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A Numerical Model for Predicting Sediment Exclusion at Intakes

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Summary

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Sediment control structures are often incorporated in the design of intakes so that problems of sedimentation are minimised in the canal systems which they supply. There is a need to predict the degree of sediment exclusion provided by these structures.

Numerical modelling is an appropriate technique for the prediction of the performance of sediment control devices. The ability to overcome the scaling difficulties associated with laboratory models for this application, and cost considerations, make numerical modelling an attractive option for either supplementing or replacing physical models.

The development of a numerical model for predicting flow patterns and sediment movement at intakes is presented. The model is based on a widely used software package which models fluid flow in three dimensions. The report describes how the package has been developed for the prediction of sediment exclusion at river intakes.

Applications of the model at two irrigation intakes, where field data was collected in order to check model predictions, are described. Agreement was found between the predictions and field observations of the sizes and concentrations of sediment diverted to the canals. An assessment was made of the sensitivity of model predictions to the assumptions required to run the model, and to uncertainties in the input data. It was found that reliable predictions will be obtained using the information that would be available in investigations of improvements to existing intakes.

The model can also be applied in design studies for new intakes. However, further work is recommended to develop and verify a procedure for predicting changes in river bed elevations resulting from the construction of the intake.

Symbols

a	constant exponent of bed shear stress
dr_j	sediment deposition rate for fraction j ($kg/m^2/s$)
D_{50}	grain size at which 50% of bed material is finer (mm)
D_{90}	grain size at which 90% of bed material is finer (mm)
E	sediment entrainment rate from river bed ($kg/m^2/s$)
g	acceleration due to gravity, $g = 9.81 m/s^2$ (m/s^2)
h	flow depth (m)
j	suffix denoting size fraction
k	constant relating shear stress to sediment entrainment rate
p_{bj}	proportion of the bed material consisting size fraction j
PR	performance ratio defined as the reduction in sediment concentration from river to canal, as a proportion of concentration in the river
PR_{sl}	performance ratio defined for the sluice channel (Kapunga intake)
S	longitudinal slope of river
tdr	total sediment deposition rate from bed cell ($kg/m^2/s$)
u_*	shear velocity, $u_* = \sqrt{h\,S\,g}$ 1 (m/s)
u_{*cr}	shear velocity at threshold for sediment motion (m/s)
V_{sj}	settling velocity of sediment size fraction j (m/s)
β	ratio of sediment diffusion coefficient to kinematic turbulent viscosity
τ	shear stress at river bed (N/m^2)

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1 Introduction

It is common practice to provide sediment control structures at an intake, so that problems of sedimentation are minimised in the canal system which the intake supplies. This report presents a method for predicting quantitatively the effect of such structures.

1.1 Sediment exclusion at intakes

Sediment exclusion at intakes is usually achieved by exploiting the large sediment concentration differences which can occur in rivers: when the bed material is relatively coarse, sediment concentrations at the bed are many times larger than those at the water surface. Thus, when flow from the upper layers of the river is abstracted, low sediment concentrations enter an intake. Sediment control structures which are applied at intakes can be categorised as:

(i) excluders, which exclude the near bed flow; examples are the tunnel type sediment excluder and the curved channel sediment excluder,

(ii) skimming weirs, which are designed to withdraw only the surface flow,

(iii) siting an intake on the concave bank of a river bend, so that the helicoidal secondary currents, which a bend produces, sweep the near bed flow away from the intake, and

(iv) still pond regulation: a sluicing pocket, with relatively deep and hence slow moving flow, traps sediment, which is then flushed away when the sluice gates are opened occasionally.

These structures can be expensive to construct and may not solve a canal sedimentation problem. For example at the Narora and Kosi barrages in Northern India (Atkinson, 1989, and Sahay et al, 1980). Alternative sediment control methods, such as sediment extractors or settling basins, may be required to control sediments comprehensively.

1.2 The need for quantitative prediction

A reliable and cost effective design is best achieved when the performance of all the sediment control options can be predicted quantitatively. The performance of each option can then be compared with the required reduction in sediment load between the river and the canal. A quantitative prediction of excluder performance also provides a basis for deriving improvements in design. It is a need for quantitative prediction which has been the major focus of research into sediment control structures at HR Wallingford.

Initially research work focused on sediment extractors (Sanmuganathan, 1976, Sanmuganathan and Lawrence, 1980, Singh, 1983, Atkinson, 1987, and Russell, 1991) and a design manual with accompanying software has now been produced, (HR Wallingford, 1993a). Subsequently settling basins and similar sluiced structures were studied (Atkinson, 1986, Fish, 1988, and Atkinson, 1992) and a software package for designing these structures has been produced.

A parameter which describes the effectiveness of an intake in excluding sediments is the performance ratio, PR. Performance ratio is defined as:

$$PR = 1 - \frac{\text{sediment concentration entering canal}}{\text{concentration being transported by the river}}$$

A performance ratio of 1.0 would indicate complete sediment exclusion, while a negative ratio would indicate that the intake is withdrawing a higher sediment concentration than the mean concentration in the river. A performance ratio of zero would indicate neutral performance, with the intake neither reducing nor enhancing sediment concentrations.

For small particle sizes sediment is well mixed in the river flow, therefore performance ratio tends towards zero as the sediment size becomes finer. Performance ratio will rise as sediment size increases at well designed intakes, where flow from the upper layers of the river is abstracted.

A sediment excluder has a large effect on the grain size distribution entering an intake, and this can be more important than its effect in reducing the overall sediment concentrations. For example, an intake with a moderate overall performance can exclude all the coarser material, so that only fine material enters the canal. A canal is often able to transport fine material, and thus the intake prevents what would otherwise be severe sedimentation in the canal. Such conditions can only be assessed with a size-by-size prediction of performance ratio. Hence, performance ratio is defined both with an overall value, which accounts for all the sediment in transport, and with an individual value for each sediment size fraction.

Material which is finer than 63 microns is classified as silts and clay. Typically, silts and clay are very well mixed in the river flow and can be transported by main canals. Such material is not included in the specification of performance ratio.

The objective of the work described in this report was to develop a reliable method for the prediction of performance ratio.

1.3 Physical modelling

Traditionally, physical models have been used to predict the performance of sediment control devices, and physical modelling is widely adopted as part of the design process for sediment exclusion measures at intakes.

Physical modelling enables a visualisation of the flow processes at an intake, and thus is often effective in producing improvements to an intake design. It also is useful in predicting the changes in river bed topography which an intake may cause. However, physical models can not normally predict sediment exclusion performance quantitatively. The problem is principally one of scaling. If sediment is scaled in proportion to the main model scale, then the material required becomes so fine that it is cohesive, and exhibits very different properties from the prototype material. Therefore, only the coarser sediments can be scaled. An alternative approach is the use of lightweight sediments in the model. However, Yalin (1971) has shown that, even with lightweight sediment, it is impossible to satisfy all the physical laws of scaling when water is used as the fluid in the model.

These difficulties usually cause the model sediment to represent relatively coarse material in the prototype. The sediment therefore moves in the model as bed load, and any correctly designed device for sediment exclusion shows a high performance ratio in the model.

Results reported in Atkinson (1989) illustrate this problem. A tunnel type sediment excluder was incorporated in the remodelled Narora Barrage, Uttar Pradesh, India in 1967. Physical model tests conducted prior to construction indicated a performance ratio of 0.92, which represents excellent performance. Field measurements at the barrage were undertaken in 1971 and showed that the excluder was having a negligible effect on sediment concentrations. Observed performance ratios were in the range -0.1 to 0.08. The material used in the model was sand of median size 0.23mm which, when scaled, represented a grain size many times larger than the median size in the river bed material on the prototype.

An associated problem with physical models is representing the grain size graduation found in rivers. The (non-cohesive) material transported by a sediment laden river can range from fine sand to gravels or even boulders, covering two or more orders of magnitude in grain diameter. It is not practical to cover such a range in a model, and indeed a model using more than a single sediment fraction would be exceptional. Therefore the important prediction of size-by-size performance ratios is extremely difficult when physical modelling is employed.

For rivers where the bed has a wide grain size distribution, the median size of the sediment in transport is many times smaller than the median size of the bed material. It is the material in transport which determines the intake performance. However, the single representative sediment size selected for a model is usually chosen to represent prototype bed material, so that the bed morphology and roughness are correctly modelled. This must imply that the material in transport is not represented accurately, and so intake performance is overestimated.

Significant inaccuracies are likely in any quantitative performance prediction using a physical model. Indeed, the purpose of physical model studies for this application is often stated as the qualitative reproduction of prototype conditions, for example Tosswell (1989).

Quantitative prediction of the performance of sediment exclusion structures is only one application of physical modelling: the shortcomings listed above do not significantly affect predictions for many other applications.

1.4 Numerical modelling

The sections above have presented the need for a new method to predict sediment exclusion quantitatively. Numerical modelling of the flow and sediment movement in the vicinity of an intake has the potential to make such predictions. Firstly, there are none of the practical difficulties of scaling and representation of more than one grain size in a numerical model. Also, numerical modelling can be less expensive than a physical model.

A numerical model of a river in the vicinity of an intake must include three dimensional effects: for example the helicoidal flow which develops in a river bend or at a curved channel sediment excluder. A model must also take account of: momentum, turbulence, bed geometry, bed friction, sediment settling velocities, and the geometry of any sediment exclusion structures or other intake features.

1.5 Previous work

The approach adopted has been to use computational fluid dynamics (CFD) software to develop the numerical model. Commercially available CFD codes have been used widely in recent years, particularly in fields such as chemical engineering and aeronautical engineering. Their use in hydraulics has been less widespread but examples include reservoir, lake and coastal flows (Neve and Gusbi, 1991; Nyberg, 1984a and Nyberg, 1984b).

The CFD code "PHOENICS" was chosen for the study. Atkinson (1989) describes the initial work on applying the code at intakes, and presents comparisons with two sets of data. In the first comparison, predictions agreed closely with observations of the abstraction of sediment into a branch from a laboratory flume. The flow pattern near the branch included significant three dimensional effects and was therefore an excellent test for the numerical model. The second comparison was with the observations of sediment exclusion at the Narora Barrage, which were discussed in Section 1.3 above. Predicted performance ratio was 0.07, which compared well with the observed value for that condition (0.04). This result was encouraging, however it was not considered a rigorous test of the method, because three dimensional effects did not influence intake performance: its low performance was due to the fine sediments being transported by the river.

1.6 This report

Since the publication of Atkinson (1989), the numerical method has been developed further and data for model verification have been collected at two intakes in the field. This report presents the work to develop the model since 1989.

2 Application of CFD code to model intakes

2.1 The CFD code

PHOENICS is a computational fluid dynamics package for simulating a wide range of fluid flows. It is based on the equations of motion for viscid compressible flow. The equations are solved over a grid of cells which can be one, two or three dimensional.

The phenomena which the code can simulate, and which are relevant to this application, include:

(i) momentum effects,

(ii) turbulence in the flow,

(iii) boundary roughness effects, such as at a river bed,

(iv) the convection and diffusion of concentrations within a fluid, and

(v) free surfaces.

Simulations can be set up in on a regular grid of rectangular cells, on a polar grid with an axis of symmetry or on a curvilinear grid of near-rectangular cells. The latter option is termed "body fitted coordinates" and has been used for this application.

The code was launched in 1981 and has since been widely used in many engineering fields, including the modelling of environmental flows. For a more detailed description of PHOENICS reference should be made to the literature produced by CHAM Ltd, who developed the program. (An address is given in the acknowledgements.)

The code has a facility which enables the user to write further coding, so that additional features can be incorporated. Extensive use of this facility has been made to develop the numerical model for intakes and many of the procedures described below are not supplied with the PHOENICS package.

2.2 Flow simulations

Simulations of the flow at intakes are performed for a particular set of river, sluice and canal discharges assuming steady state conditions. While the code does have the ability to model time varying flows, the approach would add complexity and would greatly increase computer run times. An approximation of steady conditions does not introduce significant errors.

2.2.1 Turbulence modelling

The flow in rivers is associated with very high Reynolds numbers where turbulence has a dominant effect. Predictions of turbulence are therefore required. In computational fluid dynamics such predictions are made using turbulence models, these are semi-empirical equations which describe the generation and decay of turbulence within a fluid and near its boundaries.

In recent years the k-ε turbulence model has been widely used and has been well proven in many engineering applications, including rivers (Rodi, 1984). The k-ε model has been used for this study.

2.2.2 Bed friction

Bed friction is described in the model by specifying an equivalent bed roughness height. A value for this height is derived by firstly determining the friction factor of the river channel. Either field measurements are used, or the results of an alluvial friction prediction method, such as van Rijn (1984). The equivalent roughness height which produces the correct friction factor is selected by a trial and error procedure.

2.2.3 Free surface

The free surface is modelled as a flat boundary with no friction. Potential errors are introduced by ignoring both the localised drawdown where flow accelerates and the slight damping of turbulence by the free surface. However, these errors were found to be insignificant. The former effect can be modelled where necessary.

2.2.4 Bed elevations and grid preparation

The studies presented in this report have used observed bed elevations in the vicinity of an intake as part of the model input. This approach is suited to applications where modifications to an existing intake are being assessed. For simulations of a new intake, the model should ideally predict the changes in river bed elevations which result from sedimentation or scour. Section 2.4.2 discusses methods which have been developed to predict river bed elevations.

A body fitted grid of computational cells is used to represent the irregular geometry of a river reach in the vicinity of an intake. Figure 1 illustrates the process of preparing a model grid. Other methods for describing the prototype geometry in the model have been tried, but this method has proved to be the most appropriate (Section 2.4.1).

2.2.5 Upstream boundary

The upstream boundary of the numerical model is taken at a river cross section well upstream from where the intake has influence on the flow. Velocity and turbulence variations at the upstream boundary are predicted by the model, it is assumed that an equilibrium profile has developed.

2.3 Modelling sediment

For the simulations presented in this report, sediment movement is modelled for steady state conditions. Deposition can occur, but the associated rise of bed levels with time is not simulated.

