREGATE METHOD

by

WJ van Aswegen

PLUNGE POOL SCOUR REPRODUCTION
IN PHYSICAL HYDRAULIC MODELS:
LOOSE AGGREGATE METHOD

SCOPE OF THIS REPORT

This report concludes the current investigations into the most appropriate material to be used to reproduce the plunge pool scour below dam outlet structures in scaled physical hydraulic models, specifically for modelling bedrock below a dam wall. The ideal material will accurately reproduce the physical characteristics of a plunge pool scour hole with regard to shape and depth.

This report is a continuation of research by the CSIR, which started in 1992. The Steering Committee recommended that this project should be extended to undertake additional model tests based on a method developed during a physical model study of the Maguga Dam carried out at the University of Stellenbosch Laboratories. This was accepted by the Water Research Commission during 1999. The model tests were carried out in the Kariba Dam model, at the CSIR laboratory in Stellenbosch during the period January to March 2000.

W J van Aswegen carried out the model study and compiled this report, with guidance from Prof A Rooseboom and Mr J Moes.

D Phelp
Manager: Coastal Engineering and Data Management

March 2000

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1. INTRODUCTION

The Kariba Dam, on the border between Zambia and Zimbabwe, was chosen for the plunge pool scour investigation, since no dam in South Africa has appreciable depth of scour. The scour hole below the dam extends almost 75 m below the old river bed (CSIR, 1995), which is about 80 m below the tail water level during floods.

The earlier investigations focused on cemented, lightweight material, such as coke nuts. However the repeatability in using such material was not very good (CSIR,1998). The binder elements namely cement and lime, once mixed with water, could not be controlled.

The method developed during the Maguga dam physical model study (G.M. Goody and A. Rooseboom, 1998) to reproduce the physical characteristics of a plunge pool scour hole was subsequently proposed and applied in the Kariba dam model.

The method to predict the scour hole dimensions was developed during a physical model study of the Maguga Dam in the hydraulic laboratory of the University of Stellenbosch. The project was initiated by KOBWA (Komati Basin Water Authority) and Sigma Beta consulting civil engineers conducted the model study. The method focuses on the prediction of the scour hole dimensions by utilising an aggregate of fairly homogeneous loose material representing the size of eroded elements in prototype, with no binding agent as part of the mix. This material is placed downstream of the dam wall in the area influenced by the waterjet from the overtopped dam wall. Various flood conditions are modelled to predict the resulting scour hole. As part of the method a control test (Goody and Rooseboom, 1998), with a scour hole consisting of solid steep slopes and an erosive bed, based on the predicted dimensions is conducted to confirm scour depth and flow patterns.

The aim of this extension of the WRC project is to apply the method applied in the Maguga study as accurately as possible in the Kariba dam model. The results of the model tests, conclusions drawn and recommendations made are reported here.

2. MODEL TESTS

2.1 General

The model tests were conducted in the CSIR physical modelling facility in Stellenbosch. The Kariba 1:70 scale model as described, calibrated and tested for conformity (CSIR, 1995 and CSIR, 1998) was used without any alterations (Figure 2.1).

A total of seven tests based on the method applied in the Maguga study were conducted in the Kariba model. Beside the scour prediction the tests had as additional objectives:

- to demonstrate repeatability
- to optimise aggregate size
- to evaluate the effect of relative density of the scour material
- to evaluate the effect of varying flood conditions

The basic method applied throughout the tests is described in Section 2.2, followed by a detailed description of individual tests completed.

2.2 The Loose Aggregate Method

In the Maguga dam model study a scour hole is pre-excavated to a pre-determined empirical depth. The hole is filled with loose aggregate. Flood conditions modelled up stream of the test area is released over the dam wall structure onto the test area. The related scour process is then surveyed over time.

The method is based on a "conservative" approach. The areas of conservatism include the flood modelled, the size of the aggregate in the model and the final scour dimensions. Experience gained during the Maguga model study (Goody and

Rooseboom, 1998) established the 1: 200 year flood condition (with a slightly smaller than predicted aggregate size and the largest pre-determined empirical scour dimensions) to be the basis for modelling and predicting the downstream plunge pool scour below an outlet structure. This scour hole is finally modelled as a pool with solid side slopes and an erosive bed. Previous tests flood conditions are again modelled and the erosive bed's scour monitored. The final scour depth recorded in the constructed scour pool is noted as the maximum scour depth predicted.

For instance, it was found that the predicted maximum flood (PMF) scour depth result for Maguga was only 6 m deeper than that predicted with the 1: 200 year flood (Goody and Rooseboom, 1998). Thus the 1 : 200 year flood condition is ideal for test purposes.

The prototype aggregate size expected to be worked out of the scour hole is the basis for choosing loose aggregate for modelling purposes. It would however, be preferable if the material grading would, in general, reflect a smaller than estimated grading than the expected prototype material.

In the Maguga model study, the predicted dimensions of the scour hole were finally modelled as a scour hole with steep solid side slopes and an erosive base. This modelling was done to confirm flow patterns and scour depth.

The only deviation from the method applied in the Maguga study during the tests conducted in the Kariba model was the prediction of the initial scour dimensions with an erosive riverbed. The Kariba scour hole was an existing feature constructed out of concrete with solid sides and base. This feature meant that some of the steps of the method applied in the Maguga study were bypassed in favour of immediately testing a scour hole with solid side slopes.

The model test duration was between 2 to 4 hours (model time). This caused build-up of scour material immediately downstream of the hole in the shape of a berm across the riverbed. This berm caused unwanted effects such as raised water-levels, disrupted flow patterns and localized velocity disconformities. To eliminate these effects the berm was frequently removed from the model during tests. This was justified as the scour hole at Maguga would be pre-excavated as a borrow pit.

2.3 Kariba Test Procedure

Before a test commenced the existing scour hole was filled with test material and levelled off. Profile levels were surveyed and photographs were taken before a predetermined constant flood condition was modelled.

The test was divided into 30-minute cycles after which:

- A profile was surveyed
- Photographs were taken
- Berm material was removed

The test duration was determined by one of two factors, namely, time to reach equilibrium of scour or a 4-hour time limit. Due to the limited time allowed for model tests (two weeks) a limit of 4 hours was placed on each test. After 4 hours the scour was evaluated and a decision to carry on or to abort the test was made.

Levels of the scour hole were surveyed downstream of the dam wall along the axis of the dam. There were 11 survey points at 16 m interval (prototype, scaled at 1:70). The first survey point was on a solid wooden profile used during construction of the riverbed. This served as a quality control point. Table 2.1 indicates the chainage of each survey point along the axis of the dam.

Survey point No	Chainage (m)
0	1 610
1	1 594
2	1 578
3	1 562
4	1 546
5	1 530
6	1 514
7	1 498
8	1 482
9	1 466
10	1 450

Table 2.1 Survey point chainage values

This basic test procedure was followed throughout the investigation.

2.4 Test Description

The aim of the tests was to realise, in the model, a scour depth equivalent to the prototype depth of 80 m below the tail water level of 305 m asl.

All dimensions are noted as prototype values. If reference is made to model values it is noted as such. The model scale is 1: 70 and the model is operated in accordance to the Froude law (CSIR,1995). Four types of model materials were tested namely a 19 mm loose aggregate size, a 13 mm loose aggregate size, a 13 mm loose aggregate size and coke nuts and a 9 mm loose aggregate size.

Prototype aggregate diameters modelled are derived from the average weight of a sample of 20 particles of model material and the assumption that a solid sphere represents the prototype aggregate shape transported out of the scour hole. Table 2.2 lists the model particle weights, the average model particle weight and the average prototype diameter the model material represent.

Particle Number	19 mm Aggregate Weight (g)	13 mm Aggregate Weight (g)	9 mm Aggregate Weight (g)	Degassed Anthracite Weight (g)
1	9.10	2.63	0.84	51.51
2	6.19	2.55	0.67	34.82
3	6.15	1.88	1.00	14.58
4	11.10	1.71	1.61	26.49
5	15.40	1.55	1.20	27.92
6	4.60	2.93	0.82	31.70
7	12.00	0.62	0.57	24.37
8	7.30	2.49	1.03	36.25
9	8.50	1.14	0.65	37.65
10	9.40	2.13	0.39	25.91
11	12.00	1.04	0.46	24.37
12	8.73	3.05	0.25	24.87
13	3.68	1.23	0.55	20.69
14	8.01	1.01	1.17	15.38
15	2.64	1.58	1.35	16.74
16	5.82	0.59	0.54	8.40
17	5.15	1.27	0.45	15.00
18	3.06	0.76	0.36	10.50
19	4.56	1.77	0.16	12.44
20	4.95	0.49	0.69	35.44
Avg. weight	**7.42**	**1.62**	**0.74**	**24.75**
Relative Densities	**2.6**	**2.6**	**2.6**	**1.7**
Prototype Diameters	**0.966 m**	**0.452 m**	**0.305 m**	**2.183 m**

Table 2.2 Aggregate particle weight

2.4.1 19 mm Aggregate: Tests 191 and 192

The first tests were conducted with the scour hole filled with loose 19 mm aggregate previously tested in the Kariba model (CSIR, 1995). The tests lasted for a model period of 4 hours (Test 191) and 3 hours (Test 192) respectively. The flow rate was kept constant at the maximum allowable flow permitted through three gates during both

Test 191 and Test 192.Test 192 was a duplicate of Test 191 to demonstrate repeatability in the model.

Table 2.3 summarises the relevant variables (in prototype values) of Tests 191 and 192 with this material.

Description	Test 191	Test 192
Relative density	2.6	2.6
Average stone size diameter	0.966 m	0.966 m
Scour hole levels before test	363 m asl	362 m asl
Flow (3 gates open)	4 750 m^3/s	4 750 m^3/s
Test duration	33.5 hrs	25 hrs

Table 2.3 Test variable description of 19 mm Aggregate: Tests 191and 192

2.4.2 13 mm Aggregate: Test 131

After evaluating the performance of the 19 mm stone, a 13 mm aggregate was tested for a model period of 3 hours. The flow was kept constant with three gates open and the scour hole filled with 13 mm loose aggregate.

Table 2.4 summarises the relevant variables (in prototype values) of Test 131 with this material

Description	Test 131
Relative density	2.6
Average stone size diameter	0.452 m
Scour hole levels before test	359 m asl
Flow (3 gates open)	4 750 m^3/s
Test duration	25 hrs

Table 2.4. Test variable description of 13 mm Aggregate: Test 131

2.4.3 13 mm Aggregate and Nuts: Test 13N1

The 13 mm stone test was followed by a test were the scour hole was filled with aggregates of different relative densities. The bottom of the scour hole was filled with a 21m thick degassed anthracite nuts layer (CSIR, 1995). The final 35 m thick layer of aggregate was the same 13 mm stone described in Section 2.4.2. The 13 mm stone filled the scour hole to the pre-test level.

A steady flow rate of Q = 4 750 m³/s (three gates open) was tested for a model time duration of 4 hours after which a steady flow rate of Q = 6 333 m³/s (four gates open) was tested for a model time of 0.5 hours. The total duration of the test was 4.5 hours model time, with two distinct steady flow rates. Levels of the scour profile were surveyed continuously without resetting the profiles.

Table 2.5 summarises the remainder of the variables related to this test.

Description	**13 mm aggregate**	**Nuts**
Relative density	2.6	1.7
Average stone size diameter	0.452 m	2.18 m
Scour hole levels before test	361 m asl	326 m asl

Table 2.5 Test variable description of 13mm aggregate and Nuts: Test 13N1

2.4.4 9 mm Aggregate: Tests 91, 92 and 93

For Test 91 the pre-test level represented only the bottom part of the total scour depth. It was filled with 9 mm aggregate to 327 m asl.

In Test 91 a steady flow rate of Q = 4 750 m³/s (three gates open) was tested for a model time duration of 1.5 hours, after which an increased steady flow rate of Q = 6 333 m³/s (four gates open) was tested for a model time of 1 hour. The total duration of Test 91 was 2.5 hours model time, with two distinct steady flow rates. Levels of the scour profile were surveyed continuously without resetting the profiles.

In Test 92 the 9 mm aggregate was filled to a level of 357 m while there were some changes to the scour hole. The scour hole was changed by adding four mesh panels square on to the flow direction at evenly spaced positions (Figure 2.2). The size of the mesh allowed individual particles to travel through it. If however the aggregate was packed tight against the mesh, the mesh openings would clog up and the particles would pack a vertical face against the mesh. Table 2.6 indicates the chain position of each mesh panel.

Mesh panel No	Chainage (m)
1	1 554
2	1 538
3	1 522
4	1 506

Table 2.6 Mesh panels chain values

The flow was kept steady at Q = 6 333 m³/s (four gates open) for a model time test duration of 2 hours. Levels of the scour profile were surveyed continuously without resetting the profiles.

Test 93 for the 9 mm aggregate was a repeat of Test 92 but now with a steady flow of Q = 4 750 m³/s (three gates open) and a model time duration of 3 hours.

Table 2.7 summarises the relevant variables related to these tests.

Description	**Test 91**	**Test 92**	**Test 93**
Relative density	2.6	2.6	2.6
Average stone size diameter	0.305 m	0.305 m	0.305 m
Scour hole levels before test	327 m asl	357 m asl	357 m asl

Table 2.7 Test variable description for 9mm Aggregate: Test 91, 92 and 93

3. TEST RESULTS

3.1 19 mm Aggregate: Tests 191 and 192

The 19 mm aggregate had previously been tested in the Kariba model (CSIR, 1995). This model material represented the prototype aggregate size believed to be ejected out of the scour hole by the water jet during the scour process(CSIR, 1995)..

Test 191 and Test 192 had a three-fold purpose. They would establish a continuation of work done previously, serve as starting point for the Maguga method, and prove repeatability in the model.

The history of scour development over the full test period is presented in Figure 3.1 for Test 191 and in Figure 3.2 for Test 192. In both Figures 3.1 and 3.2 the solid profile and the model material profile before the test indicate how the test would start off and what the test aimed to achieve with respect to scour. The solid profile represent the prototype scour hole, measured to be 80 m below tail water level.

The profiles surveyed after each time cycle (30 min model time) clearly showed the volume of material moved by the water jet. A scour hole formed in the region of chainage 1 524 m (Figures 3.1 and 3.2) while a berm build-up was visible on the down-stream side of the scour around chainage 1 610 m (Figures 3.1 and 3.2). The berm formed part of the survey after which it was removed to form a levelled-off profile before the next time cycle commenced.

In both Figures 3.1 and 3.2 the first 30 minutes of the test produced the most scour in one single time cycle. After this initial scour the profiles slowly progressed in a vertical direction towards a point of what seemed like equilibrium. After this initial vertical scour the greatest amount of scour occurred in the horizontal plane, both in an upstream and downstream direction. The profiles tended to establish the natural angle of repose of the material within the first hour (model time) and thereafter this slope was moved horizontally as more model material was ejected from the scour hole.