2.3.1 Settling velocities

Each sediment size fraction is modelled as a concentration within the flow and hence both its convection and diffusion are automatically simulated by the code. However, settling velocities are specified by imposing a downward flux of material for each size fraction which is proportional to its local concentration and its settling velocity. Simulations have been performed to verify that the predictions of this method agree with analytical solutions under a simple set of conditions.

2.3.2 Sediment diffusion coefficient

The CFD code predicts kinematic turbulent viscosity as part of the flow computations. However this viscosity, or "momentum diffusion coefficient", cannot be assumed to be equivalent to the sediment diffusion coefficient. Following other authors (for example van Rijn, 1984, and Kikkawa and Ishikawa, 1980) a constant ratio between the two coefficients, β, is adopted in the model.

If data are available, a value for β can be derived from observed concentration profiles in the river. For each size fraction the value of β is selected which produces the best match between predicted and observed profiles.

When observed concentration profiles are not available, a value can be selected on the basis of the observations of Coleman (1970). The following equation has been obtained by relating observed sediment diffusion coefficients derived from Coleman's data, and predictions by the model of turbulent viscosity:

$$\beta = 1 + 2.5 \frac{V_{sj}}{u_*} 3 \qquad (1)$$

where V_{sj} is settling velocity for size fraction j, and
u_* is shear velocity in the river.

2.3.3 Bed boundary condition

When all the sediment load passes through the domain being simulated, then there is no net sediment transfer between the flow and the bed. Under these conditions the bed boundary condition is specified as zero sediment flux at the bed.

If sediment transfer at the bed is important, then it is modelled as two processes:

(i) deposition onto the bed at a rate proportional to the settling velocity and the local concentration, and

(ii) entrainment from the bed at a rate determined from the local bed shear stress, sediment characteristics and the availability of sediment in the bed material.

The availability of sediment of a particular size will be related to the proportion of that size fraction within the bed material. Thus a prediction of the bed material grain size distribution is a necessary component in the bed boundary condition.

The appendix presents two methods used to determine bed boundary condition in the model, and the associated prediction of bed material sizes.

For each method, calibration is used to set an empirical constant so that, for a representative set of conditions, the predicted rate of sediment transport equals a predetermined value. This value is either derived from observations or is calculated by a sediment transport predictor, for example the Engelund and Hansen (1967) method.

2.3.4 Upstream boundary

In the same manner as for the prediction of velocities at the upstream boundary, the sediment concentration profile at that boundary is assumed to be in equilibrium.

2.4 Model development

Many aspects of the model described in this Chapter are the final result of an extensive period of development. Other, less successful, techniques were used in the model before the final method outlined above. This section presents some of this model development.

2.4.1 Model grid

A simple description of the geometry in the model was attempted, without fitting the grid to the river bed elevations and geometry in plan. A regular grid of rectangular cells was set up which covered the whole domain to be modelled. Those parts of the grid which were outside regions of flow, for example below the river bed in shallower parts of the domain, or the inside of a river bend, were blocked in the model. The potential advantage of this approach was the speed and ease of model convergence when rectangular cells are used. Three difficulties arose. Firstly, many more grid cells were needed to adequately represent the complex geometries. Secondly, the model produced more false diffusion which can, for example, reduce the strength of predicted secondary currents. Thirdly, the need to represent changes in river bed elevations as discrete steps in the bed gave rise to inaccuracies in the representation of the river bed boundary conditions.

The density of grid cells required to adequately represent prototype conditions in the model was determined by a trial and error procedure. For the vertical direction ten evenly spaced cells were found to be suitable. No improvement was found from grouping cells more towards the bed, which is a technique applied in some numerical models of flow and sediment transport (for example Kerssens et al, 1979, and de Vriend, 1981). The grids in plan were found to be adequately dense when a channel, such as a sluice channel, has at least ten cells across its width. The cell density longitudinally can be much less than the lateral density, due to a more gradual rate of change in conditions in that direction. Model grids in plan are shown for each field site discussed in Chapters 3 and 4.

2.4.2 Prediction of river bed elevations

The need to predict the effect of the intake structures on river bed elevations has been highlighted in Section 2.2.4.

Initially, an approach based on the prediction of critical shear stress at the threshold of movement was applied to the largest sediment size in the river bed. This produced a set of bed elevations at which all the material would be in transport during a large flood. The flood discharge was set using the concept of a dominant discharge for the river, which was assumed to be the bankfull discharge. Predicted bed elevations were found to represent poorly the observed bed elevations at the field sites discussed in Chapters 3 and 4. The method has not been adopted.

A method based on predicting the evolution of bed elevations with time has since been developed. The model grid is updated in response to the predicted deposition or scour, after each of a series of time increments. Long model run times are required for this method, and techniques to prevent grid distortions near fixed structures are yet to be developed. However, the method has been applied to a proposed intake in the UK, but no data are available for verification (HR Wallingford, 1993b).

Prototype river cross sections were used as model input for the simulations reported in chapters 3 and 4.

2.4.3 Flow resistance at banks

Flow resistance at the river bed is represented using mathematical formulae known as "wall functions". The same approach was initially used for the river banks. However, grid cells at the river banks are much wider than they are high. Conditions in a bank grid cell, especially the level of turbulence, are dominated more by the local bed friction than the bank friction. The CFD code used for the model requires turbulence to be determined from only one of these sources. Thus frictional forces produced by river banks are included in the model, but wall functions were not used to set turbulence parameters at the banks.

2.5 Model output

Model output can include plots of velocity vectors or streamlines on a plan of the river reach at the intake, or at cross sections. Contour plots of bed shear stress, sediment concentrations, deposition rates or bed material sizes can also be made.

However, when the performance of sediment exclusion facilities at an intake is to be assessed, the output of most direct significance are the sediment loads entering the canal for each sediment size fraction. These loads are used to calculate the overall performance ratio and the performance ratios for each fraction.

3 Comparison with data from Agno intake

The Agno intake is located on the Agno river, Pangasinan in Central Luzon, the Philippines. River discharges during the wet season are typically in the range 50 to 300m^3/s between floods. Bed material in the river consists of sands, gravel and cobbles.

The intake was constructed in the 1950's and supplies the Agno River Irrigation System which had a design service area of 18 500 hectares. A plan view showing the layout of the intake is given in Figure 2.

Sediment loads entering the canal system have been high, and severe sedimentation has occurred. Loss of conveyance capacity in the canals of the irrigation system has caused a reduction in cropped area, which fell below 6000 hectares during the period 1989 to 1990, Chancellor (1991).

Field monitoring was undertaken at the intake site in 1989. The study, which was undertaken jointly by HR Wallingford and the National Irrigation Administration of the Philippines, had the following objectives:

(i) to enable a comparison between field data and the predictions of the numerical model, and

(ii) to provide data to allow the design of improved sediment control facilities at the intake.

3.1 Data collection

Data collection covered the period from late August 1989 to March 1990, which represents long periods during the wet season (up to November) and during the dry season.

3.1.1 Sediment load and discharge measurements

Sediment load and discharge data were collected from the river at a bridge located 480m upstream from the intake. The pump sampling technique, as described by Crickmore and Aked (1975) and by Atkinson (1991), was used. This involved attaching a current meter and sampling nozzle to a sinker weight and suspending it from the bridge deck.

A sieve analysis was performed on all sediment samples collected, and the river sediment load was determined for seven size fractions, ranging from 0.09mm to 1.5mm.

Sediment load and discharge measurements were also made at; the head of the canal, the sluice gates and a section in the sluice pocket just downstream from its entrance, (Figure 2). Performance ratio was determined from these measurements. It was computed individually for each size fraction, and as an overall ratio for the intake.

3.1.2 Bed elevations

River bed surveys were conducted before and after the wet season (June to November) in 1989. Cross sections were taken at the upstream end of the sluice pocket and at 10m, 20m, 30m, 50m, 75m, 100m, 150m, 200m, 400m and 481m upstream. The cross section at 481m upstream from the sluice pocket was at the bridge from where measurements were being taken.

3.1.3 Bed material sizes

Samples were taken from the river bed material. Due to the difficulties in obtaining samples containing cobbles of up to 200mm in size, the samples were not collected under flowing water during the measurement period. Twelve samples were collected from the river bed in the period December 1989 to March 1990, which was during the low flow period after the 1989 wet season.

Figure 3 shows grading curves for the samples collected.

3.2 Application of the numerical model

A model of the Agno intake was set up which covered the reach of river from the weir to 190m upstream and included the sluice pocket, sluice gates and canal gates. Bed elevations were used to form the model grid as discussed in Section 2.2.4. However, there was uncertainty over the datum for some of the survey cross sections and inconsistencies in the bed elevations were overcome by adjusting datums. The model grid employed is shown in Figures 4 and 5. The edges of the water surface moved tens of meters as river discharge varied, so the grid could not be fitted to well defined river banks. Therefore a rectangular grid in plan was chosen for this application.

Observed flow conditions at the bridge on 25th August 1989, and the measured river slope of 0.0042, were used to set roughness in the model, as described in Section 2.2.2. Data from 25th August 1989 were also used to set the sediment diffusion coefficient, β, as described in Section 2.3.2.

3.3 Results

3.3.1 Qualitative results

Field measurements indicated that the intake was effective in excluding sediment from the canal. The mean value for performance ratio, PR (as defined in Section 1.2), was 0.52, which implies a 52% reduction in sediment concentration between the river and the canal entrance. The model also gave this result (the mean of predicted PR was 0.52).

The mechanism by which the intake excludes sediment can be discerned from the numerical model output. Figure 6 shows predicted flow vectors plotted on a plan view of the intake, flow at the surface and at the bed is shown. As the flow approaches the intake, it has higher momentum near the surface and so the flow path is less curved at the water surface. The greatest curvature of flow is at the bed, where the sediment has highest concentration. Thus the sediment is swept away from the intake gates. Predicted streamlines, which are plotted on Figure 7, show how the flow paths approaching the intake at the river bed pass through the sluice gate furthest from the intake, while the flow approaching the intake at the river surface passes close to the intake. Figure 8, a cross section through the sluice pocket upstream from the canal entrance, shows the effect of the bed load sweep on sediment concentrations: the lowest sediment concentrations were found at the right hand side of the sluice pocket, where the canal intake gates are sited.

Figure 9, shows sand concentrations entering the canal plotted against concentration at the river surface. The figure demonstrates that these concentrations are similar, and so it is largely the flow at the river surface which was entering the canal. Thus the water with lowest sand concentrations was entering the intake and, therefore, the intake was performing near the optimum possible for conditions of continuous sluicing.

3.3.2 Comparison with observations

Five sets of conditions observed at the Agno intake are listed in Table 1. Both observed and predicted performance ratios are given in the table, and agreement between them is good: a discrepancy ratio, defined as observed PR divided by predicted PR, has a mean value of 0.99 and standard deviation of 0.25.

Performance ratio has also been analyzed for each size fraction (Section 3.1.1). Figure 10, which shows PR plotted against sediment grain size, indicates a trend of rising PR with grain size. This is due to the larger material being more concentrated towards the river bed. Predicted performance ratios have been included in Figure 10 and the good agreement between model results and observations at Agno can be seen.

Predictions using the model are included on Figure 9. Again there is agreement between prediction and observation, which gives further confidence in the accuracy of the model.

3.3.3 Sensitivity tests

Conditions at Agno on 22nd August 1989 were used to test the sensitivity of the model predictions to changes in the model input and some modelling assumptions.

The following tests were made.

(i) Sensitivity to changes in bed roughness, which was derived by calibration, was investigated by doubling the bed roughness height.

(ii) The sediment diffusion coefficient was derived by calibration against data. When no data are available then equation (1) is recommended, the effect of using equation (1) in place of calibration was assessed.

(iii) The method for calculating sediment entrainment at the bed is given maximum accuracy by ensuring that its predictions agree with those of the well established Engelund and Hansen (1967) sediment transport predictor (Section 2.3.4). This is achieved by adjusting an empirical constant in the formulae on which it is based (the Garcia and Parker (1991) method; see Appendix, Section A2). The effect of not including this adjustment, which had reduced predicted transport rates by a factor of 5.3, was tested.

(iv) The effect of using the alternative function for predicting sediment entrainment at the bed (as presented in the Appendix, Section A3) was tested.

(v) Hiding effects in sediment mixtures is a component of the alternative function for predicting sediment entrainment. The effect of hiding predictions was quantified by omitting this component in a test model run.

(vi) There was some uncertainty over river bed elevations (Section 3.2) sensitivity to this was assessed by lowering bed elevations in the model. Bed elevations near the weir were known with most certainty, so lowering of the bed was performed about an axis at the weir. The greatest lowering of 1m applied at the upstream end of the model.

The following table gives the results of the sensitivity tests:

Test	Description	PR Prediction :	
		Value	Change
-	Base condition	0.52	
(i)	Doubling the bed roughness height	0.47	-10%
(ii)	Sediment diffusion coefficient derived using Equation (1)	0.50	-4%
(iii)	No adjustment of the sediment entrainment formulae	0.48	-8%
(iv)	Alternative function for predicting sediment entrainment	0.55	+6%
(v)	No hiding effects included with (iv)	0.57	+4% *
(vi)	Uncertainty over river bed elevations	0.71	+37%

Note: * change is with respect to (iv)

These results show that the model of the Agno intake is not unduly sensitive to any of the assumptions or uncertainties in model input, with the exception of the river bed levels. However, in the case of bed elevations, a 1m change at the upstream end is large when compared against likely errors in the survey data.

4 Comparison with data from Kapunga intake

The Kapunga intake supplies the primary canal of the 3,800 hectare Kapunga Rice Project in Southern Tanzania. The design capacity of the canal is $4.6m^3/s$.

Water is drawn from the Great Ruaha river which typically has a discharge in the wet season ranging from $15m^3/s$ to $50m^3/s$. Flood peaks rise above $200m^3/s$ in some years. The river bed material consists of sands and gravel, but the material in transport has been found to consist largely of fine sand and silts.