After 4 hours Test 191 was stopped. The profiles surveyed at 3.5 hours and 4 hours (Figure 3.1) were virtually duplicates of each other, indicating that equilibrium had been reached. This result was the basis for the decision on the 3 hour limit as placed on Test 192.

Table 3.1 summarises the maximum depths surveyed per time cycle and the chainage where each depth occurred for Test 191 and Test 192. The depth of the scour was measured below a tail water-level (TWL) of 385 m. asl.

Prototype Time	Chainage Test191	Scour Test 191	Depth below TWL	Chainage Test 192	Scour Test 192	Depth below 385m asl.
(hr)	(m)	(m asl)	(m)	(m)	(m asl)	(m)
0	-	363.00	22.00	-	362.00	23.00
4.18	1 514	332.52	52.48	1 514	335.11	49.89
8.37	1 498	330.00	55.00	1 514	330.77	54.23
12.55	1 498	327.62	57.38	1 530	328.46	56.54
16.73	1 530	326.36	58.64	1 530	327.48	57.52
20.92	1 530	324.61	60.39	1 498	328.39	56.61
25.1	1 498	323.14	61.86	1 530	325.73	59.27
29.28	1 514	323.49	61.51	-	-	-
33.47	1 514	323.49	61.51	-	-	-

Table 3.1 Maximum scour depths for 19 mm Aggregate: Tests 191 and 192

The level surveyed before Test 191 commenced, was at 22 m below 385 m asl. After 4.18 hours Test 191 reached a depth of 52.48 m and after 25.1 hours 61.86 m below 385 m asl. Between 25.1 hours and 33.47 hours the maximum depth measured for Test 191 was between 61.51 m and 61.86 m. Most of the maximum depths measured for each profile during Test 191 occurred between chainage 1 498 m and 1 530 m.

The level before Test 192 was measured at 23m below the 385 m asl. During the first 4.18 hours Test 192 reached a depth of 49.89 m and at 25.1 hours 59.27 m. As in Test

191 most of these depths occurred between chainage 1 498 m and 1 530 m.

Figures 3.3 to 3.8 compare Test 191 profiles with Test 192 profiles taken after each time cycle. At the end of the first time cycle (4.18 hours) the two profiles had the same shape although the levels were not comparable. As the test duration progressed the profiles of Test 191 and Test 192 became more compatible after each survey. The profiles of Test 191 and Test 192 had the closest resemblance after 16.73 hours (Figure 3.6).

3.2 13 mm Aggregate: Test 131

The 19 mm aggregate test results, with a maximum scour depth of 61.86 m (Table 3.1) prompted the investigation of a smaller aggregate size to achieve the scour depth of the Kariba scour hole. A 13 mm loose aggregate was chosen and the loose aggregate method applied during **Test 131**.The purpose of **Test 131** was to achieve the scour depth of the Kariba scour hole, that is, a depth of 80 m.

The history of scour development over the full test period is summarised in Figure 3.9. The profiles surveyed after each time cycle (30 min model time) clearly show the volume of material moved by the water jet. A scour hole formed in the region of chainage 1 530 m (Figure 3.9) while a berm build-up was visible on the downstream side of the scour hole. The berm formed part of the survey, after which it was removed to form a levelled-off profile before the next time cycle commenced.

In Figure 3.9 the first 30 minutes of the test produced the most scour in one single time cycle. After this initial scour the profiles slowly progressed in a vertical direction towards a point of what seemed like equilibrium. After this initial vertical scour the greatest amount of scour occurred in the horizontal plane both in an upstream and downstream direction. The profiles tended to establish the natural angle of repose of the model material within the first hour (model time) and thereafter this slope was moved horizontally as more material was worked out of the scour hole. After 3 hours (model time) the horizontal scour process reached the solid slopes of the Kariba scour hole (Figure 3.9).

The test was limited to 3 hours (model time) because the vertical equilibrium seemed to have been established at this time(Figure 3.9).

Table 3.2 summarises the maximum depths surveyed per time cycle and the chainage where each depth occurred for **Test 131**. The depth of the scour was measured below a tail water-level (TWL) of 385 m. asl.

Prototype Time	Chainage Test 131	Scour Test 131	Depth below 385m asl
hrs	(m)	(m asl)	(m)
0	-	360.00	25.00
4.18	1 514	331.54	53.46
8.37	1 514	328.53	56.47
12.55	1 514	326.99	58.01
16.73	1 514	324.61	60.39
20.92	1 482	323.28	61.72
25.1	1 546	322.09	62.91

Table 3.2 Maximum scour: depths for 13 mm Aggregate: Test 131

The level surveyed before Test 131 commenced was at 25 m below 385 m asl. After 4.18 hours Test 131 reached a depth of 53.46 m and after 25.1 hours 62.91 m. Most of the maximum depths measured for each profile during Test 131 occurred at chainage 1 514 m. The maximum depths measured for 20.92 hours and 25.1 hours occurred at chainage 1482 m and 1546 m respectively; close to the solid sides of the Kariba scour hole.

3.3 13 mm Aggregate and Nuts: Test 13N1

The **13 mm Aggregate: Test 131** achieved a scour depth of 62.91 m (Table 3.2) after 25.1 hours. It was decided to test a combination of 13 mm stone and degassed anthracite placed in two separate layers as **Test 13N1**.The aim of this combination of materials was to achieve the depth of scour in the Kariba scour hole. The test duration was set at 4.5 hours (model time) to further investigate the change of maximum scour depth from chainage 1 514 m to chainage 1 546 m (Table 3.2) in **Test 13N1**.

The history of scour development over the full test period is illustrated in Figure 3.10. The profiles surveyed after each time cycle (30 min model time) clearly showed the volume of material moved by the water jet. Initially a scour hole formed in the region of chainage 1 530 m (Figure 3.10) while a berm build-up was visible on the downstream side of the scour hole. Later in the test the scour hole moved downstream of the initial scour to chainage 1 562 m. In Figure 3.10 the first 30 minutes of the test produced the most scour in one single time cycle. After this initial scour, the profiles slowly progressed in a vertical direction towards a point of what seemed like equilibrium. After this initial vertical scour the greatest amount of scour occurred in the horizontal plane both in an upstream and downstream direction. The profiles tended to establish the natural angle of repose of the model material within the first hour (model time) and thereafter this slope was moved horizontally as more material was ejected from the scour hole. After 2.5 hours (model time) the horizontal scour process reached the solid slopes of the Kariba scour hole (Figure 3.10). Between 2.5 hours and 4.5 hours (model time) the vertical scour process again became dominant at a new deeper point of scour formed close to the solid sides of the Kariba scour at chainage 1 562 m.

Table 3.3 summarises the maximum depths of scour surveyed per time cycle and the chainage where each depth occurred for **Test 13N1**. The depth of the scour was measured below a tail water-level (TWL) of 385 m. asl.

Prototype Time	Chainage Test 13N1	Scour Test 13N1	Depth below 385m asl
hrs	(m)	(m asl)	(m)
0	-	361.00	24.00
4.18	1 530	332.52	52.48
8.37	1 514	328.39	56.61
12.55	1 514	326.36	58.64
16.73	1 530	326.64	58.36
20.92	1 530	324.19	60.81
25.1	1 530	319.92	65.08
29.28	1 562	317.33	67.67
33.47	1 562	316.07	68.93
37.65	1 562	307.27	77.73

Table 3.3 Maximum scour depths for 13 mm Aggregate and Nuts: Test 13N1

The level surveyed before the test commenced was at 24 m below 385 m asl. After 4.18 hours the scour reached a depth of 52.48 m and after 37.65 hours it reached 77.73 m. Most of the maximum scour depths measured for each profile during the test occurred at chainage 1 514m and 1 530 m. The maximum scour depth measured for 29.28 hours to 37.65 hours however occurred at chainage 1 562 m. Between 33.47 hours and 37.65 hours there was a sudden increase in scour depth from 68.93 m to 77.73 m. The time of 37.65 hours marked the end of a steady flow of Q = 4 750 m^3/s tested. The last period of the test between 33.47 hours and 37.65 hours had a flow rate Q = 6 333 m^3/s hence the increase in scour depth.

3.4 9 mm Aggregate: Tests 91, 92 and 93

The **13 mm Stone and Nuts: Test 13N1** achieved a scour depth of 77.73 m (Table 3.3) after 37.65 hours and two types of steady flow rates. The position where this maximum scour depth occurred was of some concern (Figure 3.10). The maximum scour depth occurred at the down stream interface of the solid profile and the fill material. It was therefore decided to test a loose aggregate which represented aggregate of common size found downstream of the Kariba dam along the riverbed (CSIR, 1995), as **Test 91**.The aim of this test was to achieve the depth of scour in the

Kariba scour hole with aggregate of a size found on-site. The test duration was set at 2.5 hours (model time) since the scour hole was only filled to 327.7 m asl.

The history of scour development over the full test period is presented in Figure 3.11. The profiles surveyed after each time cycle (30 min model time) clearly showed the volume of material moved by the water jet. The scour occurred mainly at chainage 1 562 m close to the solid side of the Kariba scour.

Table 3.4 summarises the maximum depths surveyed per time cycle and the chainage where each depth occurred for **Test 91**. The depth of the scour was measured below a tail water-level (TWL) of 385 m. asl.

Prototype Time	Chainage Test 91	Scour Test 91	Depth below 385m asl
Hrs	(m)	(m asl)	(m)
0.00	-	328	57
4.18	1 562	317	68
8.37	1 546	317	68
12.55	1 562	308	77
16.73	1 562	307	78
20.92	1 546	303	82

Table 3.4 Maximum scour depths for 9 mm Aggregate: Test 91

The level surveyed before the test commenced was at 57 m below 385 m asl. After 4.18 hrs the scour reached a depth of 68 m and after 20.92 hours it reached 82 m. Most of the maximum scour depths measured for each profile during the test occurred at chainage 1 546 m and 1 562 m. The time of 12.55 hours marked the end of a steady flow of Q = 4750 m^3/s tested. The last period of the test between 12.55 hours and 20.92 hours had a flow rate Q = 6 333 m^3/s.

The objectives of **Test 92** and **Test 93** were to evaluate the increased flow rate to scour depth development and secondly to improve the slope angle of scour development by introducing mesh panels in between the 9mm aggregate .

The flow rate was kept steady at Q = 6 333 m^3/s for **Test 92**. The history of the scour development over the full **Test 92** period is summarised in Figure 3.12. The scour occurred mainly at chainage 1 562 m close to the solid side of the Kariba scour.

Table 3.5 summarises the maximum depths surveyed per time cycle and the chainage where each depth occurred for **Test 92**. The depth of the scour was measured below a tail water-level (TWL) of 385 m. asl.

Prototype Time	Chainage Test 92	Scour Test 92	Depth below 385m asl
Hrs	(m)	(m asl)	(m)
0.00	-	357	28
4.18	1 562	333	52
8.37	1 546	322	63
12.55	1 562	311	74
16.73	1 562	307	78

Table 3.5 Maximum scour depths for 9 mm Aggregate: Test 92

The level surveyed before the test commenced was at 28 m below 385 m asl. After 4.18 hrs the scour reached a depth of 52 m and after 16.73 hours it reached 78 m. Most of the maximum scour depths measured for each profile during the test occurred at chainage 1 546 m and 1 562 m.

The flow rate was kept steady at Q = 4 750 m^3/s for **Test 93**. The history of the scour development over the full **Test 93** period is summarised in Figure 3.13. The scour occurred mainly at chainage 1 530 m.

Table 3.6 describes the maximum depths surveyed per time cycle and the chainage where each depth occurred for **Test 93**. The depth of the scour was measured below a tail water-level (TWL) of 385 m. asl.

Prototype Time	Chainage Test 93	Scour Test 93	Depth below 385m asl
Hrs	(m)	(m asl)	(m)
0.00	-	357	28
4.18	1 530	330	55
8.37	1 514	331	54
12.55	1 530	328	57
16.73	1 530	329	56
20.92	1 530	325	60
25.1	1 546	323	62
25.1	1 514	323	62

Table 3.6 Maximum scour depths for 9 mm Aggregate: Test 93

The level surveyed before the test commenced was at 28 m below 385 m asl. After 4.18 hrs the scour reached a depth of 55 m and after 25.1 hours it reached 62 m. Most of the maximum scour depths measured for each profile during the test occurred at chainage 1 530 m.

4. EVALUATION OF TEST RESULTS

4.1 19 mm Aggregate: Test 191 and 192

The scour hole formed by the 19 mm aggregate reached a point of equilibrium in the vertical as well as the horizontal direction. This point of equilibrium was reached after 25 hours.

The model reproduced the two tests to such a degree (Figures 3.3 to 3.8) that accurate repeatability is possible in the model. Table 3.1 shows test results, which differ on average by 3.45%, with a maximum difference of 6.6% and a minimum of 1.4 % when comparing Tests 91 and 92 scour depths.

Figure 4.1 shows the scour depth progression with time of Tests 91 and 92. In the first 4.18 hours nearly 84 % of scour depth below the tail water-level was reached during both tests. The curve in Figure 4.1 indicates an equilibrium being reached, with a scour depth of 62 m.

4.2 13 mm Aggregate: Test 131

The scour hole formed by the 13 mm stone reached a point of vertical equilibrium only. This point was reached after 20.92 hours (Table 3.2). The horizontal scour component continued to scour for up to 25.1 hours at which point the profile of the 13 mm aggregate formed a horizontally levelled shape (Figure 3.9). The 13 mm aggregate size introduced a more aggressive horizontal scour mechanism.

Figure 4.2 represents Table 3.2 as scour depth progression with time. The first 4.18 hours accounted for 85 % of the scour depth. The graph indicates a point of vertical equilibrium as the curve reaches to between 62 m and 63 m scour depth.

4.3 13 mm Aggregate and Nuts: Test 13N1

Figure 4.3 describes the scour depth progression with time. The first 4.18 hours indicate a rapid vertical scour producing 67 % of the final scour depth. Between 12.55 hours and 20.92 hours the scour mechanism adjusted to the change from the layer of one type of model material (13 mm aggregate) to the next layer of degassed anthracite. The scour continued until 33.47 hours in the new layer of material. A sudden change in values occurred between 33.47 hours and 37.65 hours when the flow rate increased from Q = 4 750 m^3/s to Q = 6 333 m^3/s.

At 25 hours, with a scour depth of 65.08 m (Table 3.3), the vertical scour equilibrium was reached (Figure 3.10). Between 25 hours and 37 hours the horizontal scour component continued to scour until it reached the solid sides of the Kariba scour (Figures 3.10 and 4.3). Following the route of least resistance the scour mechanism continued down along the solid side deeper into the degassed anthracite layer. This shift in the position of the maximum scour depth from chainage 1530 m after 25 hours to 1562 m is noted in Table 3.3 and Figure 3.10.