River sediment loads were not known at the time of design, but high sediment loads observed in rivers nearby prompted the adoption of sediment exclusion measures at the intake. The intake is sited on the outer bank of a river bend to encourage relatively sediment free water to enter the canal. Also, a curved channel sediment excluder has been included in the design of the intake to reduce further the entry of sediment to the canal. A plan view of the intake is shown in Figure 11.

4.1 Data collection

Monitoring of the intake was undertaken during the wet season (February to May) in 1991 and 1992. The monitoring was carried out in collaboration with the National Agriculture and Food Corporation, Ministry of Agriculture, Government of Tanzania, (NAFCO).

Atkinson (1991) presents the details of the measurements taken at Kapunga and details of their analysis. Atkinson (1994) presents the results of the monitoring over both seasons and discusses their implications for intake operation and future maintenance requirements. In this report only an outline of the measurements is given.

Figure 12 shows the measurement locations on a plan view of the intake site.

4.1.1 Sediment load and discharge measurements

Discharges and sand concentrations in the river were monitored at a cableway sited about 150m upstream from the weir. The pump sampling technique was applied in a similar manner to that employed at the Agno intake. Again, a sieve analysis was performed on the sediment samples obtained, so that river sediment load was determined for each of four size fractions in the range 0.07mm to 0.3mm (some coarser sand was present in transport, but the presence of organic matter of similar size rendered the sediment load values for the coarser material inaccurate).

Sediment concentration and discharge were also measured at the head of the canal and in the sluice channel immediately downstream from the intake (Figure 12). Comparisons between concentrations enabled the sand exclusion performance of the intake to be assessed both as an overall performance and as a performance for the curved sluice channel only.

4.1.2 Bed elevations

A survey of river bed elevations was undertaken at the end of each season of measurements. The location of the survey sections are shown on Figure 12. Comparison between the two surveys indicated that no significant changes in bed elevations occurred between the two seasons of monitoring (Atkinson, 1994).

4.1.3 Bed material sizes

River bed material samples were collected in March 1991 and March 1992. Figure 13 shows the grading curves derived from these samples.

4.2 Application of the numerical model

A river reach of about 150m was included in a model of the Kapunga intake. The upstream boundary of the model was set at the cableway site (Figure 12), and the downstream boundary was the weir. The 1991 survey results were used to form bed elevations in the model grid (Section 2.2.4). The model grid employed is shown in Figure 14.

Typical values for depth and velocity at the cableway site during the 1991 measurements were used to set roughness in the model. The local river slope was not measured, and so the friction prediction method of van Rijn (1984) was used to determine river slope.

Observations of sediment concentrations at the cableway site were used to set the sediment diffusion coefficient, β.

4.3 Results

4.3.1 Qualitative results

Field measurements indicated that the intake was effective in excluding sediment from the canal (Atkinson, 1991, and Atkinson, 1994). The mean value for performance ratio was 0.57 when results for both seasons of observations were averaged. A set of five observed conditions, which covered the range of discharges and water levels found at the intake, were chosen to test the model. The mean PR for these five points was 0.58.

The principal mechanism by which the Kapunga intake was excluding sediment during the observations in 1991 and 1992 appeared to be secondary currents produced by the river bend (Atkinson, 1994). Both the similarity between the sand concentrations at the river surface and those entering the canal, and the relatively low PR value for the sluice channel (PR_{sl}), indicated this mechanism. However there was no direct evidence that secondary currents in the river bend were strong.

Figures 15 and 16 show model predictions of flow directions in the river bend at Kapunga. The predictions indicate relatively weak secondary currents. This result was unexpected because relatively strong secondary currents had been observed on a physical model of the intake site. However, the result was later confirmed by field observations of floating trash paths. Further simulations of flow in the river bend showed that the expansion was the cause of the weak secondary currents: a river bend of similar dimensions but a with no lateral expansion at its entry showed relatively strong secondary currents.

The model results indicated that the mechanism by which sediment exclusion was being achieved was a combination of secondary currents in the river bend and some temporary sediment deposition in the river. When predictions of deposition were included in the model, it indicated a mean PR of 0.80 over the five conditions simulated, and when deposition was prevented in the model this value was 0.47. It is likely that the effect of temporary sediment deposition on the prototype is considerably less than these predictions indicate. The predicted performance ratio with no deposition was closer to observation (PR=0.58) than that with deposition, and the predicted rate of deposition was too large to be sustained. Deposition rates at the entrance to the sluice channel ranged from 0 to 23mm/day, with a mean of 9mm/day. It is possible that the bed was lowered by erosion during the periods of low water levels in the dry season and then deposition occurred during the higher water levels in the wet season, when the measurements were taken. However deposition at 9mm/day sustained over the whole wet season would imply a total deposition depth of about 1.4m. The flow depths at that point ranged from 1.7m to 3.0m so it is unlikely that the bed could rise 1.4m without causing strong re-erosion.

Model predictions with no sediment deposition represent conditions in the wet season after a period of bed level adjustment, i.e. after any deposition has occurred. The measurements were also taken after any bed level adjustments during the dry season. Comparisons between model predictions and observations have been performed with the assumption of no deposition.

4.3.2 Comparison with observations

The five sets of conditions covering the range of flows at the intake, which were chosen for testing the model, are listed in Table 2. Both observed and predicted performance ratios are given in the table and agreement between them is reasonable: a discrepancy ratio, defined as the mean observed PR divided by the mean predicted PR, has a value of 1.23. The standard deviation of the discrepancy ratios for the individual comparisons is 0.68, which represents considerable scatter.

The predictions of the performance ratio for the sluice channel are also good: mean discrepancy ratio is 1.07. A standard deviation for individual discrepancy ratios would not be meaningful due to their low absolute values. Nevertheless, scatter in PR_{sl} predictions is considerable.

Performance ratio has also been analyzed for each size fraction and Figure 17 shows PR plotted against sediment grain size. Again the trend of rising PR with grain size is apparent, and good agreement between model results and observations can be seen. The equivalent plot for performance of the sluice channel (Figure 18) shows the lower PR_{sl} values and the greater scatter in both observations and predictions. Agreement between prediction and observation is still reasonable.

Figure 19 shows sand concentrations entering the Kapunga intake plotted against concentrations at the river surface. The similarity between these concentrations shows that intake performance is near optimum. Model predictions have been included on the figure, and again there is good agreement between prediction and observation.

4.3.3 Sensitivity tests

At Kapunga a representative set of flow conditions were used for the sensitivity tests. They were: a river discharge of $32m^3/s$, a canal discharge of $1.2m^3/s$, a sluice discharge of $5.7m^3/s$, a water level of 1059m and a sand concentration in the river of 133ppm.

The sensitivity tests described in Section 3.3.3 were repeated for the model of the Kapunga intake. The following table gives the results of tests which could be applied when no deposition is assumed:

Test	Description	P R Prediction :	
		Value	Change
-	Base condition	0.55	
(i)	Doubling the bed roughness height	0.59	+7%
(ii)	Sediment diffusion coefficient derived using Equation (1)	0.33	-40%

Little sensitivity to uncertainties in river roughness was shown. However, sensitivity to the choice of sediment diffusion coefficient was apparent.

Therefore, it is recommended that the sediment diffusion coefficient should be determined wherever possible from measurements of sediment concentration profiles in the river.

Many of the sensitivity tests described in Section 3.3.3 relate to the simulation of sediment deposition. Therefore the tests were also applied at Kapunga for the case when sediment deposition is included in the predictions of the model. The results are listed below:

Test	Description	PR Prediction :	
		Value	Change
-	Base condition	0.79	
(i)	Doubling the bed roughness height	0.81	+3%
(ii)	Sediment diffusion coefficient derived using Equation (1)	0.76	-4%
(iii)	No adjustment of the sediment entrainment formulae	0.85 #	+8%
(iv)	Alternative function for predicting sediment entrainment	0.81	+3%
(v)	No hiding effects included with (iv)	0.82	+1% *

Notes: # the adjustment increased predicted transport rates by a factor of 5.3
* change is with respect to (iv)

In this case, the model predictions show no undue sensitivity to the modelling assumptions or uncertainties in input.

4.4 Physical model of the Kapunga intake

A physical model study was employed during the design process of the Kapunga intake to test and refine the design of the curved channel sediment excluder (Tosswell, 1989). While some quantitative measurements of sediment abstraction by the intake were made on the model, the results were not claimed to have direct quantitative relevance to the prototype. These measurements do, however, allow for a further qualitative test of the numerical model.

Observations on the physical model indicated that the performance of the prototype would be good. Secondary currents were observed to be strong in both the river bend and the sluice channel. When quantitative measurements of sediment loads were made on the model, almost complete sediment exclusion was observed (PR > 0.9). The numerical model also predicted this good performance when it was used to simulate the physical model. The reasons for the overestimate of performance ratio produced by the physical model were threefold. Firstly, there were the problems of sediment scaling discussed in Section 1.3. Secondly, the expansion at the river bend was more pronounced on the prototype than was represented on the physical model. This expansion was found to inhibit the development of secondary currents (Section 4.3.1). Thirdly, scaling effects caused the roughness of clean concrete in the sluice channel to be overestimated in the physical model. Friction factor (defined as the ratio of mean velocity to shear velocity) in the sluice channel was approximately halved for the scale of the physical model, causing secondary currents to be overestimated at a model scale.

At an early stage in the testing programme on the physical model, it was found that visual examination of flow patterns (such as the strength of the secondary flow patterns in the sluice channel) was the most satisfactory method for assessing each proposed design. Figure 20 shows the initial layout for the intake structures and the final layout following the tests. When the numerical model was applied at the scale of the physical model, the numerical model predicted a large improvement in sediment exclusion between the initial and final layouts: a sevenfold reduction in sediment abstraction was predicted (Atkinson et al, 1993).

When the effect of the design improvements was assessed using the numerical model of the field scale intake, the predicted improvement was only slight. This conclusion was supported by the field measurements. They indicated that the impact of the sluice channel on sediment concentrations entering the intake was relatively low, and so improvements to its design could not have had a significant effect on overall sediment exclusion.

The comparison between the physical model, the field observations and the numerical model applied at both scales highlights the relative advantages of the numerical model in predicting the quantitative performance of the prototype.

5 Conclusions and recommendations

(i) A technique for the numerical modelling of flows and sediment movement at river intakes has great potential. The technique enables quantitative prediction of the performance of sediment control structures at an intake, which is difficult to achieve with physical models due to scaling problems. Numerical modelling is also usually cheaper.

(ii) A numerical model has been described and its comparison with field data collected at two intakes has been presented. When the performance of an intake is assessed using the performance ratio, PR, defined:

$$\mathrm{PR} = 1 - \frac{\text{sediment concentration entering intake}}{\text{concentration being transported by the river}}$$

the field verification of the model yielded:

Field site	Number of observations	Mean predicted PR	Mean observed PR	Discrepancy ratio *	
				Mean	Standard deviation
Agno	5	0.52	0.52	0.99	0.25
Kapunga	5	0.47	0.58	1.23	0.68

Note: * discrepancy ratio is defined as the ratio of observed PR to predicted PR

The numerical modelling technique has also been applied to an intake at the Narora barrage, for which field data is reported in the literature. Predicted and observed PR for this site were 0.07 and 0.04 respectively.

(iii) Sensitivity tests on the results for Agno and Kapunga showed that the predicted PR was not strongly dependent on the modelling assumptions or uncertainties in input parameters. However, there was a significant dependence on the river bed elevations for the Agno intake, where survey difficulties caused uncertainty in the model input. There was also sensitivity to the choice of sediment diffusion coefficient for the Kapunga intake. Both these potential causes of inaccuracy can be prevented by utilising the results of flow measurements: river bed surveys and sediment concentration profiles in the river respectively.

(iv) A summary of results for these three sites, two of which also have predictions of performance derived from physical modelling are:

Field site	Mean predicted PR	Mean observed PR	Mean PR from physical model
Agno	0.52	0.52	-
Kapunga	0.47	0.58	0.99 *
Narora	0.07	0.04	0.92 +

Notes: * the model observations to derive this value were qualified as having "limited use for design".

+ reasons for this overestimate include an inappropriate choice of bed material in the model.

Figure 21 shows these results graphically. The potential advantage of a numerical modelling approach, when quantitative predictions are required, is demonstrated.

(v) A limitation in applying the numerical model is the need to include observed river bed elevations in the input. When the model is required to make predictions at an intake before its construction, the effect of the intake structures on bed elevations can only be estimated. The model, as described in this report, is therefore primarily suited to assess modifications to existing intakes rather than the design of new intakes.

Further model development has been undertaken to enable predictions of river bed elevations in the vicinity of a new intake. However, no comparisons with data have been made.

6 Acknowledgements

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Tables

Table 1 Summary data for model verification at Agno

Date	Discharge in			Stage at intake (m)	Sand concentration		Performance Ratio		
	River (m^3/s)	Sluice (m^3/s)	Canal (m^3/s)		River (ppm)	Canal (ppm)	Observed	Predicted	Ratio
22.8.89	290	129	11.0	98.76	1090	331	0.70	0.52	1.35
25.8.89	183	133	9.2	98.2	269	144	0.46	0.50	0.92
28.8.89	134	117	8.0	98.2	156	82	0.47	0.57	0.82
5.9.89	145	99	11.4	98.5	186	70	0.62	0.56	1.11
3.10.89	183	137	16.8	98.59	269	178	0.34	0.47	0.72
					Mean		0.52	0.52	0.99

Table 2 Summary data for model verification at Kapunga

Date	Discharge in			Stage at intake (m)	Sand concentration		Performance Ratio		
	River (m^3/s)	Sluice (m^3/s)	Canal (m^3/s)		River (ppm)	Canal (ppm)	Observed	Predicted	Ratio
8.4.91	28.4	4.2	0.99	1059.57	167	35	0.79	0.37	2.14
15.5.91	15.8	6.1	1.22	1058.26	25	13	0.48	0.73	0.66
29.2.92	40.7	8.0	3.07	1058.90	87	37	0.57	0.50	1.14
16.4.92	54.7	7.2	2.20	1059.52	117	67	0.42	0.19	2.21
23.4.94	38.0	4.8	2.96	1058.47	100	35	0.65	0.54	1.20
					Mean		0.58	0.47	1.23

Figures

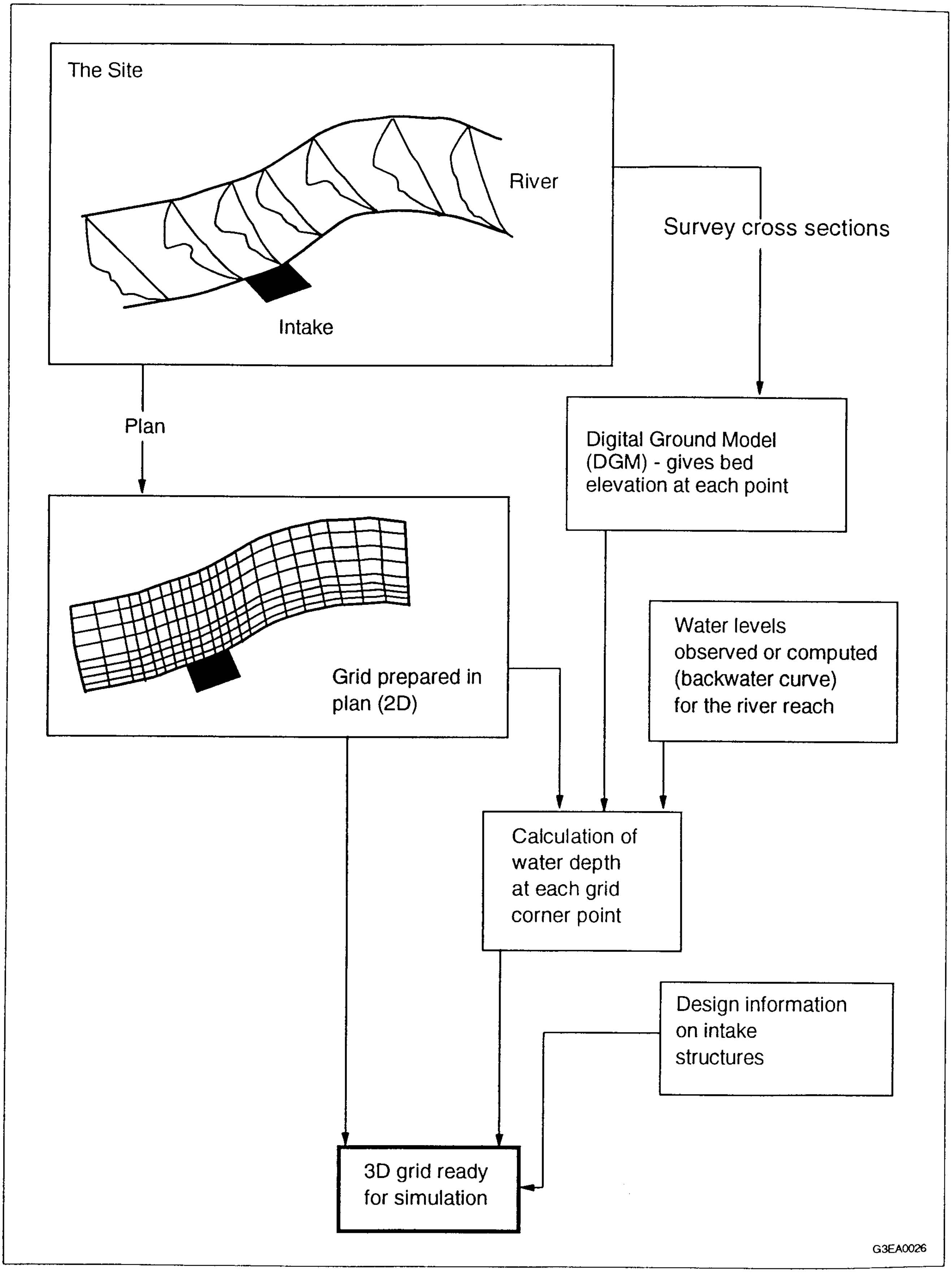


Figure 1 Preparing the model grid

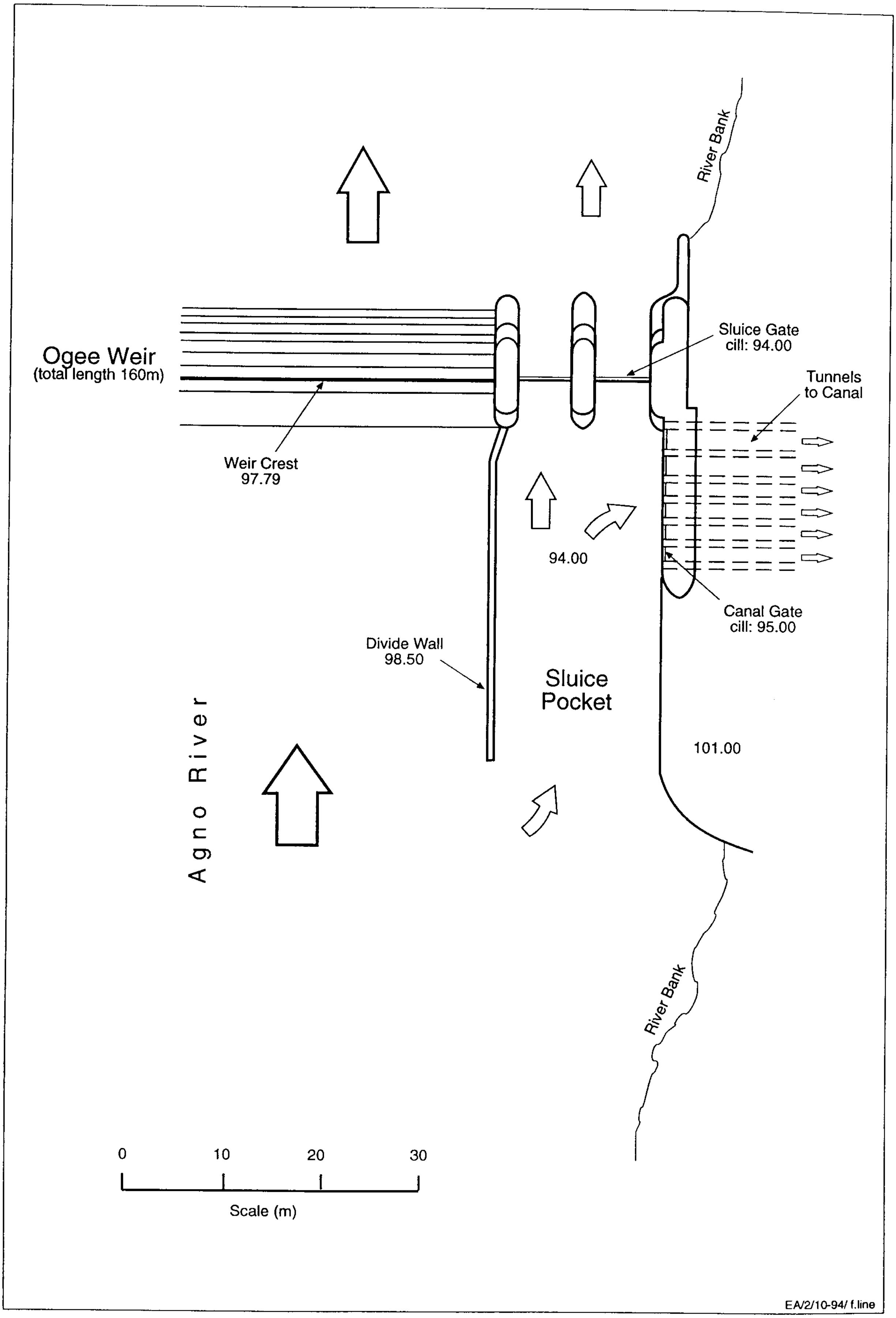


Figure 2 Layout of the Agno Intake

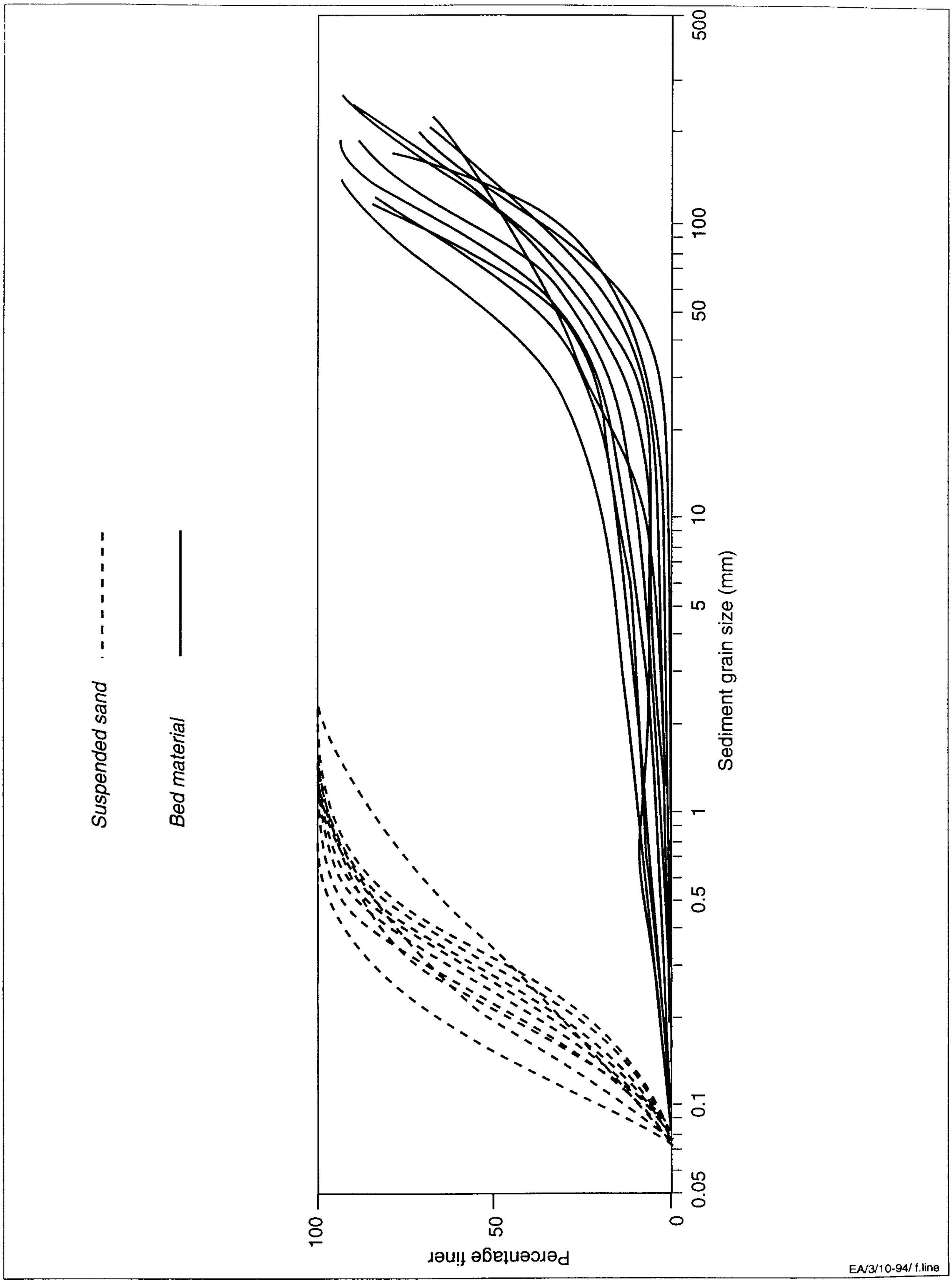
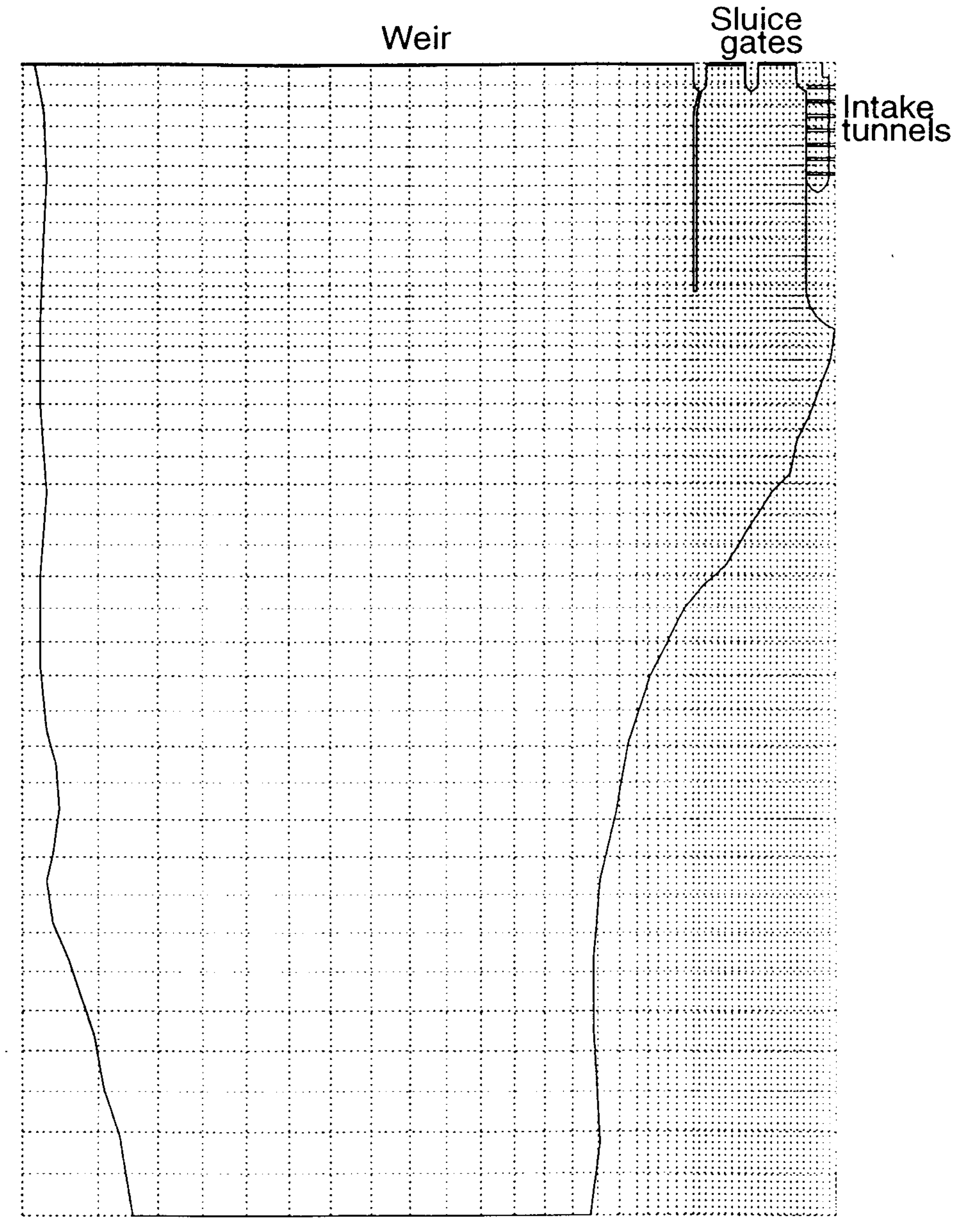