The scour between 25.1 hours and 37.65 hours was disregarded for the time being and the vertical maximum scour depth reached was taken as 65.08 m (Table 3.3).

4.4 9 mm Stone Aggregate 91, 92 and 93

Figure 4.4 shows the scour progression with time described in Table 3.4, Table 3.5 and Table 3.6. The initial scour was not captured during **Test 91** because the material level was already at 328 m asl. The point of vertical scour equilibrium was reached after 4.18 hours, where after the horizontal scour mechanism continued towards the solid sides of the Kariba scour hole and down along the solid side to a depth of 82 m below tail water level. The point where the scour started to move along the solid side is clearly visible in Figure 4.4 as a small jump between 8.37 hours and 12.55 hours for **Test 91**. The scour between 8.37 hours and 20.92 hours was disregarded and the maximum scour depth recorded was at 68 m after 8.37 hours.

Test 92 followed the scour development in full. During the first 4.18 hours 67 % of the final scour depth had already been achieved. Vertical equilibrium was reached after 8.36 hours. The mesh panels did not seem to prevent the horizontal scour mechanism, which continued until it reached the solid sides of the Kariba scour. At this point the scour redeveloped and continued along the solid side downwards to a depth of 78 m after 16.73 hours. No significant change in the angle of repose of the model material was noted (Figure 4.5).

Figure 4.4 highlights that **Test 91** results obtained with Q = 6 333 m^3/s and **Test 92** (Q = 6 333 m^3/s and mesh panels) results indicate the same trend of time related scour depth.

The **Test 93** on the other hand indicated some interference in the scour mechanism related to the mesh panels. At 4.18 hours 89 % of the scour depth had been reached. A vertical equilibrium was reached after 20.92 hours. At this point the horizontal scour continued until 29.3 hours.

The maximum scour depth was 62 m. The mesh however seemed to improve the material's angle of repose somewhat (Figure 4.6) during **Test 93**.

Figure 4.4 indicated that Test 93 was inclined to converge to a scour depth of 62 m compared to the 68 m of Test 91.

4.5 Scour Depth Evaluation

The dynamics of the water jet causing the scour are simplified into four distinct mechanisms namely:

- The water jet scour mechanism with a vertical and horizontal component
- Uplift mechanism of the turbulent flow inside the scour hole

- The horizontal transport mechanism of the downstream flow.
- Energy required for break-up

The first mechanism to consider is the water jet and how it impinges on the riverbed. This impingement has a vertical and horizontal scour ability induced by the high pressures at the point of impact.

The next mechanism is the vertical uplift capacity of the turbulent water flow inside the scour hole. The ability to lift suspended scoured material to the surface is important.

The horizontal flow velocity and the ability to transport this suspended material down-stream is the third mechanism acting to produce the scour hole. The turbulence of the water will keep the scoured material in suspension.

The in-situ geology determines the amount of energy required to break-up bedrock into transportable components.

These distinct mechanisms provide an insight into how the model and scour material should react to successfully reproduce the scour hole.

In general, the vertical component of the water jet scour mechanism is more active during the early stages of the tests. This can be linked to the water depth (tail water) or the lack thereof, which acts as a damping mattress.

As soon as the water depth reaches a point were the vertical scour component is damped to such a degree that the horizontal scour component is the route of least resistance, the scour depth progression decreases. This decrease is indicated by the near horizontal part of the curves in Figures 4.1 to 4.4.

The horizontal scour component erodes the material in an upstream as well as a down-stream direction, as the profiles in Figures 3.1 to 3.13 indicate. A horizontal equilibrium was only reached during **Tests 191 and 192**. In all the other tests the scour continued until the solid sides of the Kariba scour was reached. At this point the scour mechanism turned downwards and focused the scour at this new point. This phenomenon was also noted during the Maguga study (Goody and Rooseboom, 1998).

The uplift mechanism of the turbulent water inside the scour hole was for the most part adequate to surface all types of material in suspension. The horizontal transport mechanism, which removed all the suspended material near the surface, was kept realistic by the frequent removal of the berm, which formed downstream during tests.

Table 4.1 summarises the scour depths for the different materials tested. These values are compared to the maximum scour depth of 80 m surveyed at Kariba in 1981(CSIR, 1995) below a tail water level of 385 m asl.

Type of material	Maximum scour depth in metres	Percentage of Kariba max. scour depth reached
19 mm Aggregate	62 m	77.5%
13 mm Aggregate	62 m	77.5%
13 mm Aggregate and nuts	65 m	81.3%
9 mm Aggregate Test 91	68 m	85 %
Test 92	63 m	78.8 %
Test 93	62 m	77.5 %

Table 4.1 Scour depth evaluation

Table 4.1 clearly shows an underestimate of between 15 % and 23 % of the final Kariba scour depth. Not only was the scour depth underestimated, but also the exact shape of the scour hole could not be reproduced correctly.

The equilibrium slopes formed by the loose aggregates tested did not represent the properties of the prototype bedrock, even with the mesh panels in **Tests 92 and 93**, and therefore did not represent the profile along the dam axes. The slopes formed by the loose aggregates tested are at a far more relaxed angle along the dam axes than the measured slopes in prototype, as modelled by the fixed scour hole in (Figures 3.1 to 3.13).

5. CONCLUSIONS

The following objectives were achieved by applying the loose aggregate method in the Kariba model:

- Repeatability of results was established.
- The optimum sized material, producing the best results in the Kariba model, was identified as the 9 mm loose aggregate.
- Relative density of the aggregate only becomes relevant if the model particles represent the expected prototype aggregate weight.
- The flow of Q = 4 750 m^3/s (three gates open) can be increased to Q = 6 333 m^3/s (four gates open) in the Kariba model to achieve the same scour results in less time.

The solid profile of the Kariba model pit resulted in unwanted effects, where the horizontal scour mechanism reached the solid sides of the Kariba profile and continued as a vertical scour mechanism down in to the model material along the solid profile.

The uplift mechanism modelled inside the scour hole was well developed, this can be attributed to the contained flow patterns, caused by the solid Kariba scour hole.

The horizontal transport mechanism, discharging suspended scour material downstream of the scour hole, was of some concern in the model. The velocity profiles and flow patterns downstream of the scour might not develop as in prototype because of the position and shape, in the model, of the weir to regulate the tail water-levels. The weir is placed in the model as an artificial component on the border of the model to simulate tail water levels and might form unrealistic flow convergence close to the down stream end of the scour in the model.

Not one of the materials tested modelled the scour depth and scour profile to complete satisfaction with maximum 85% of depth of scour obtained. The natural angle of repose for all model materials tested was the main characteristic influencing the results. If material characteristics like size and relative density could stay the same and the angle of repose would change to a value closer to that found in the Kariba scour, the scour depth and profile might be achieved. The mesh panels did not succeed in increasing the angle of repose (Figures 4.5 and 4.6).

The loose aggregate method of scour prediction has produced the best results to date in the Kariba model. The loose aggregate method, if applied with the necessary expertise and knowledge of how a physical hydraulic model reacts during the scour process, can produce valuable results and institutes therefore, a promising approach to modelling of plunge pool scour.

6. RECOMMENDATIONS

It is recommended that research into the scour hole development should continue and focus on further development of the loose aggregate method. In addition methods to improve the angle of repose of the model materials must be investigated.

To further investigate the loose aggregate method, the plunge pool area of the Kariba model must be converted into an erosive bed model, downstream of the dam wall, filled with a 1 m to 1.3 m deep loose aggregate layer. A test series to determine the characteristics of the scour hole in the erosive bed should lead to a comparison with the steep-sided Kariba scour surveyed in 1981 and the test results of the as presented test series in this report.

These tests will highlight the effect of the concentration of flow patterns at the solid-sided scour hole and the influence, which this concentration has on the scour depth and shape of the hole. A relationship between model scour depth and expected prototype scour depths can then be developed taking into consideration model effects caused by model test material characteristics, which are different from the characteristics of the prototype material.

To investigate a model material to reproduce the scour exactly, a binder material with a degree of viscosity is recommended. A gel or resin, which does not set into a hard substance, must be used to coat the individual aggregate particles to increase the angle of repose of this model material. The coat must form a tacky surface so that particles stick lightly to one another. The present imbalance of the vertical and horizontal scour mechanism should be carefully assessed in these new tests to determine the strength of the binder material.

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CSIR (1995).Plunge Pool Scour Reproduction in Physical Hydraulic Models. *CSIR Report* EMA-C 95098, Stellenbosch.

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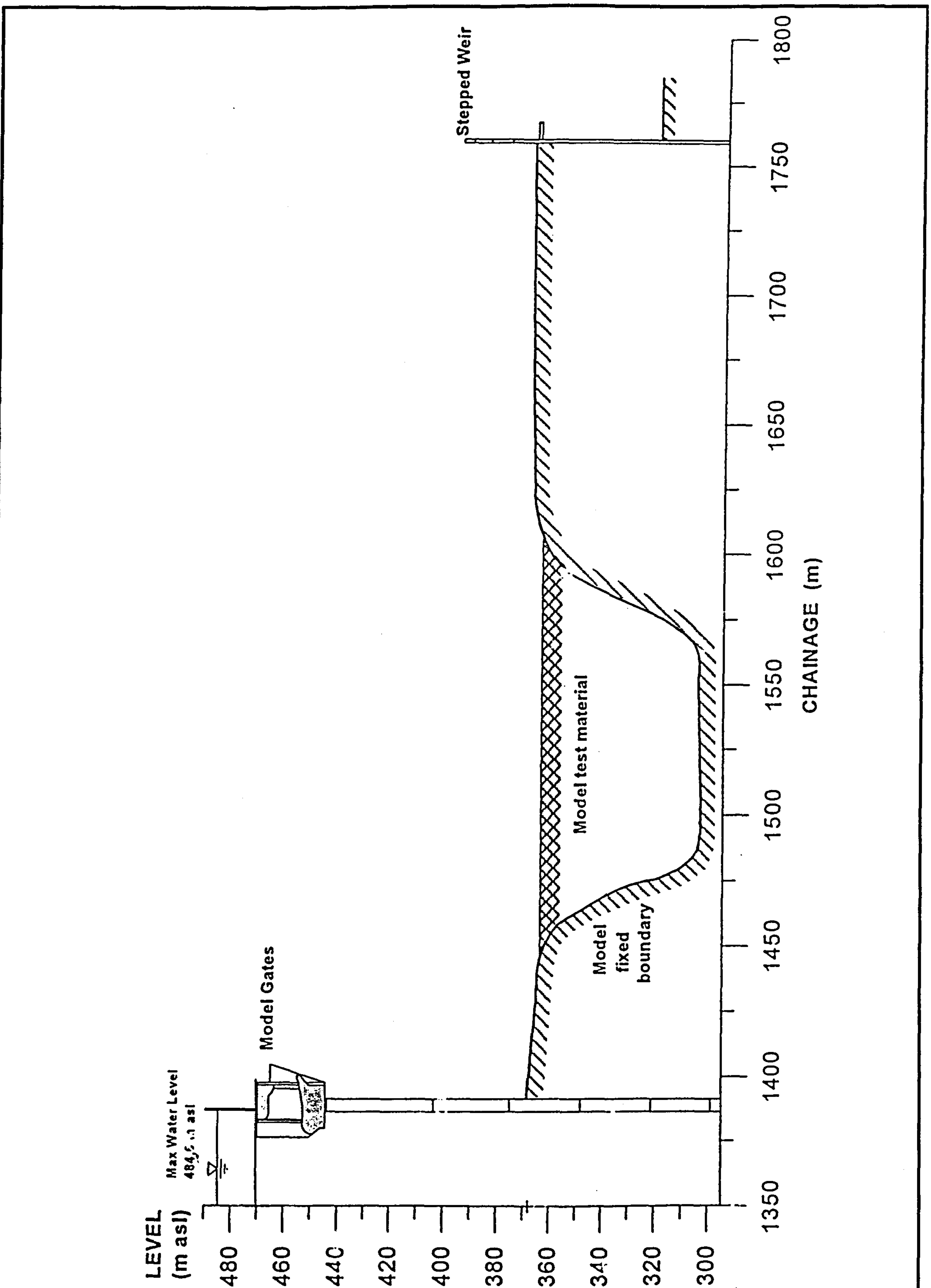
Goody, G. M. and A. Rooseboom (1998). SIGMA BETA Consulting Engineers Report on Maguga Dam Model Study, Stellenbosch



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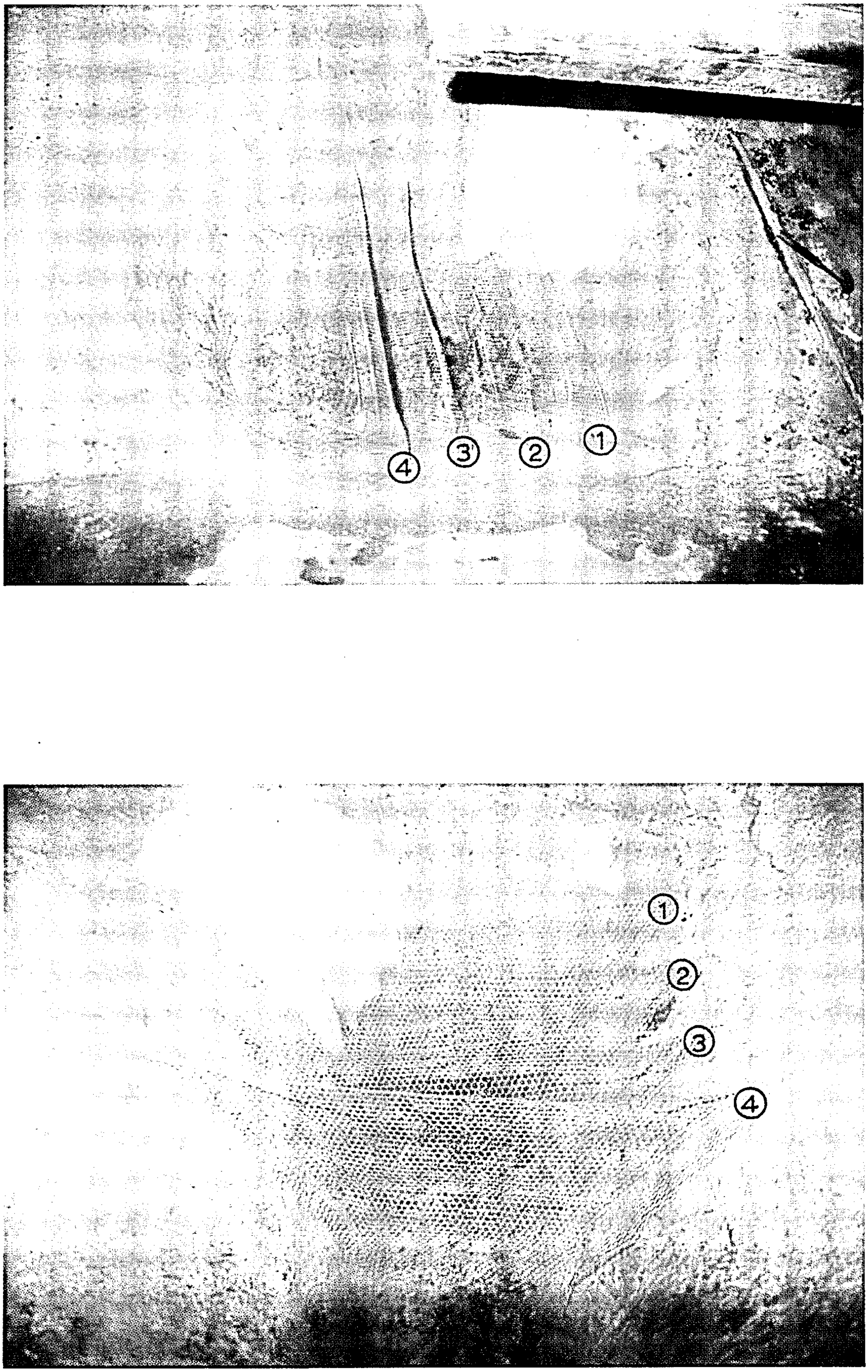
KARIBA MODEL

LONGITUDINAL SECTION ALONG DAM AXIS

FIGURE 2.1



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KARIBA DAM MODEL

MESH PANELS IN SCOUR HOLE: TEST 92 AND TEST 93

FIGURE 2.2

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SCOUR PROFILES : TEST 191

KARIBA MODEL

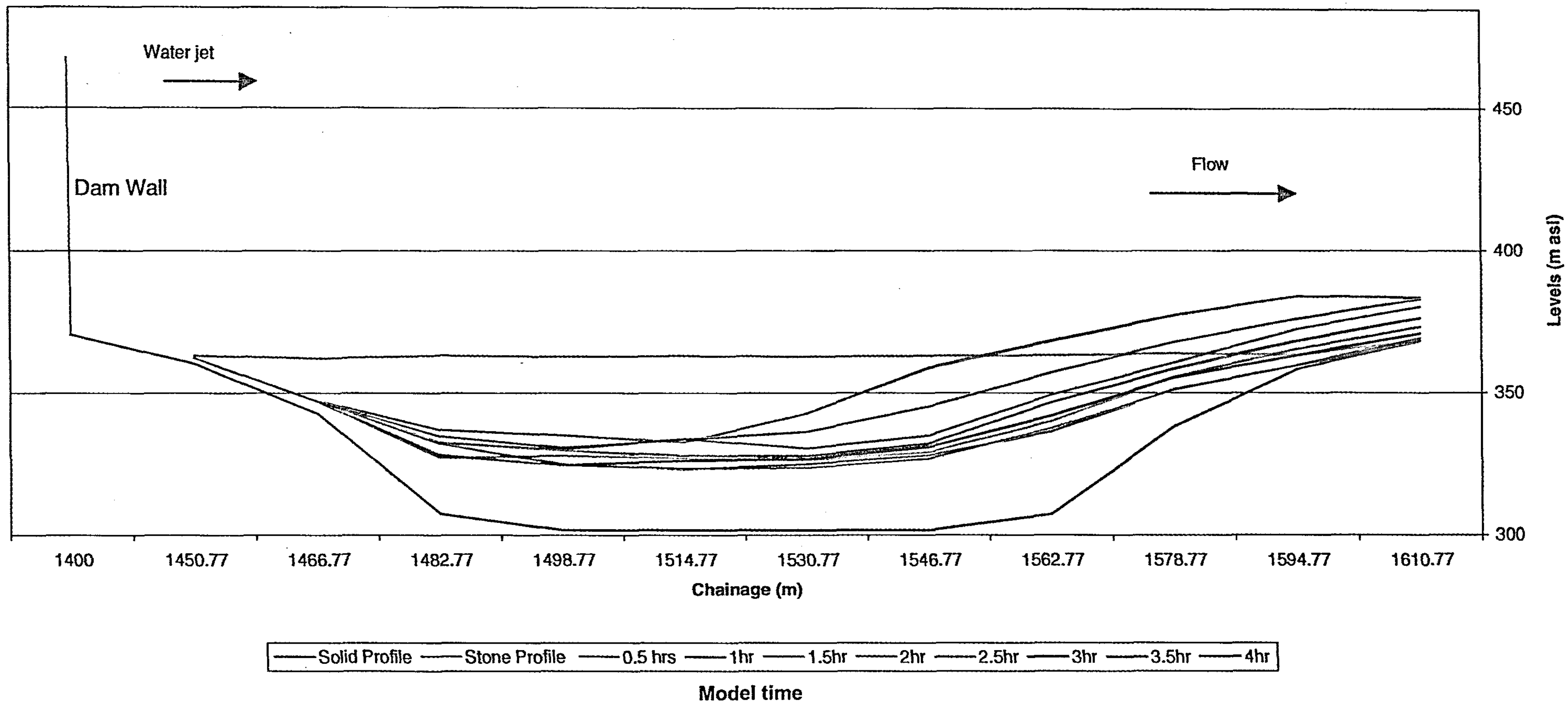

FIGURE 3.1

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SCOUR PROFILES : TEST 192

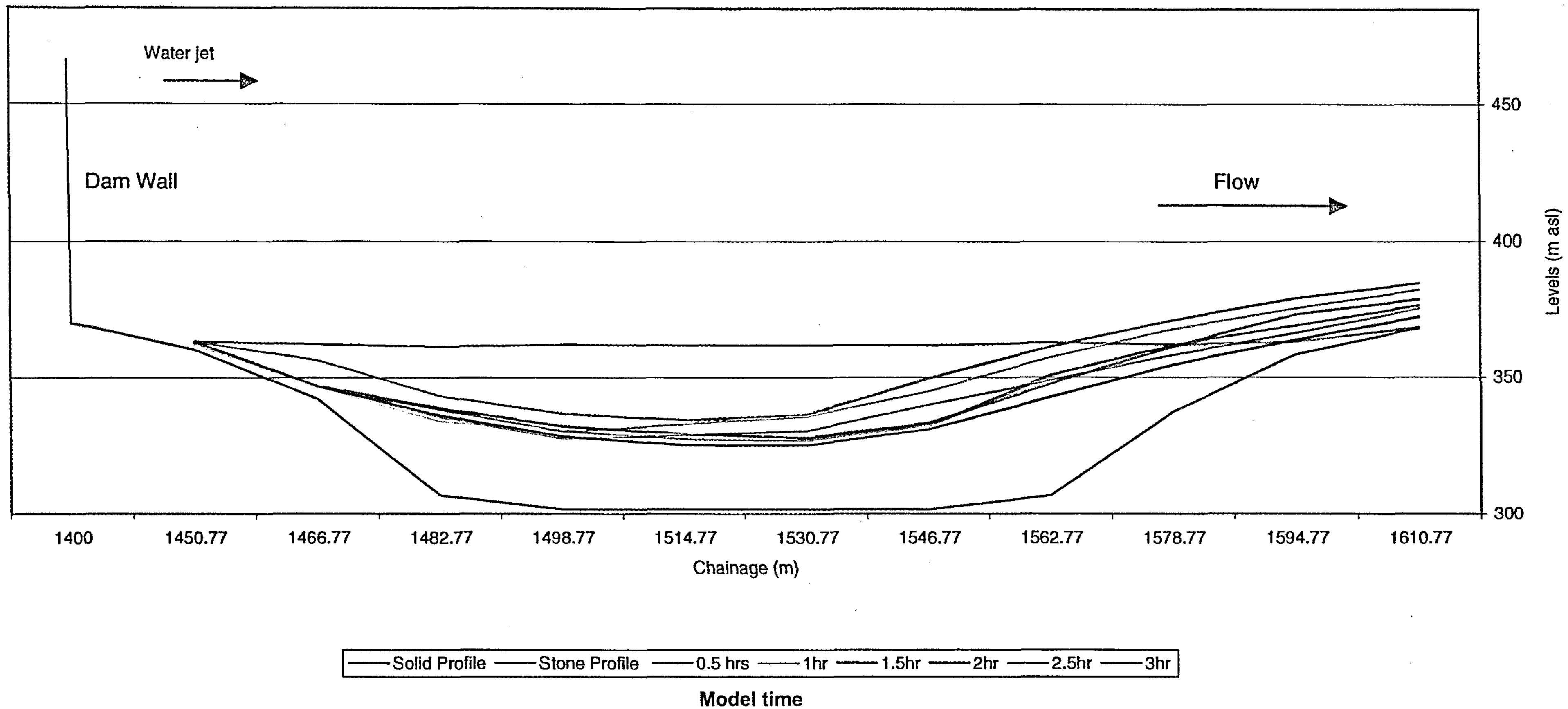
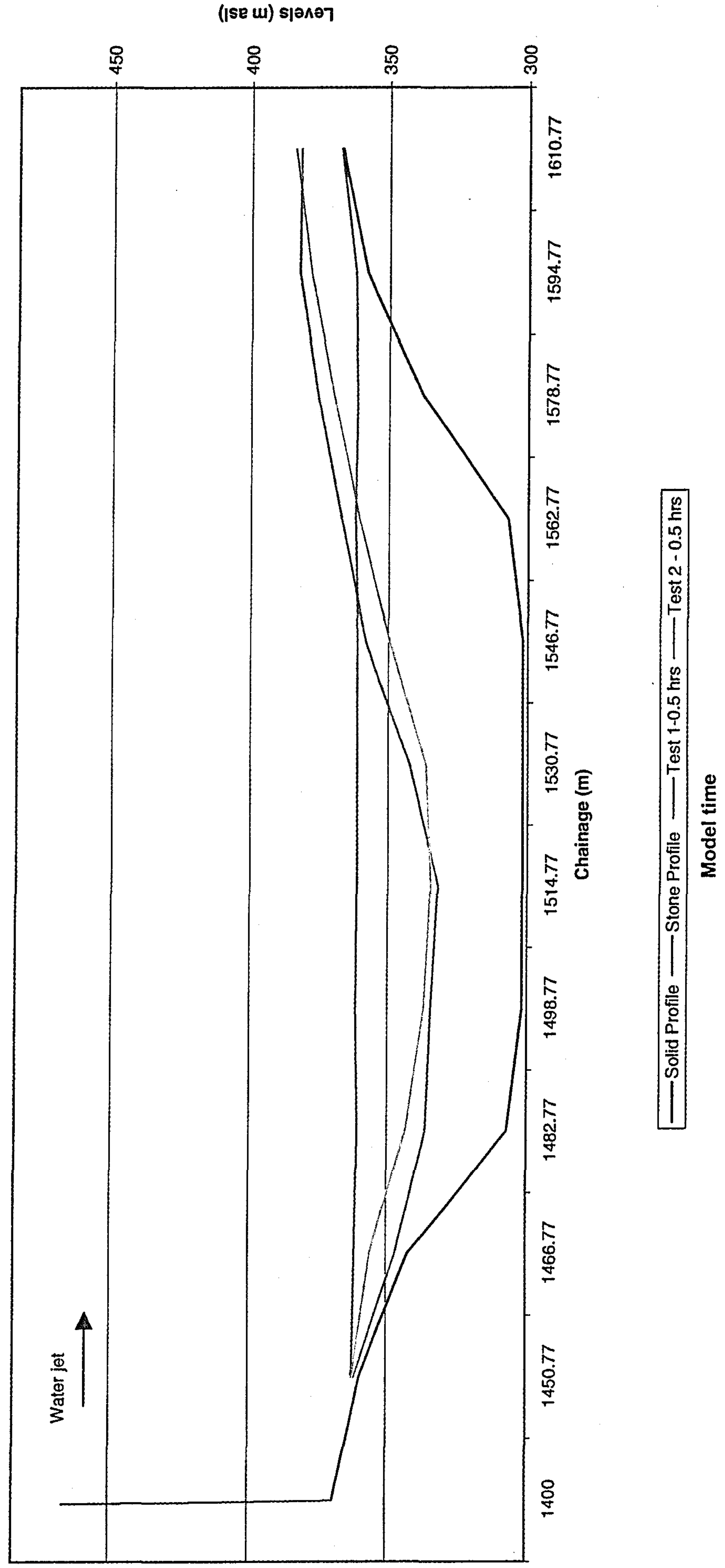
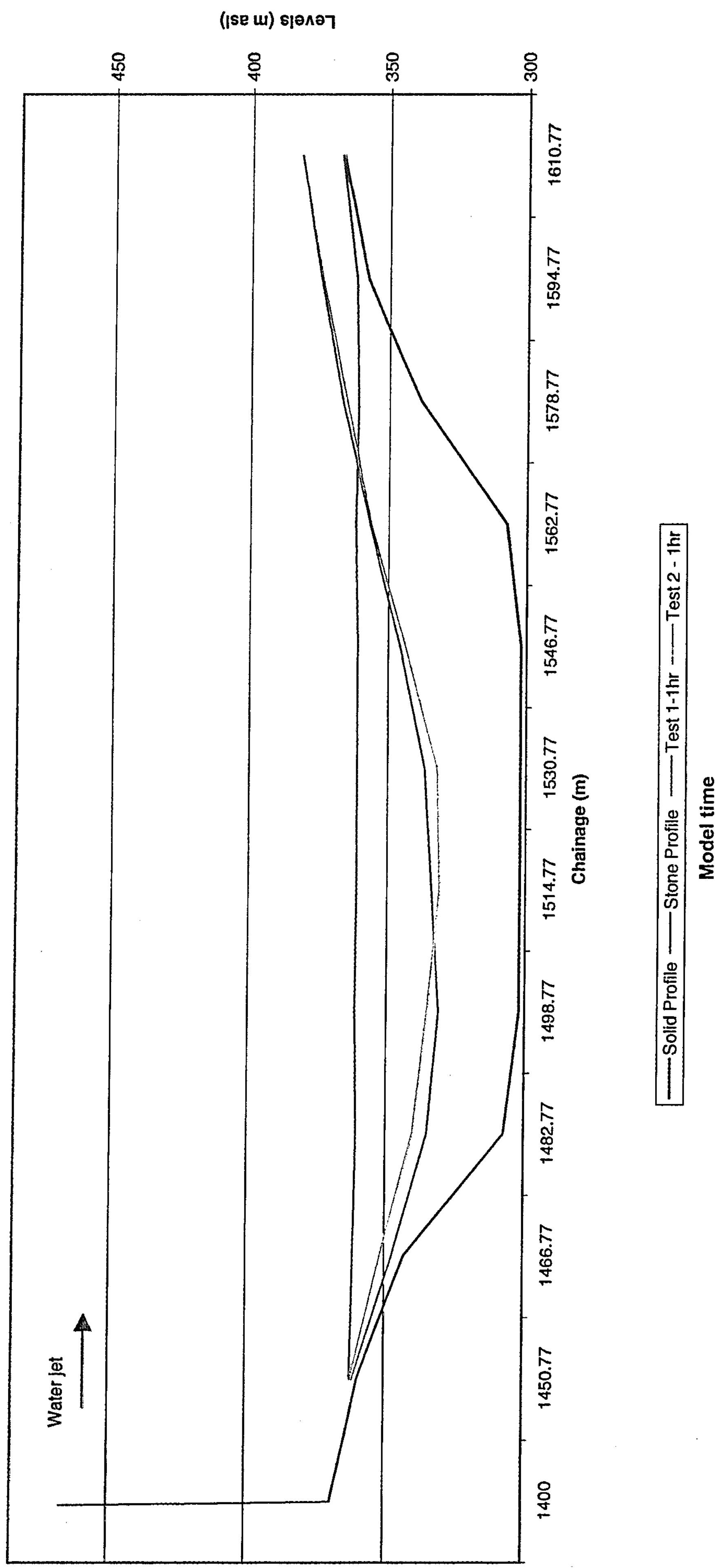