Figure 3 Sediment size grading curves for the river, Agno



(Plan shows the approximate location of the edge of the water surface on 25th August 1989)

G3EA0013

Figure 4 Model grid in plan, Agno

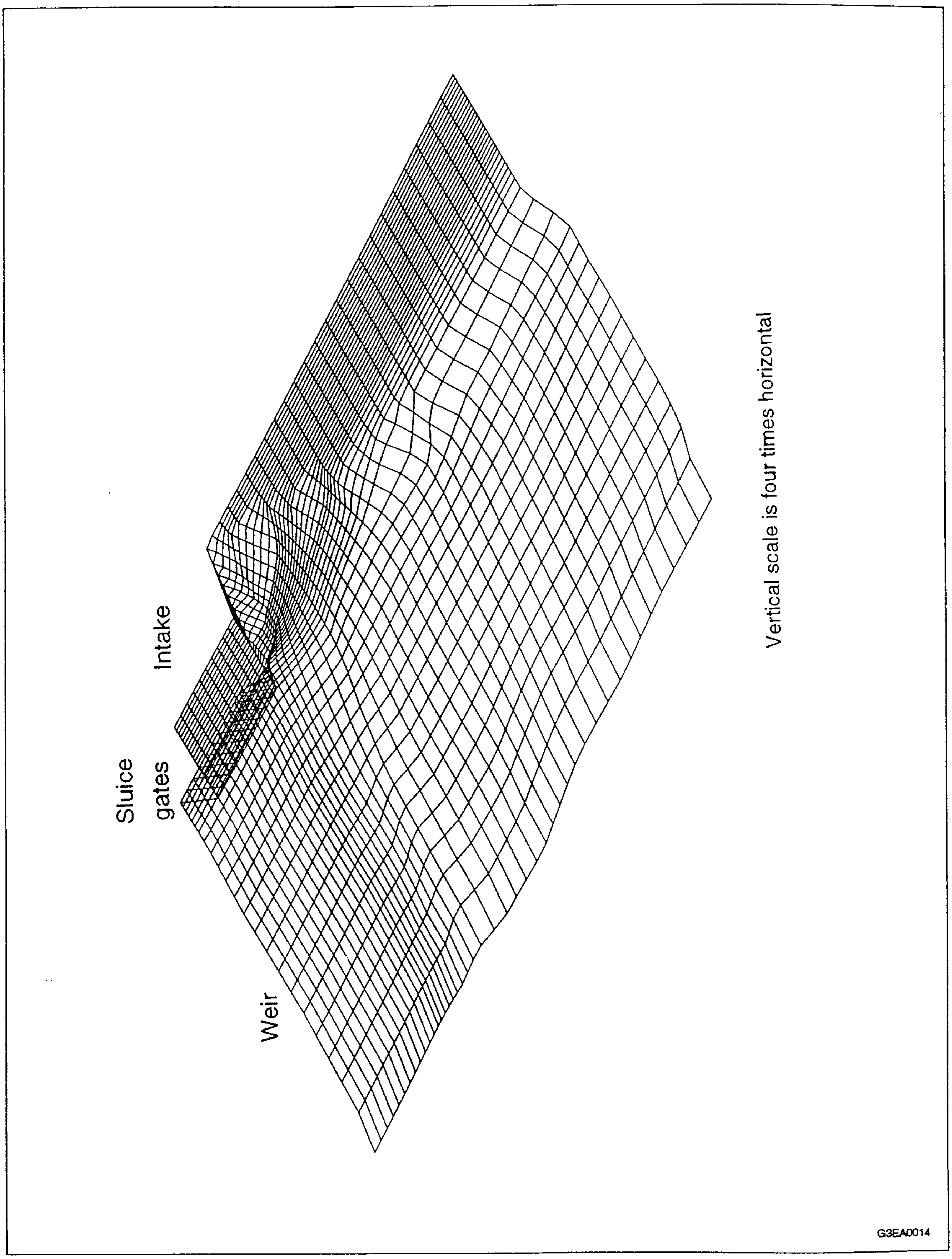
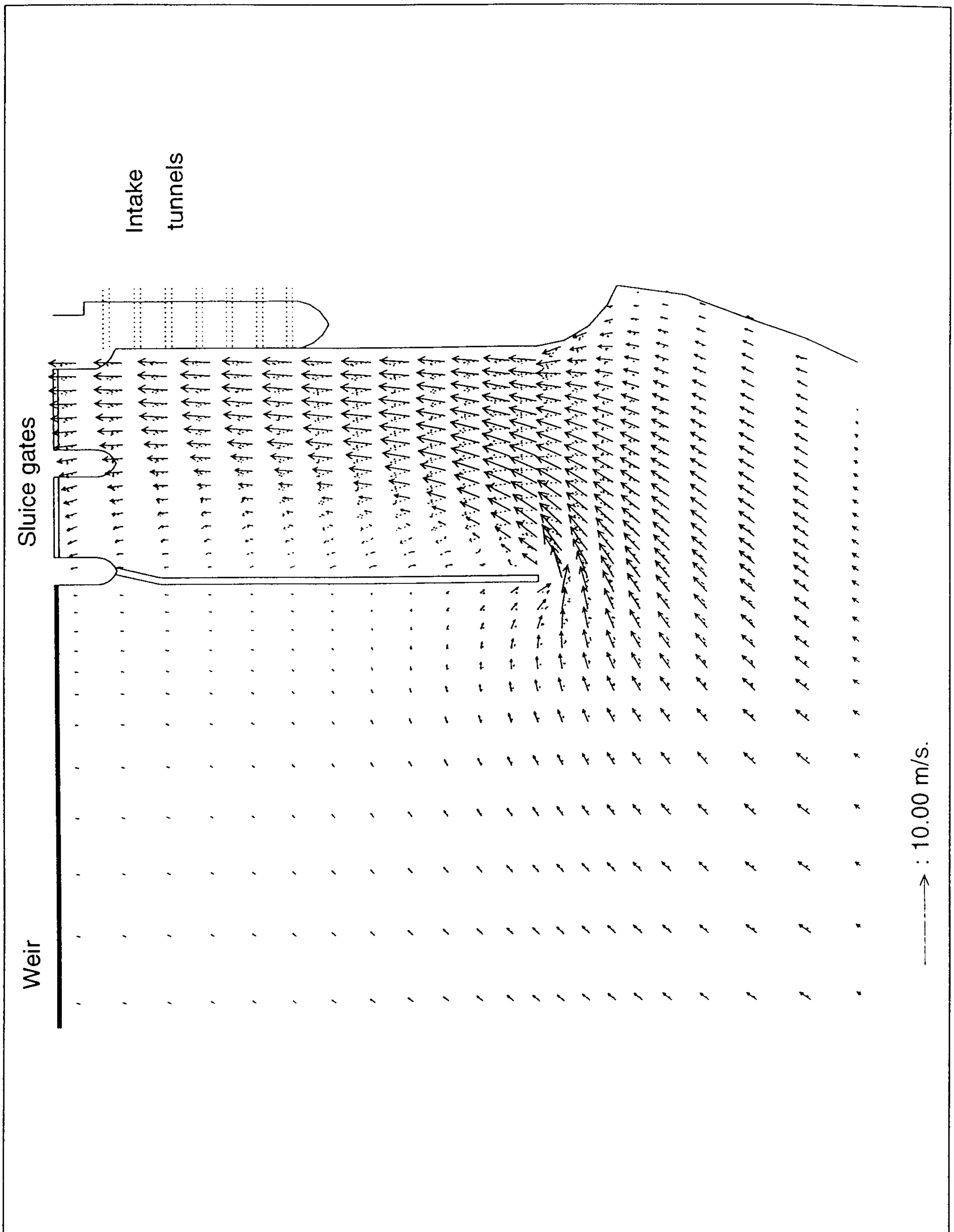


Figure 5 Isometric view of river bed, model grid for Agno intake



Flow vectors at the surface are shown as full lines, flow vectors at the bed as dotted lines