FIGURE 3.2



CSIR Your Technology Partner	KARIBA MODEL COMPARISON OF TEST 191 AND TEST 192- 0.5 HOUR PROFILES	FIGURE 3.3



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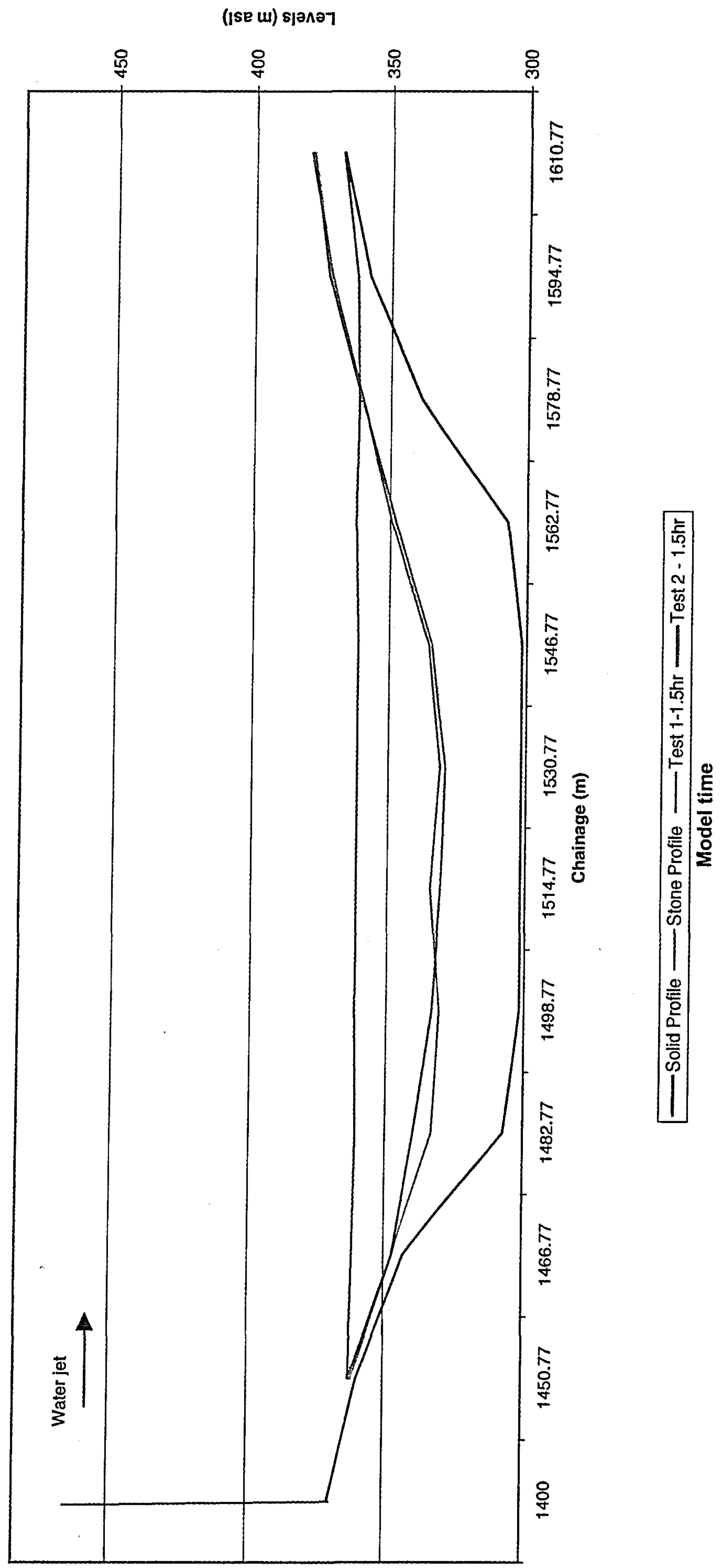
KARIBA MODEL

COMPARISON OF TEST 191 AND TEST 192-
1 HOUR PROFILES

FIGURE
3.4



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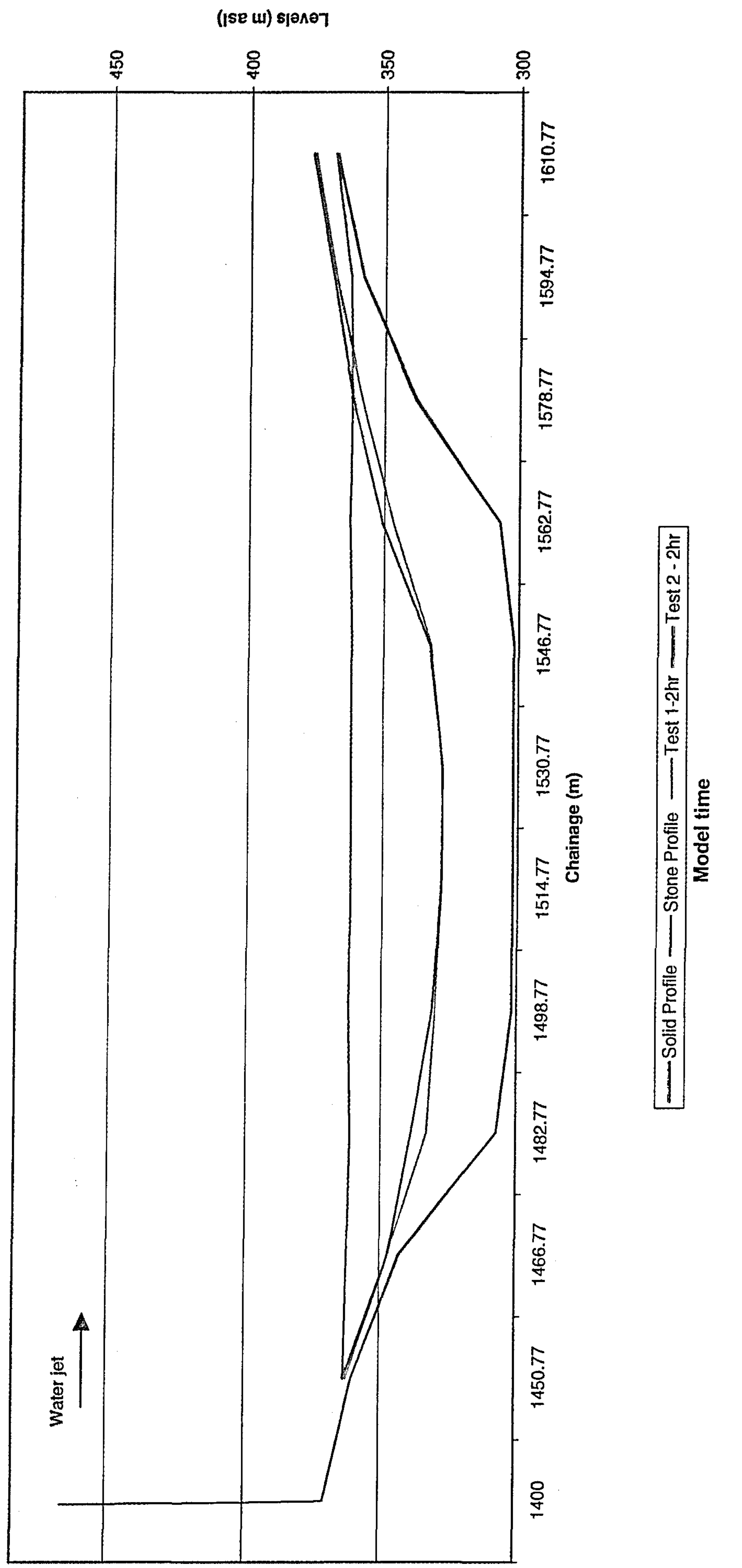
KARIBA MODEL

COMPARISON OF TEST 191 AND TEST 192-
1.5 HOUR PROFILES

FIGURE

3.5



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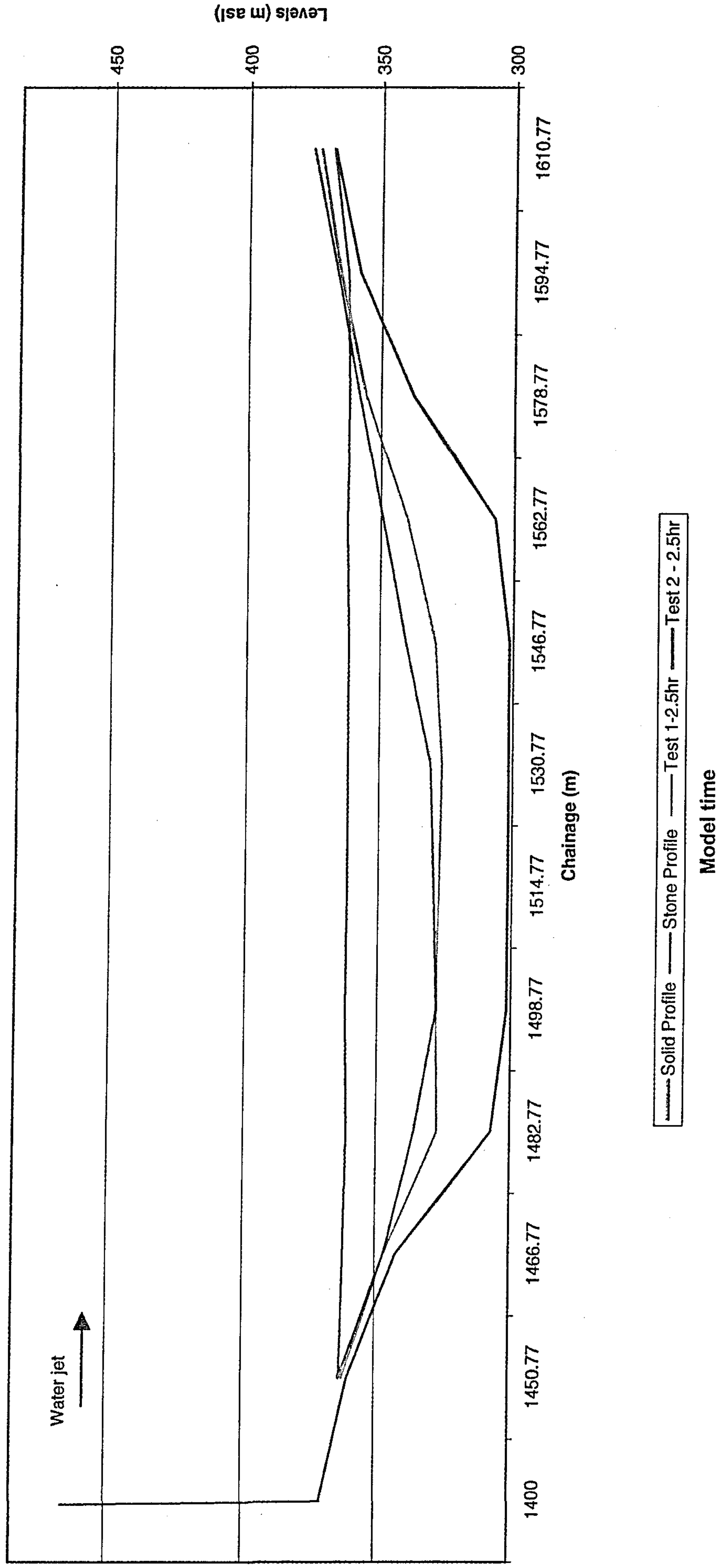
KARIBA MODEL

COMPARISON OF TEST 191 AND TEST 192-
2 HOUR PROFILES

FIGURE
3.6



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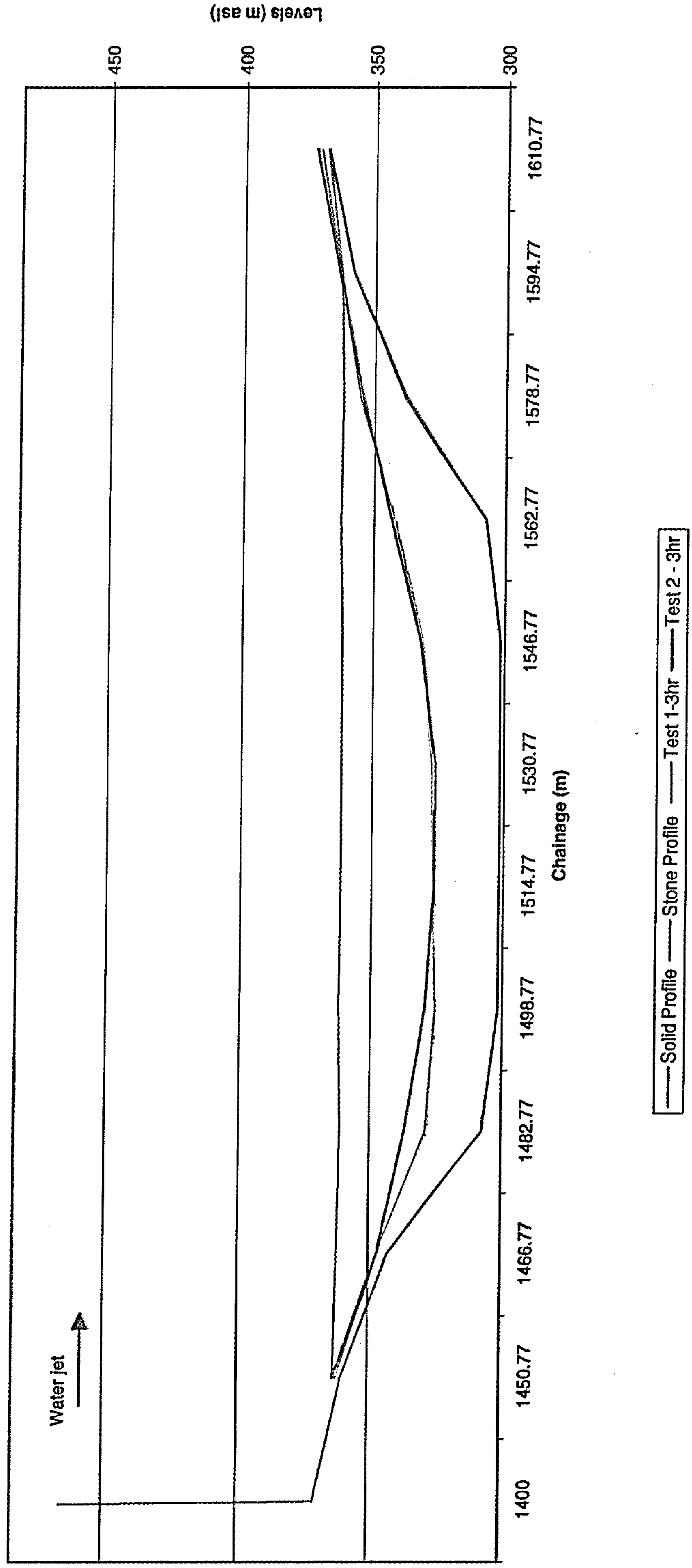
KARIBA MODEL

COMPARISON OF TEST 191 AND TEST 192-
2.5 HOUR PROFILES

FIGURE
3.7



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KARIBA MODEL

COMPARISON OF TEST 191 AND TEST 192-
3 HOUR PROFILES

FIGURE

3.8

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KARIBA MODEL

SCOUR PROFILES : TEST 131

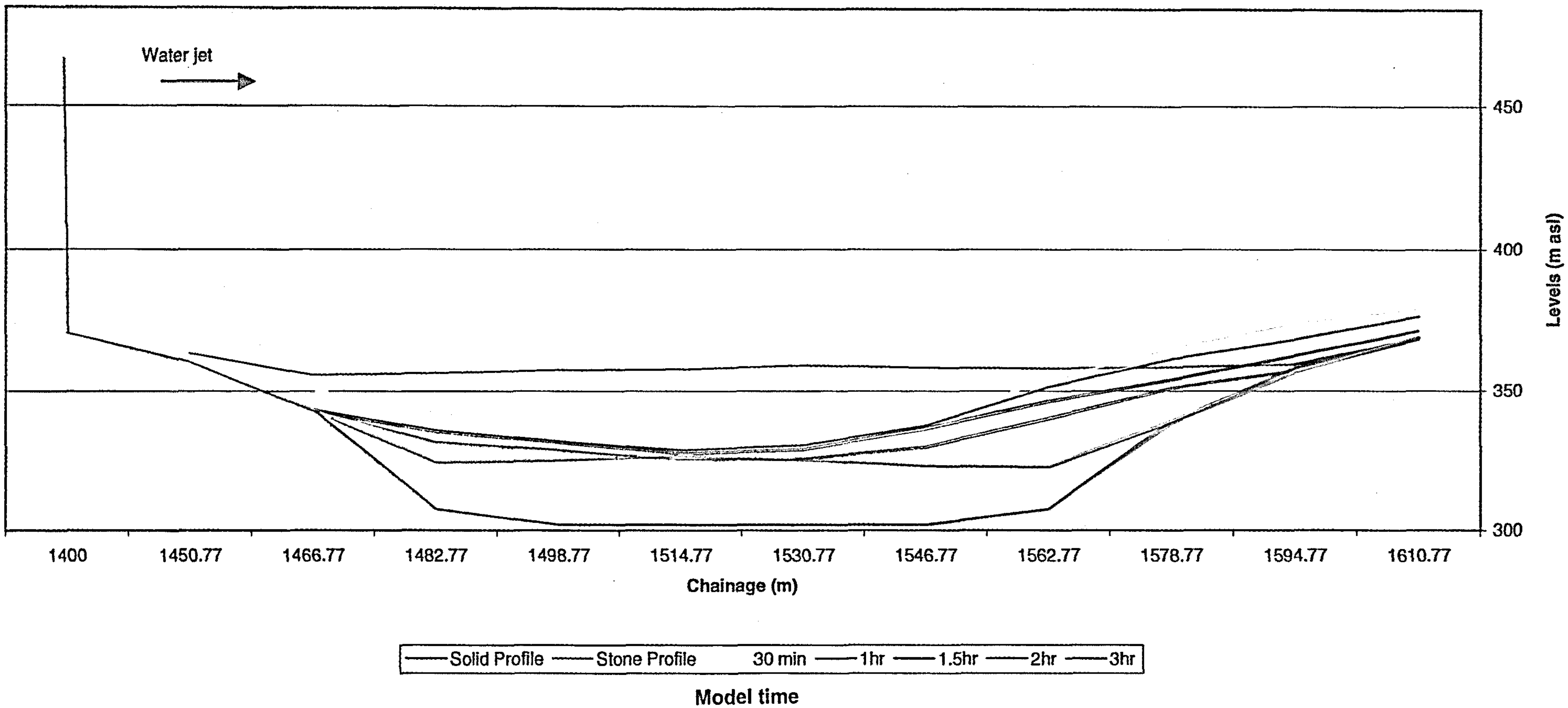
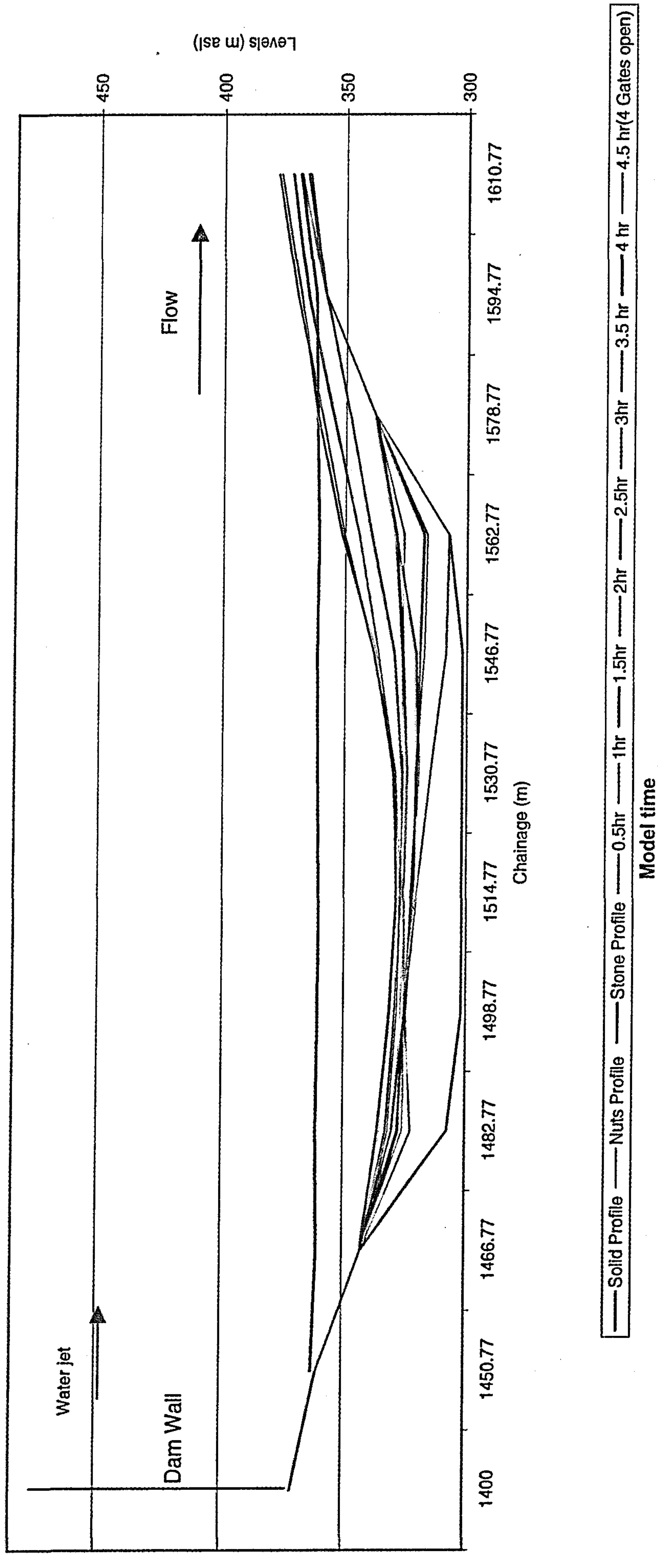