G3EA0015

Figure 6 Predicted flow vectors, Agno

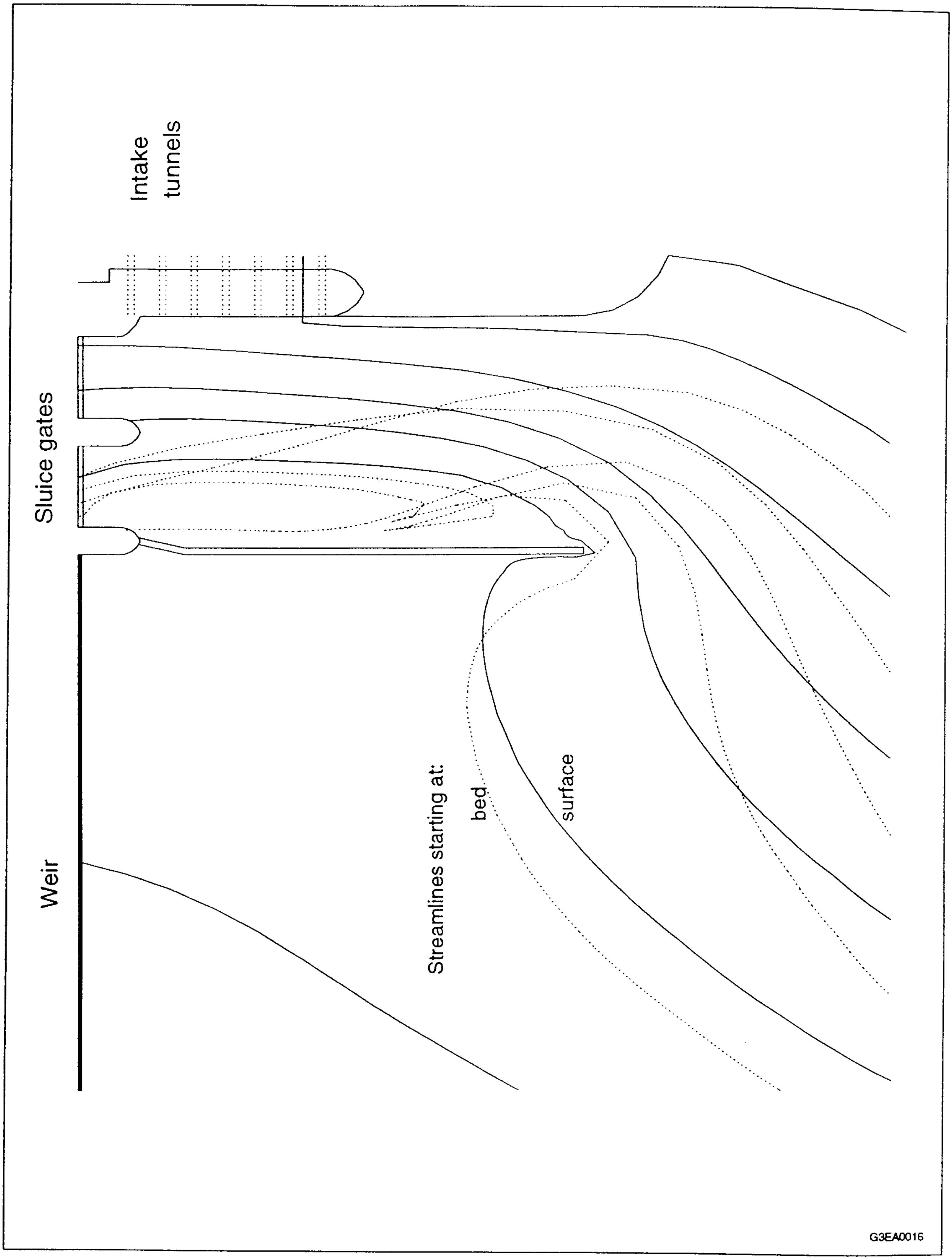


Figure 7 Predicted streamlines, Agno

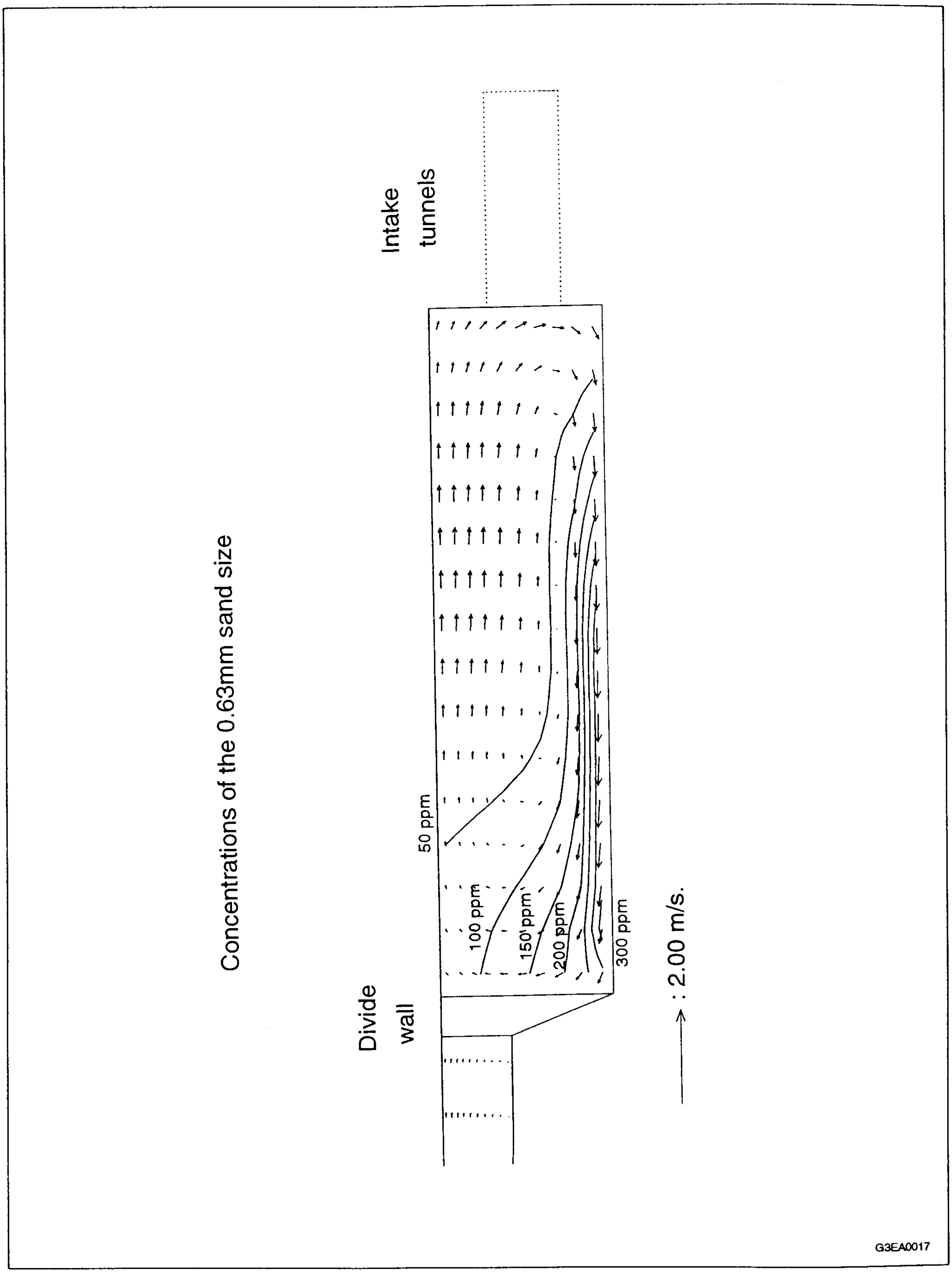


Figure 8 Cross section through sluice pocket, Agno Intake, showing flow and sediment concentrations

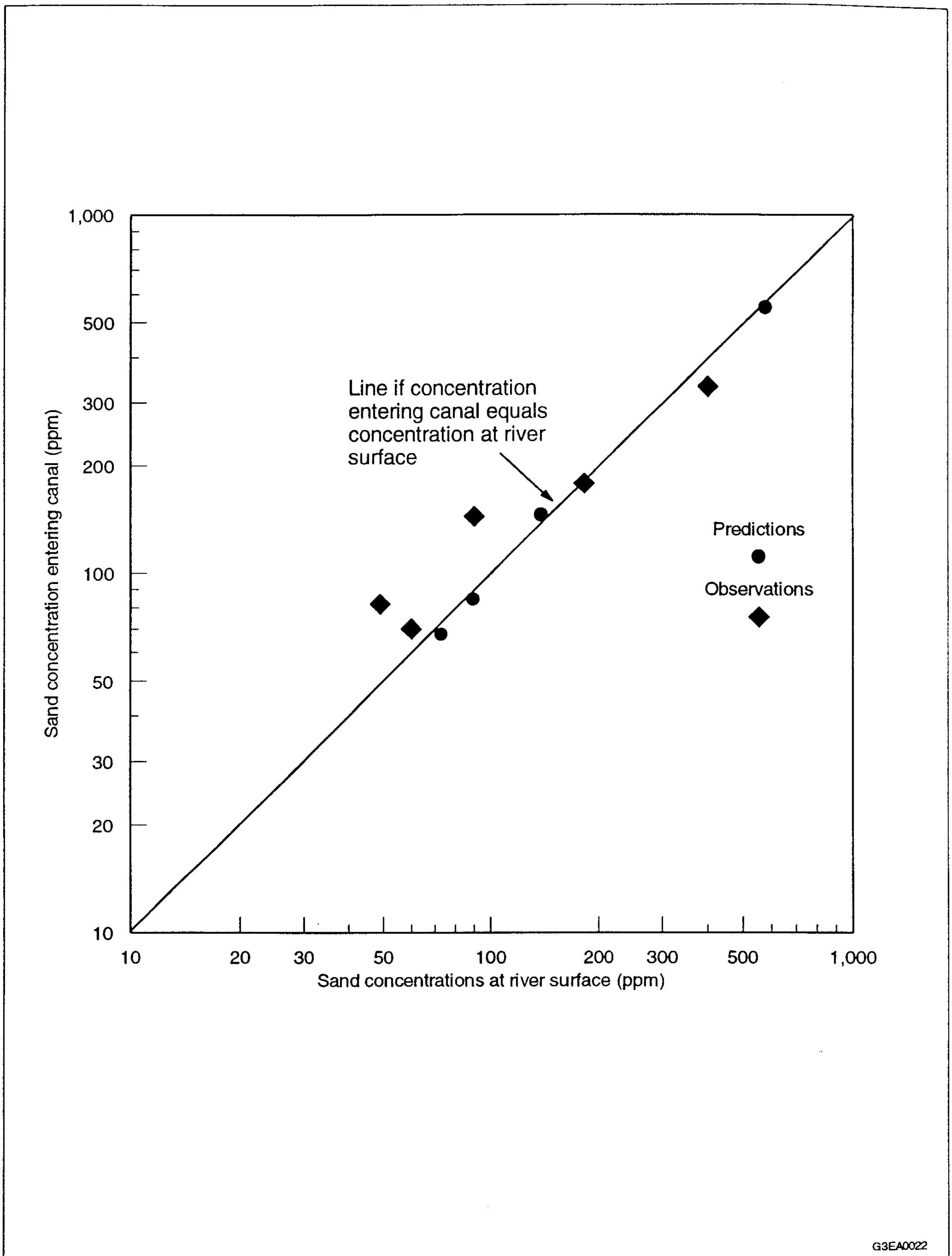


Figure 9 Comparisons between sand concentrations at river surface and entering the canal, Agno

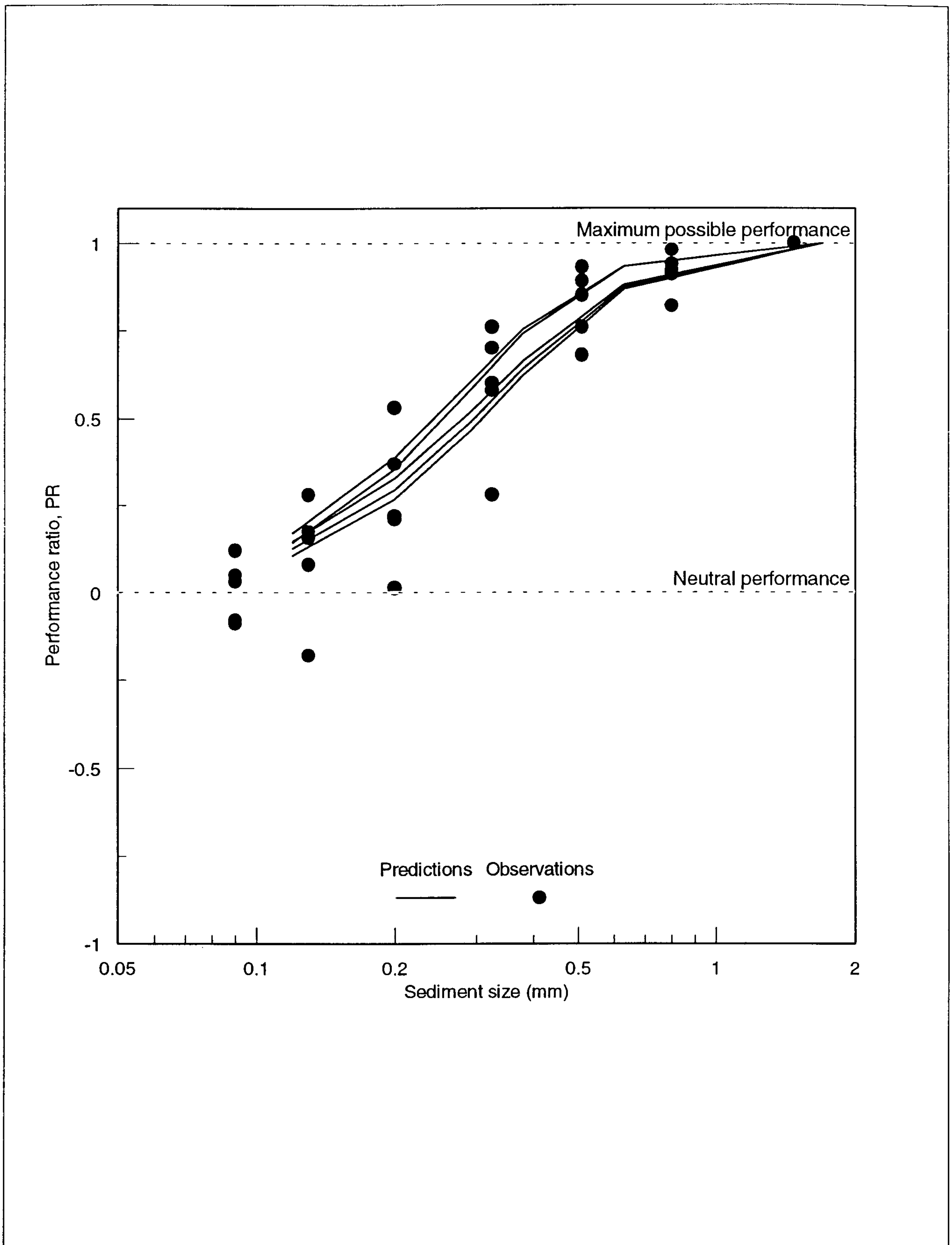


Figure 10 Predicted and observed performance ratio as a function of sediment size, Agno