FIGURE 3.9



CSIR Your Technology Partner	KARIBA MODEL SCOUR PROFILES : TEST 13N1	FIGURE 3.10

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KARIBA MODEL

SCOUR PROFILES : TEST 91

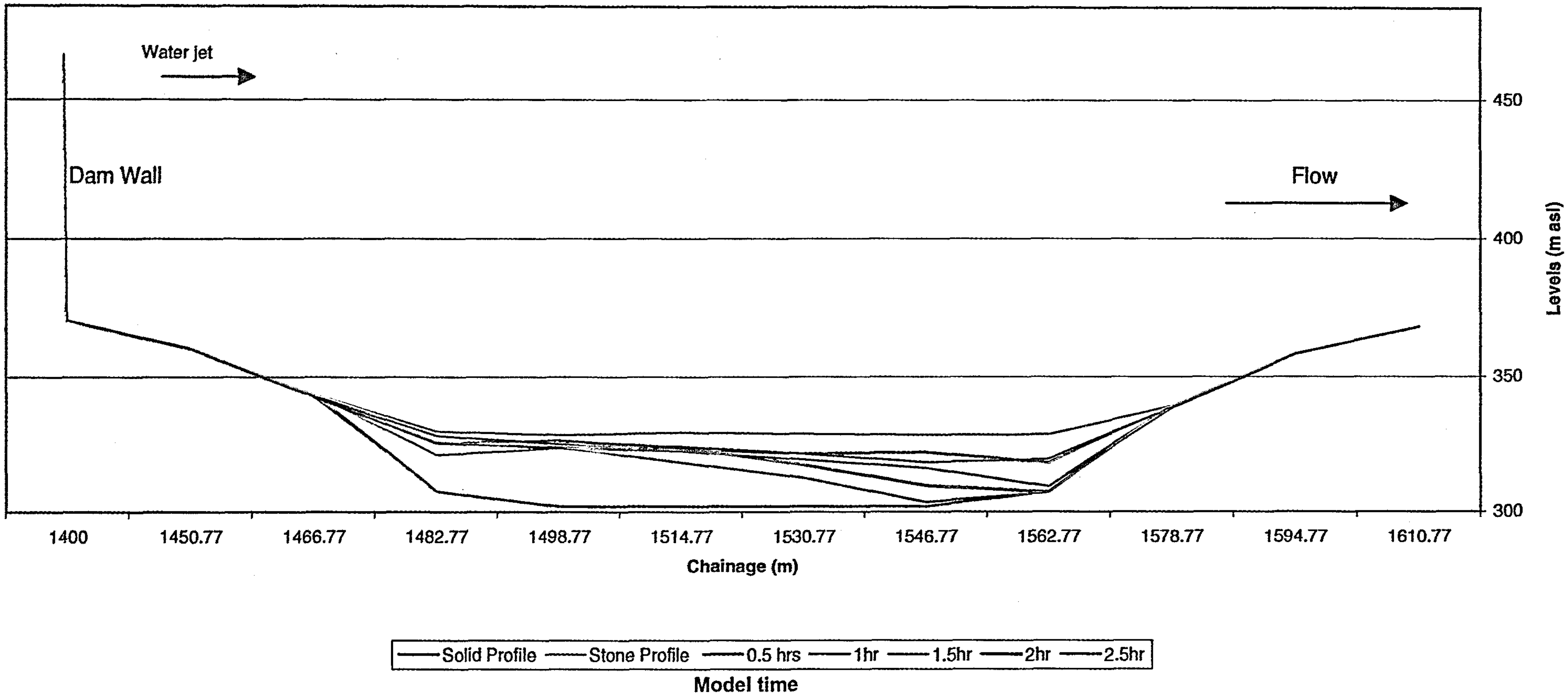

FIGURE 3.11

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KARIBA MODEL

SCOUR PROFILES : TEST 92

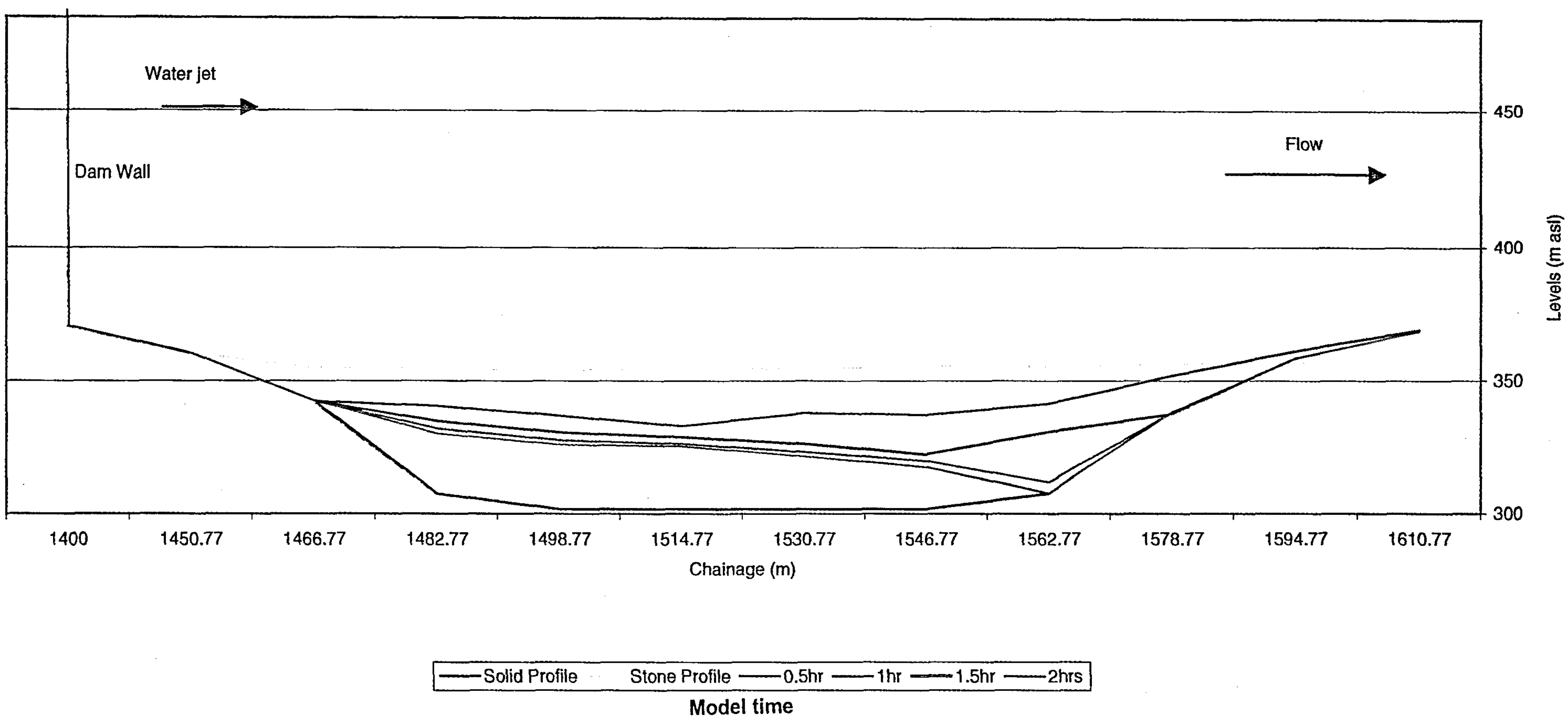
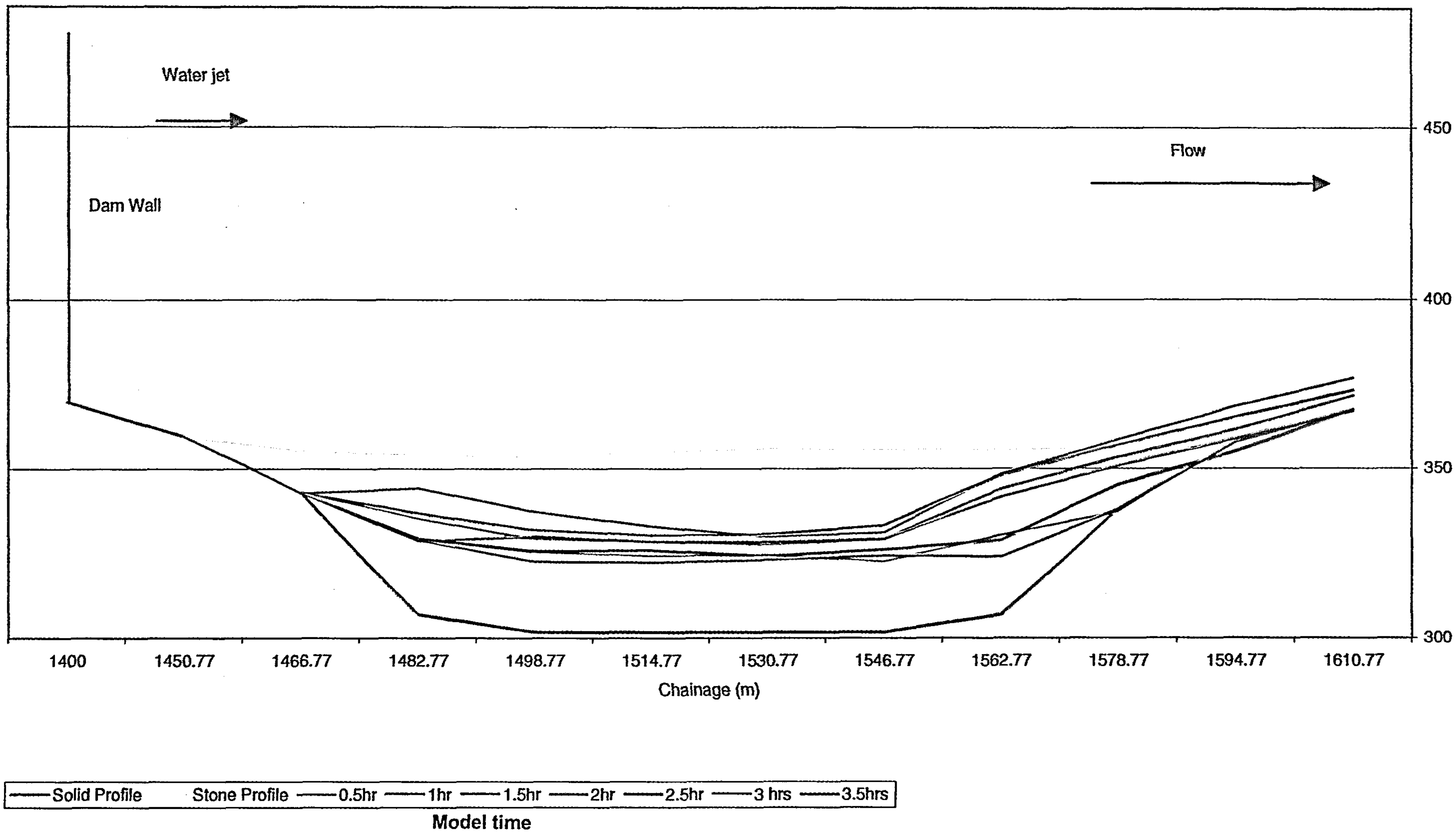


FIGURE 3.12



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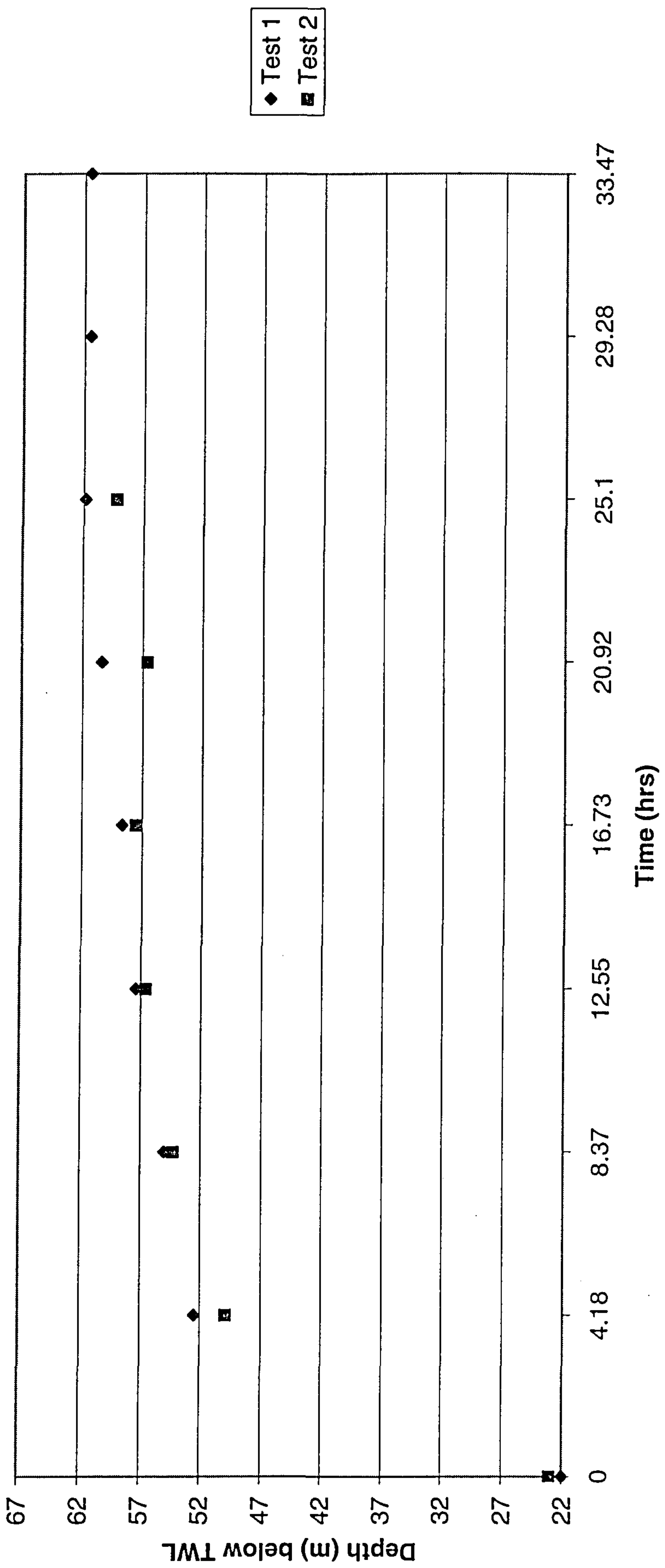
KARIBA MODEL

SCOUR PROFILES : TEST 93

FIGURE 3.13



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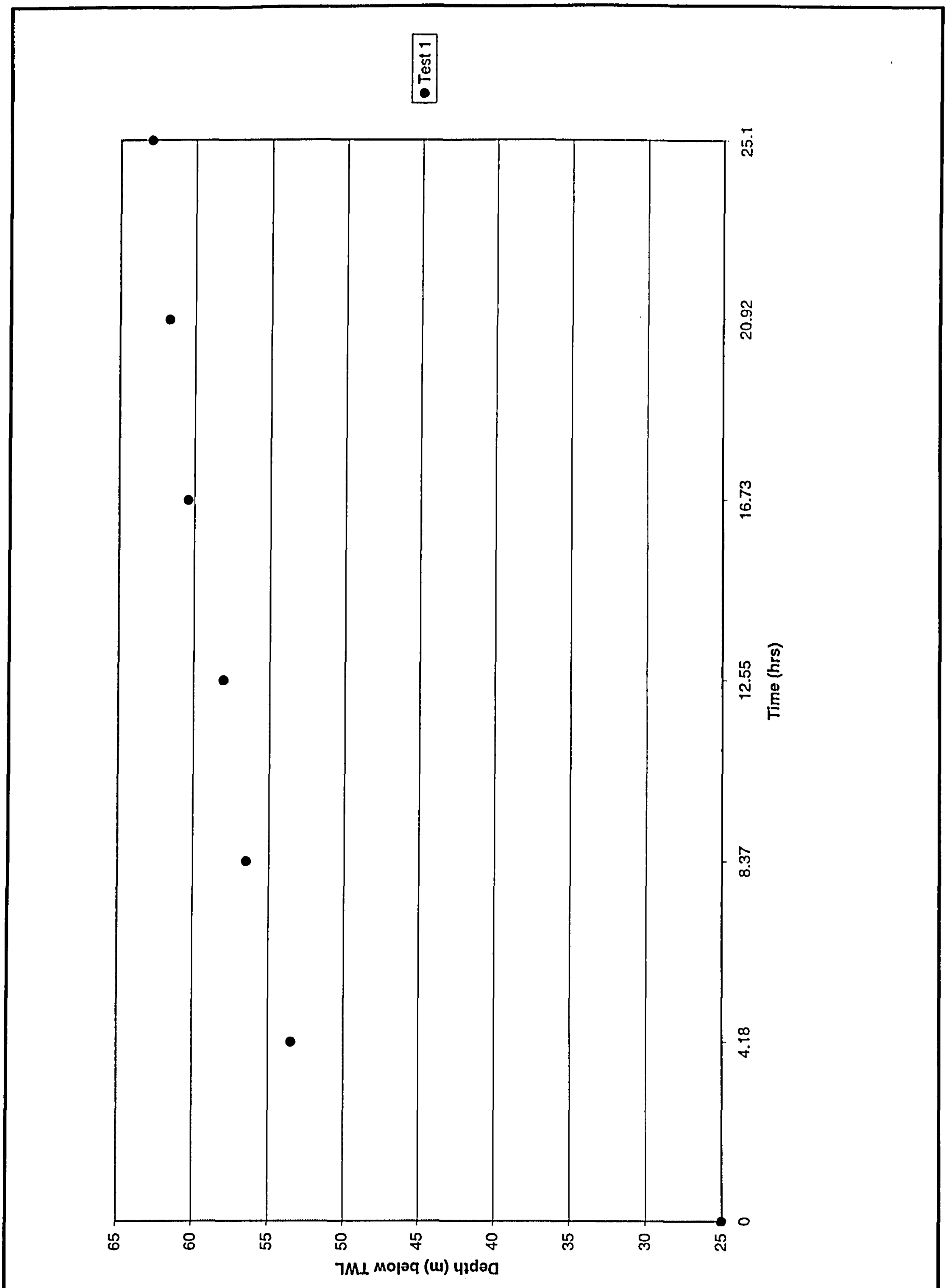
KARIBA MODEL

SCOUR DEPTH OVER TIME PLOT : TEST 191 AND TEST 192

FIGURE 4.1



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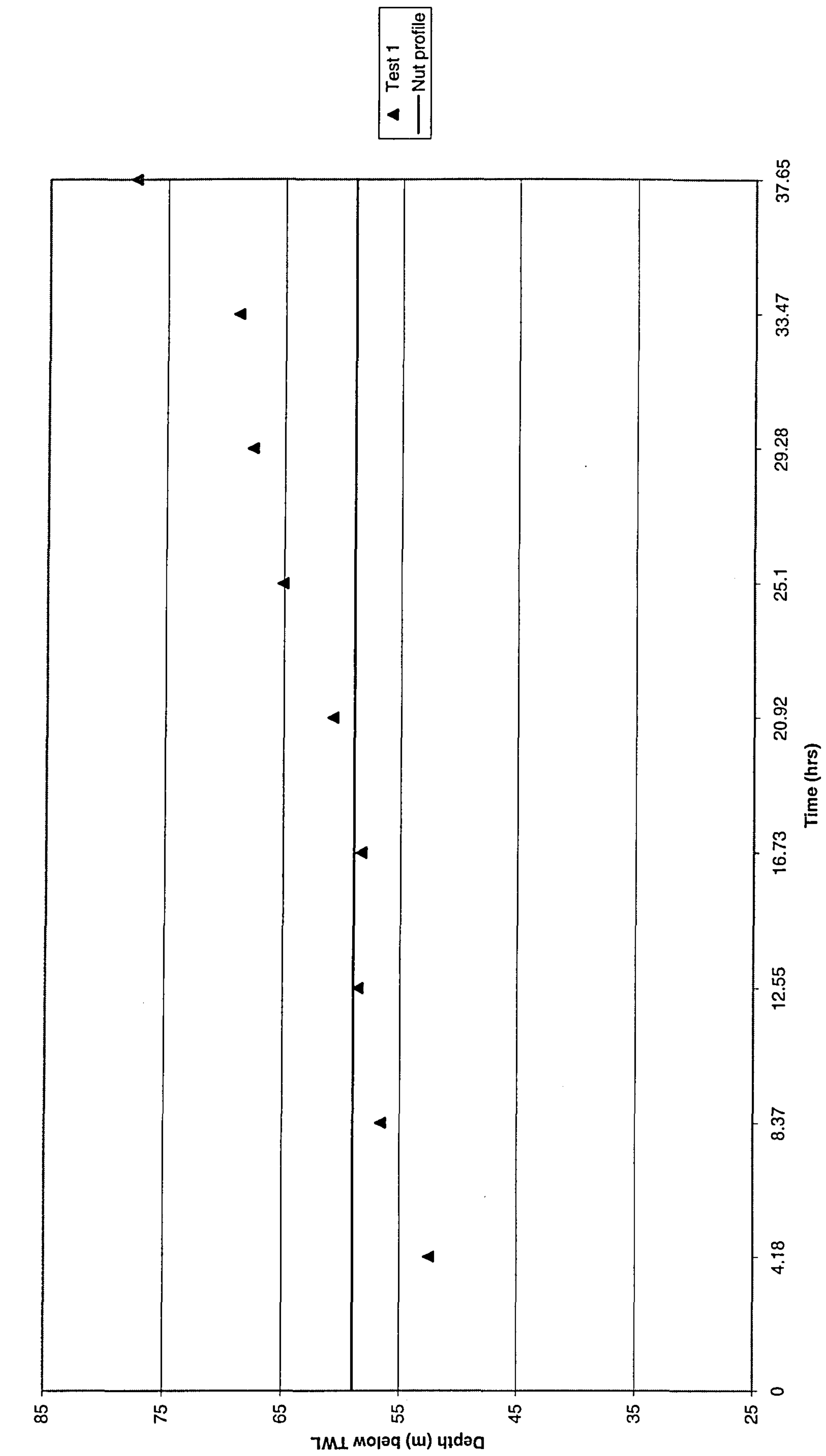
KARIBA MODEL

SCOUR DEPTH OVER TIME PLOT : TEST 131

FIGURE 4.2



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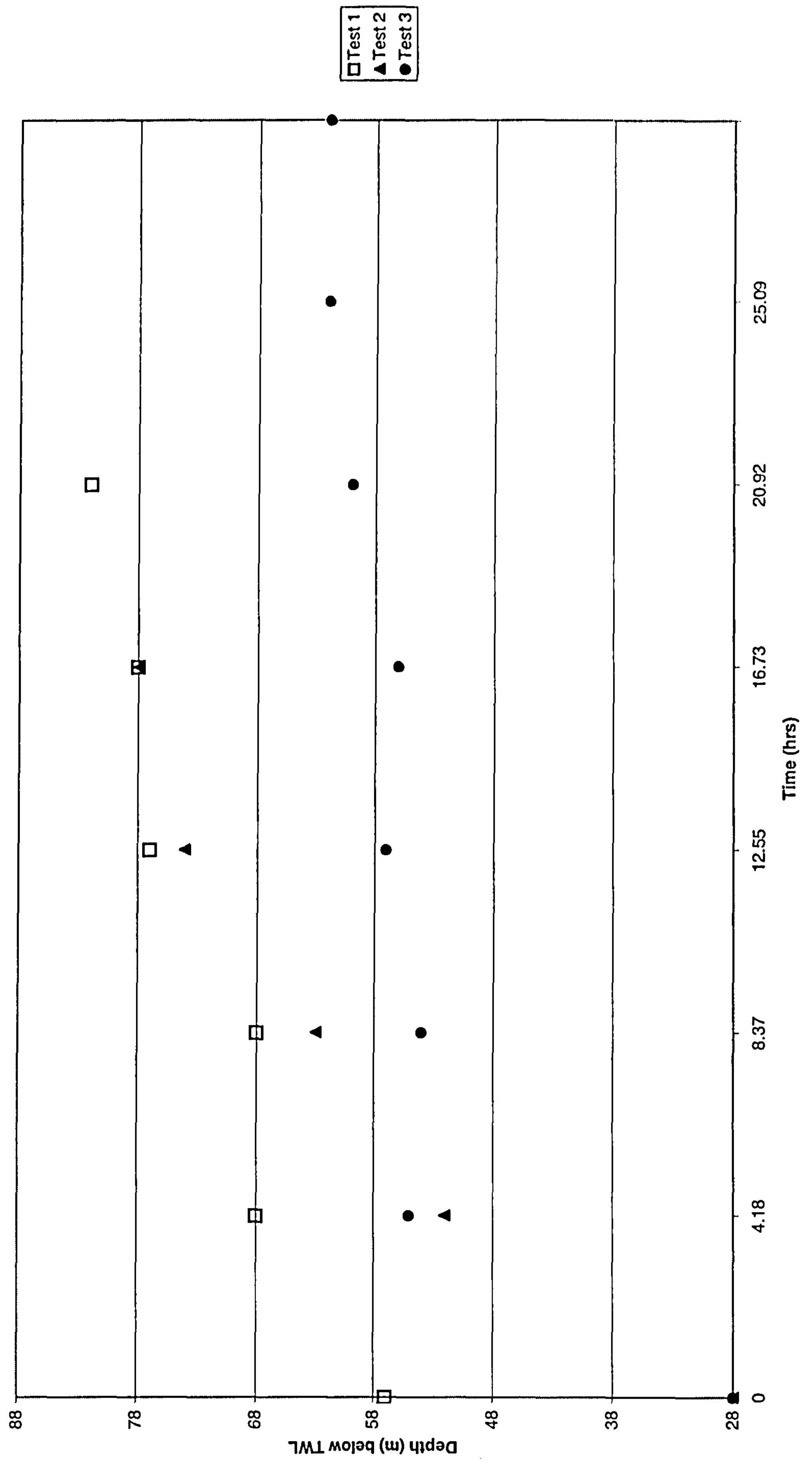
KARIBA MODEL

SCOUR DEPTH OVER TIME PLOT : TEST 13N1

FIGURE 4.3



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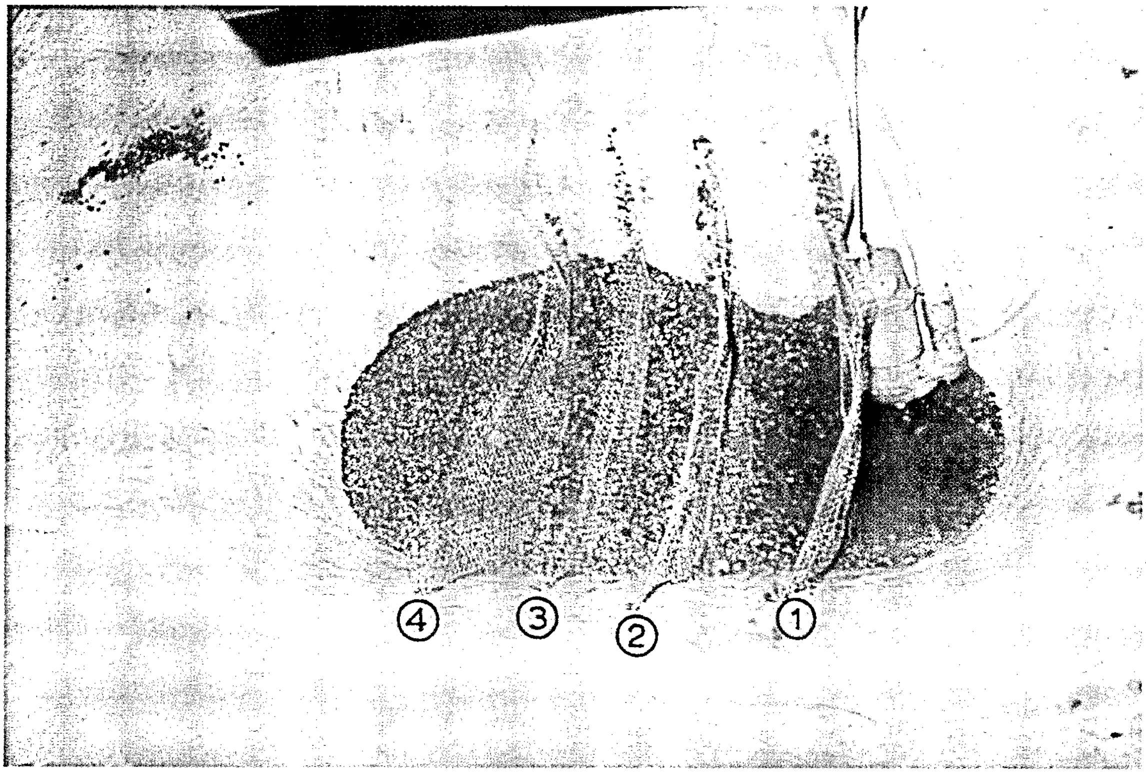
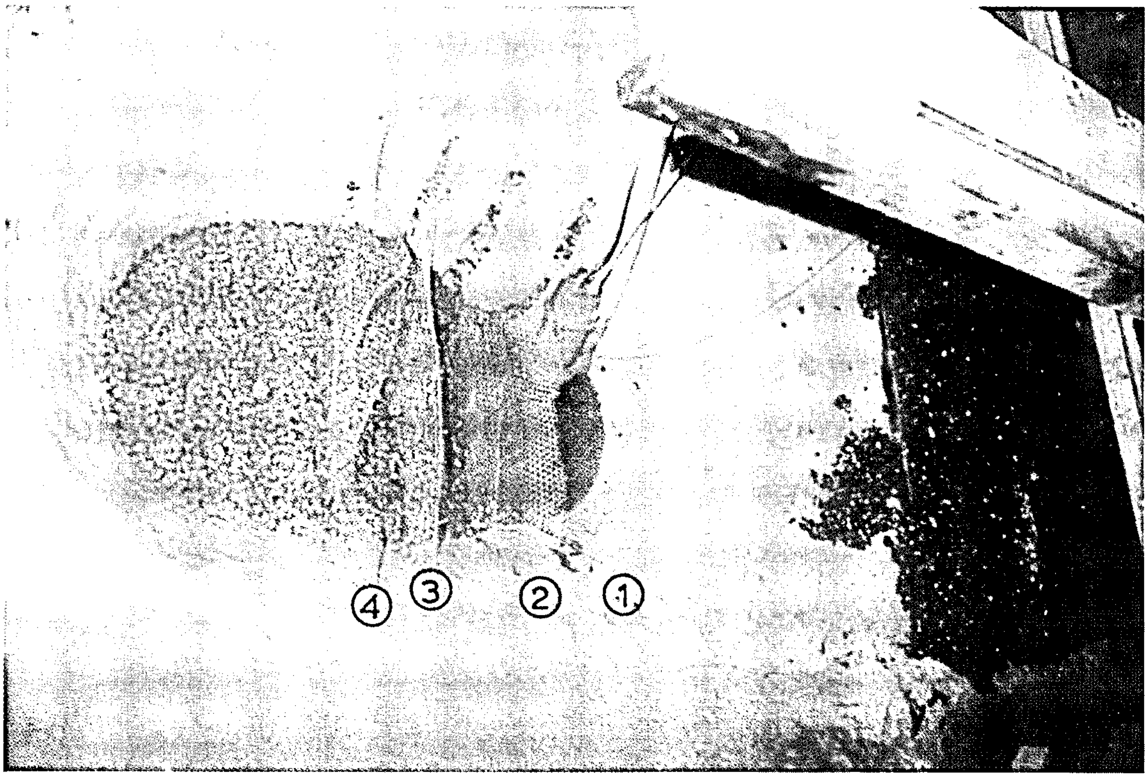
KARIBA MODEL

SCOUR DEPTH OVER TIME PLOT : TEST 91, TEST 92 AND TEST 93

FIGURE 4.4





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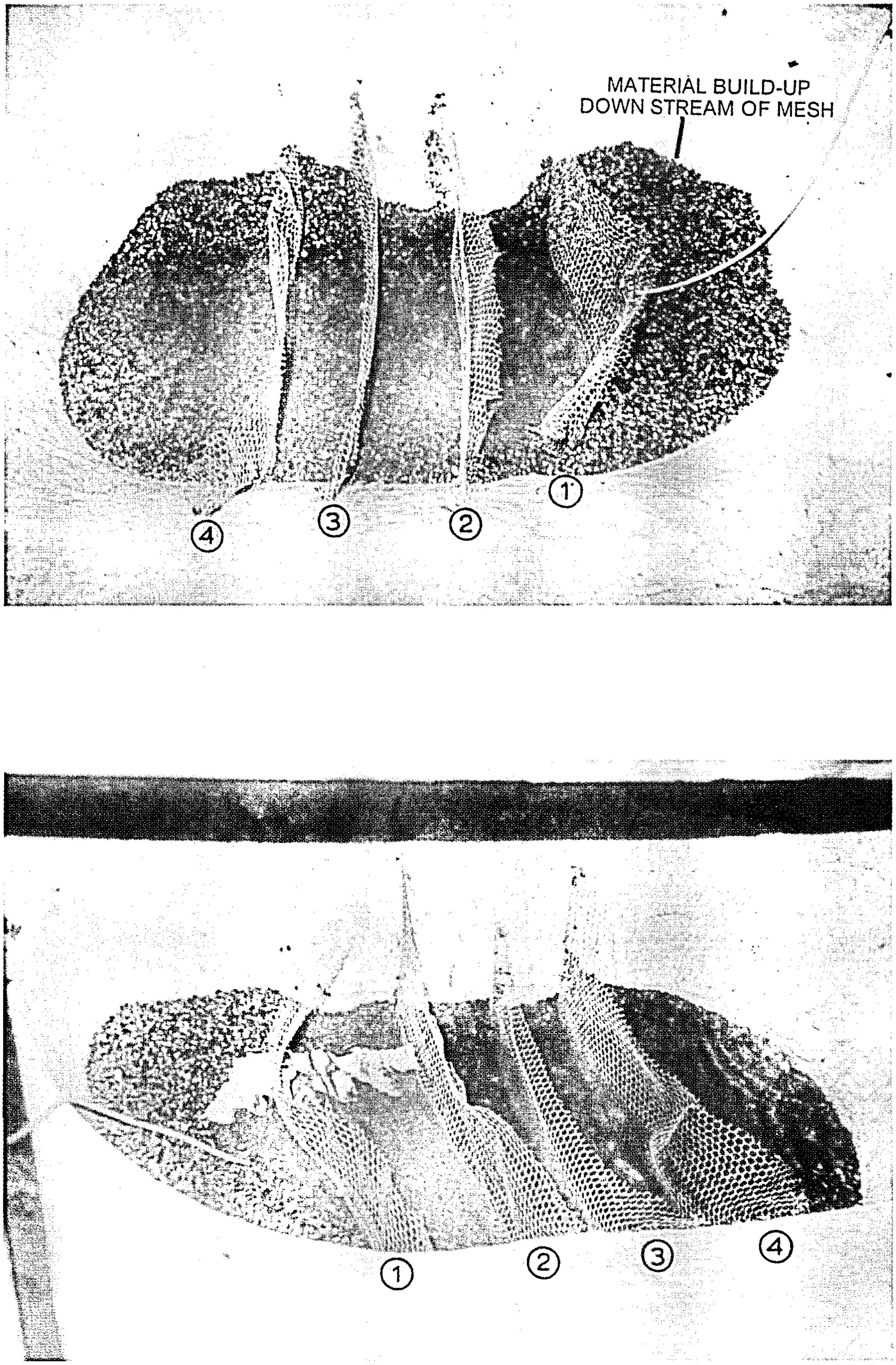
KARIBA MODEL

TEST 92 MESH PANELS DO NOT IMPROVE THE MATERIAL'S ANGLE OF REPOSE

FIGURE

4.5



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TEST 93 MESH PANELS IMPROVE THE MATERIAL'S ANGLE OF REPOSE

FIGURE 4.6