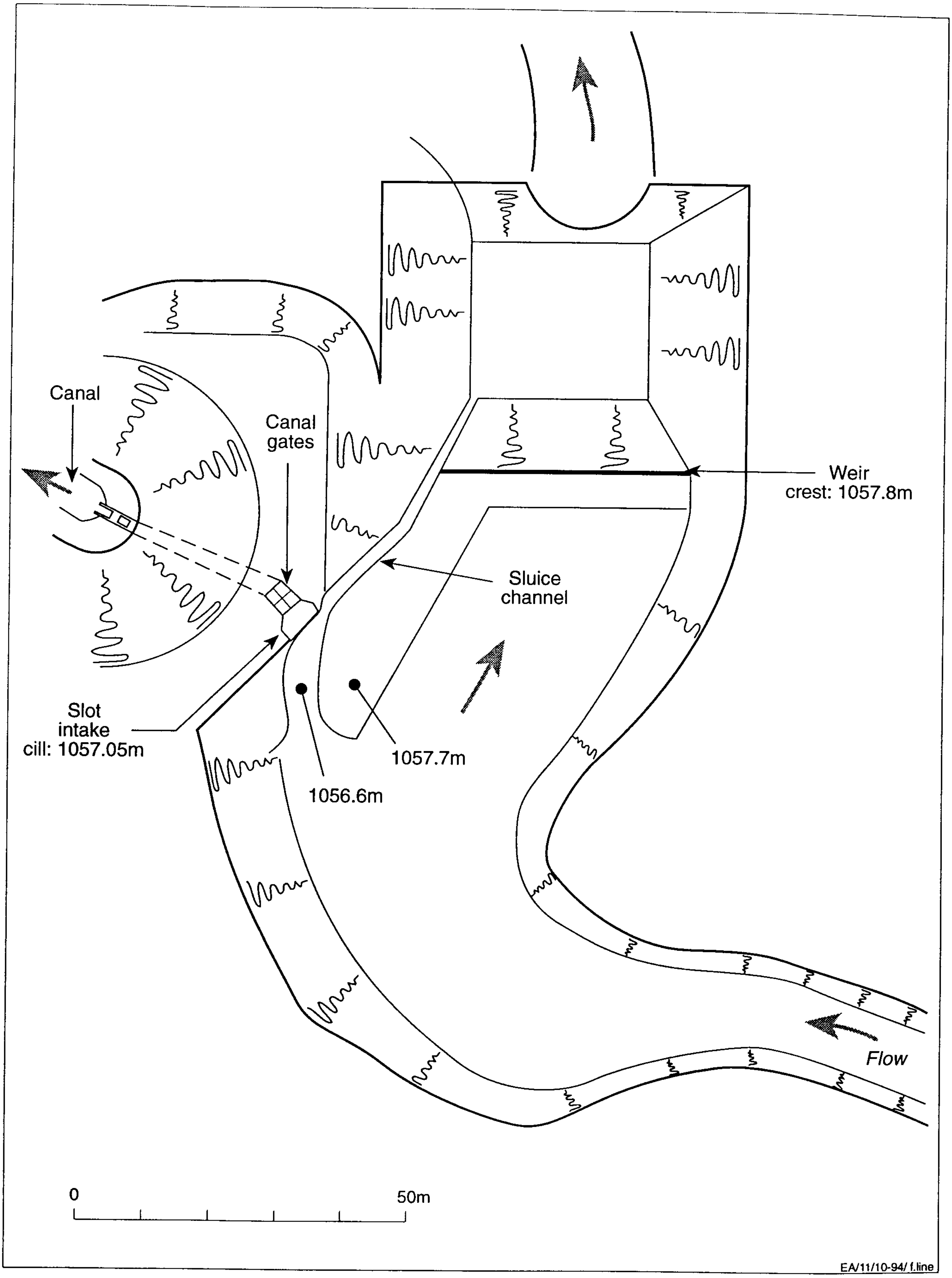


Figure 11 Plan of Kapunga headworks

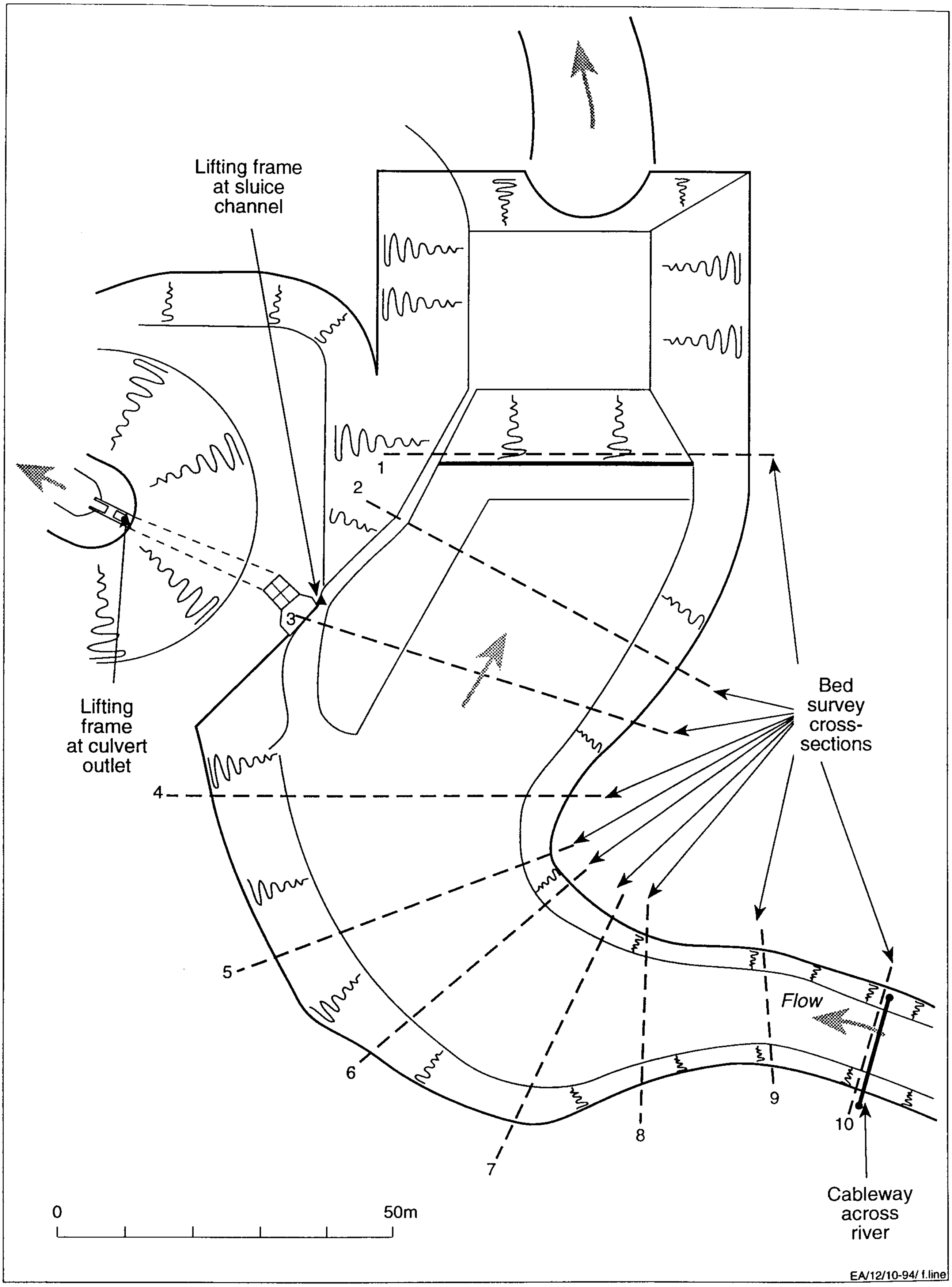


Figure 12 Plan of Kapunga headworks showing measurement locations

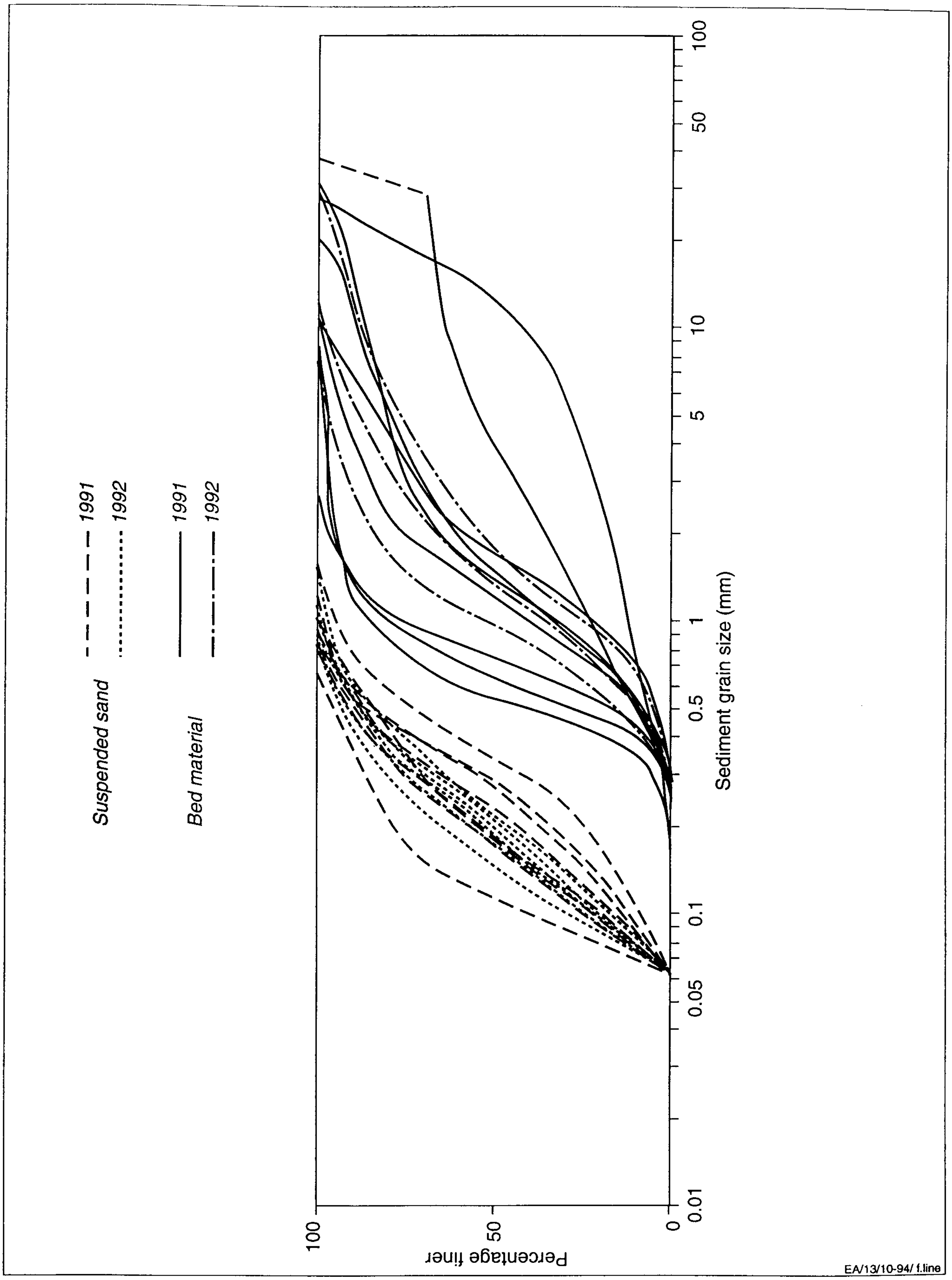


Figure 13 Sediment size grading curves for the river, Kapunga

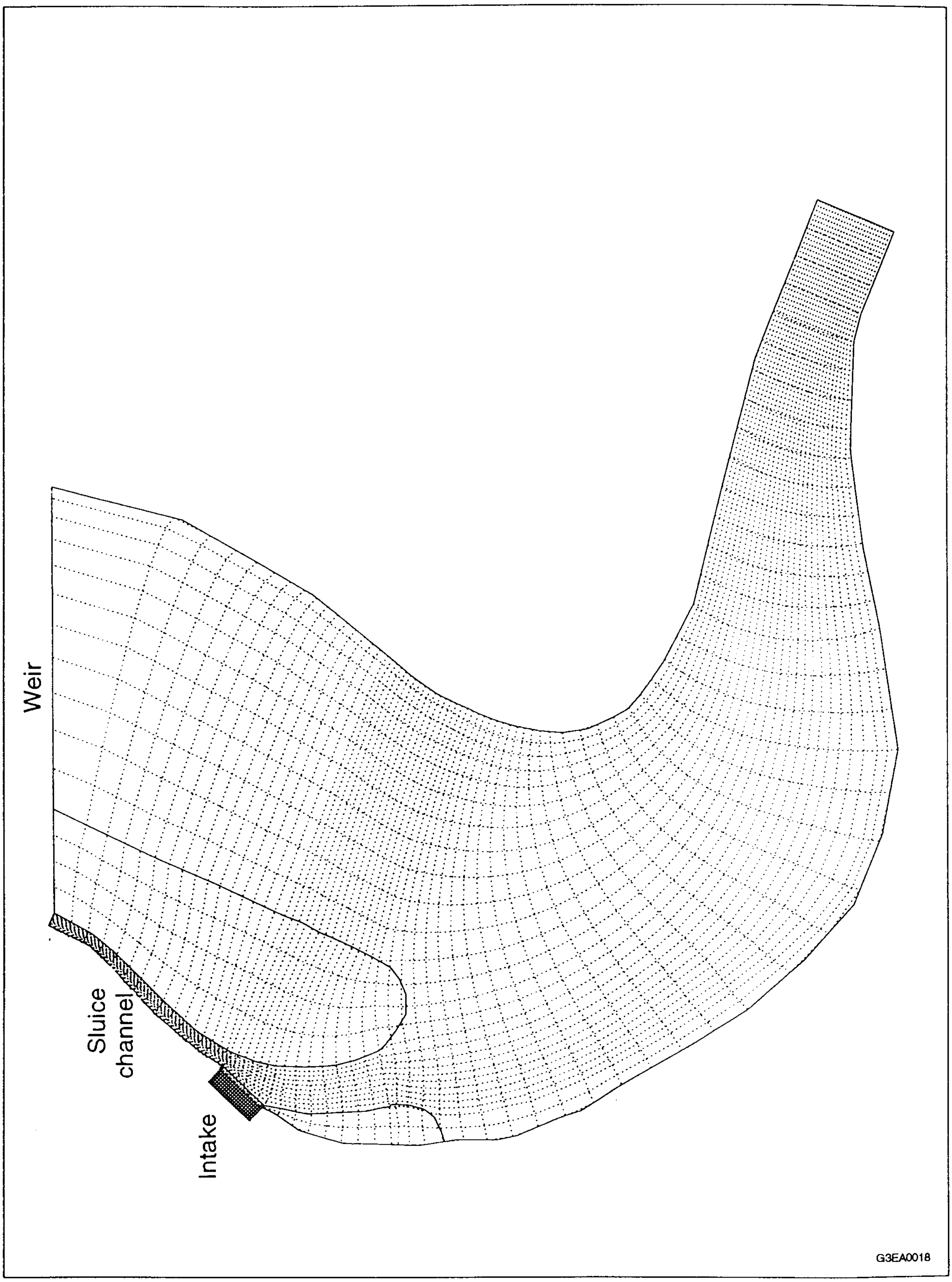


Figure 14 Plan view of model grid, Kapunga

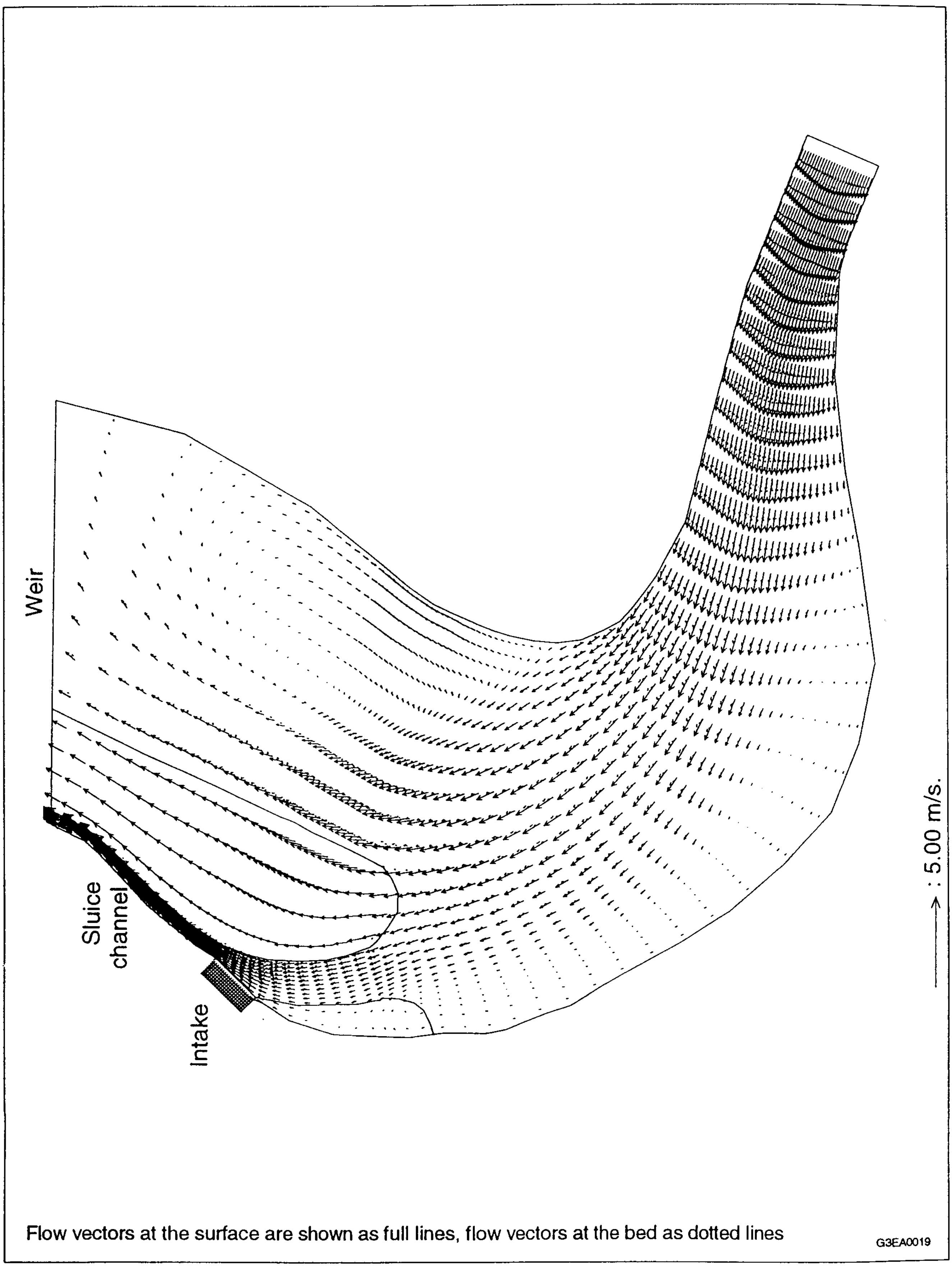


Figure 15 Predicted flow vectors, Kapunga

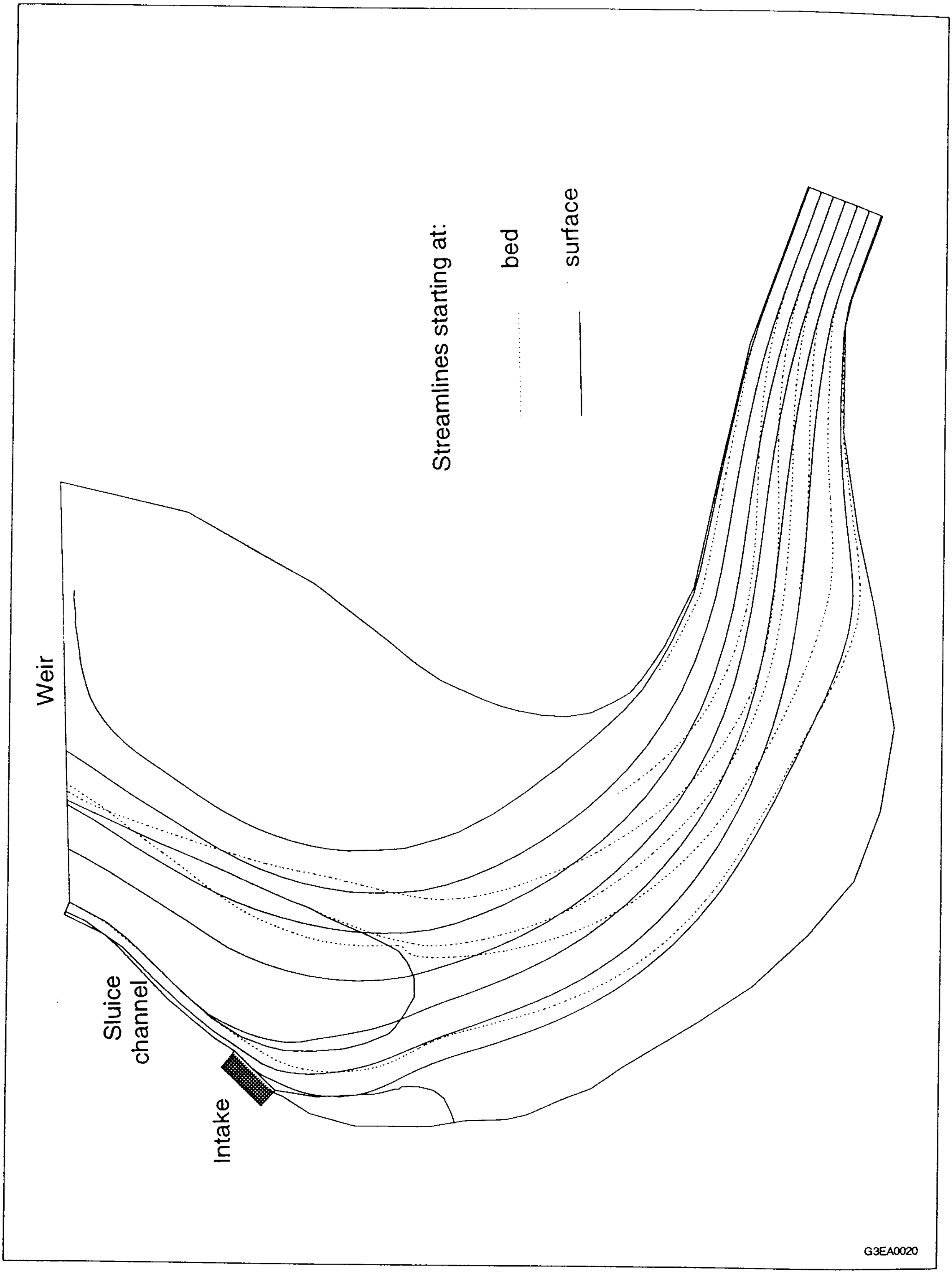


Figure 16 Predicted streamlines, Kapunga

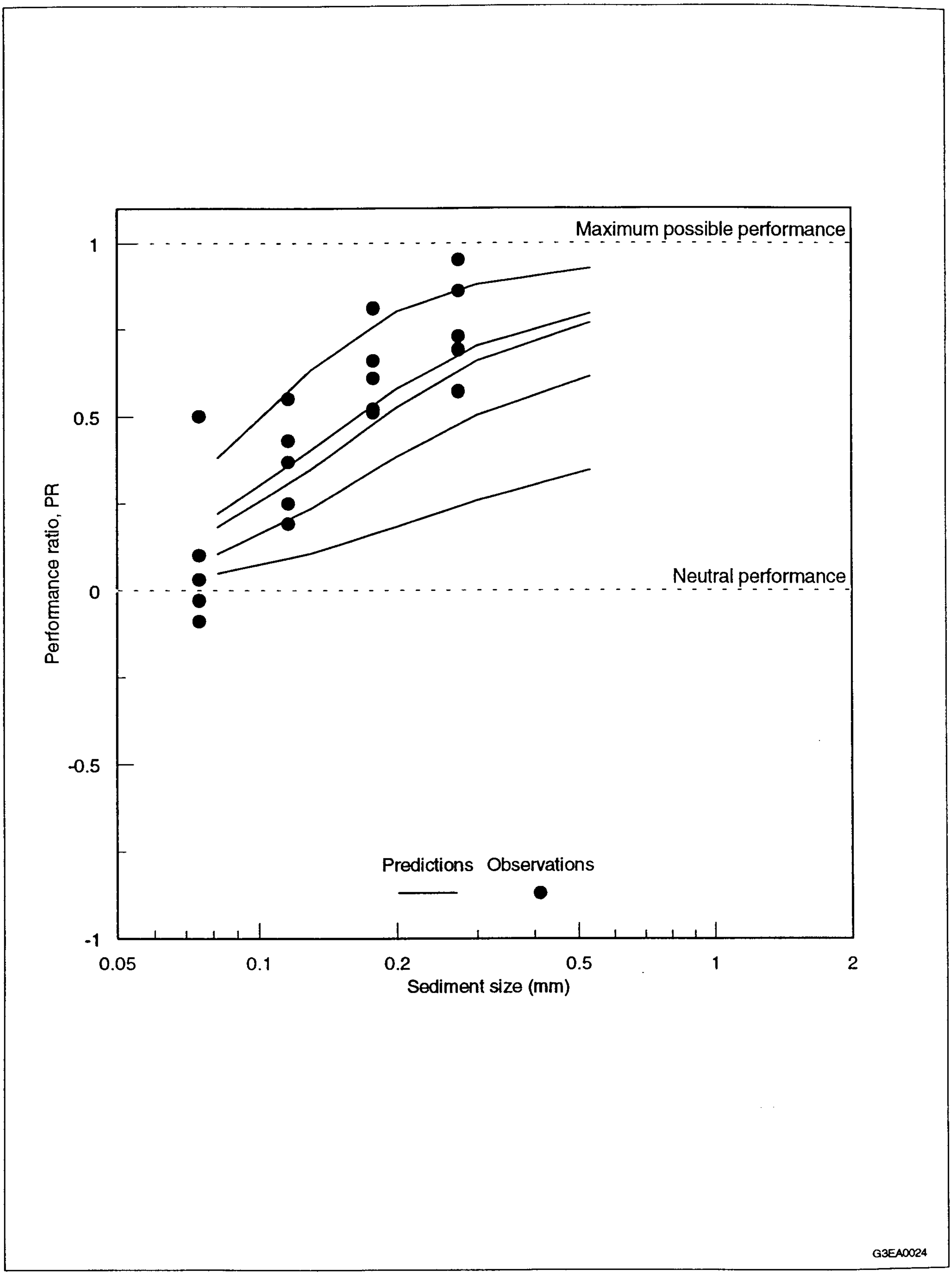


Figure 17 Predicted and observed performance ratio as a function of sediment size, Kapunga

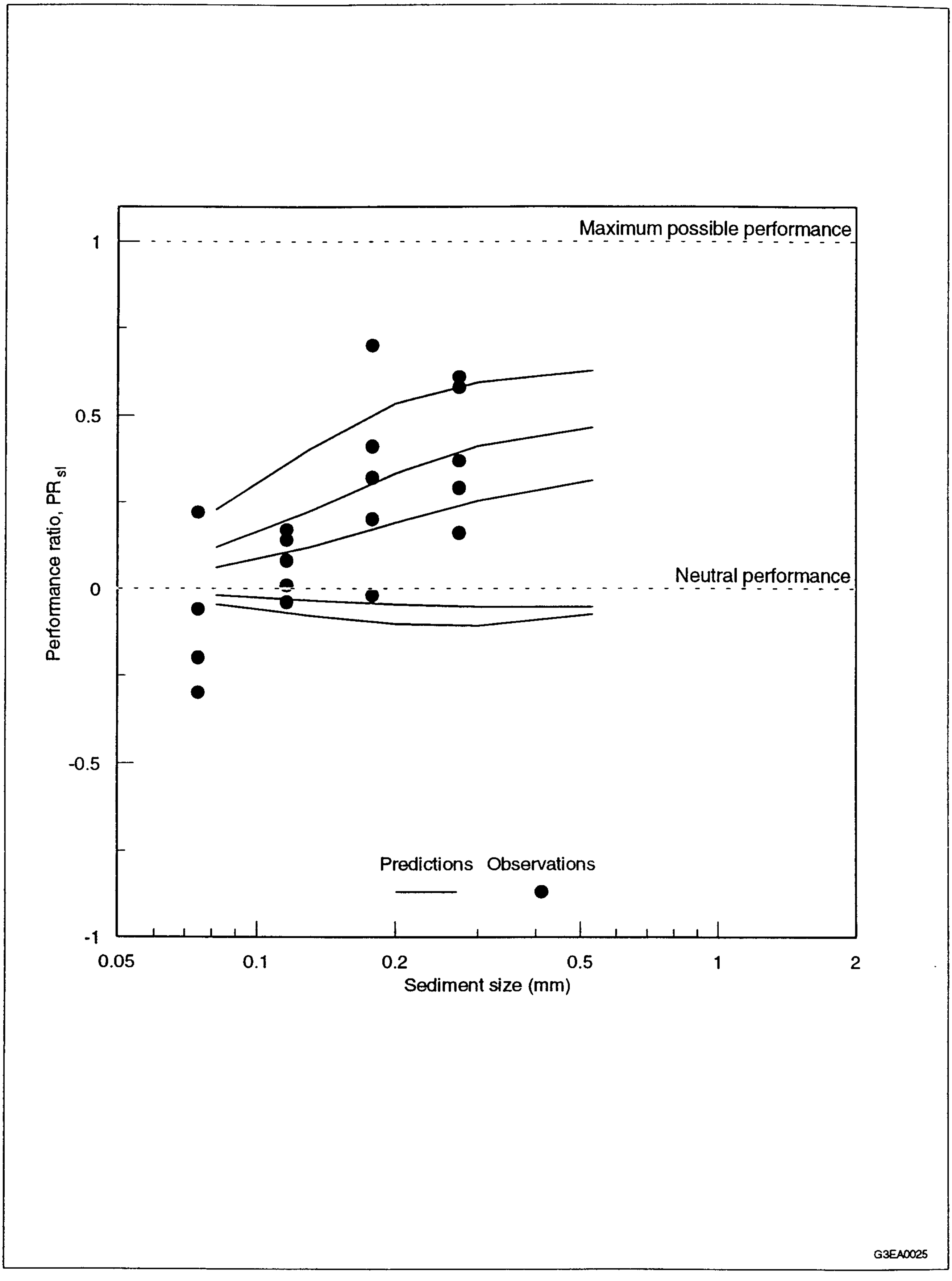


Figure 18 Predicted and observed performance ratio of sluice channel as a function of sediment size, Kapunga

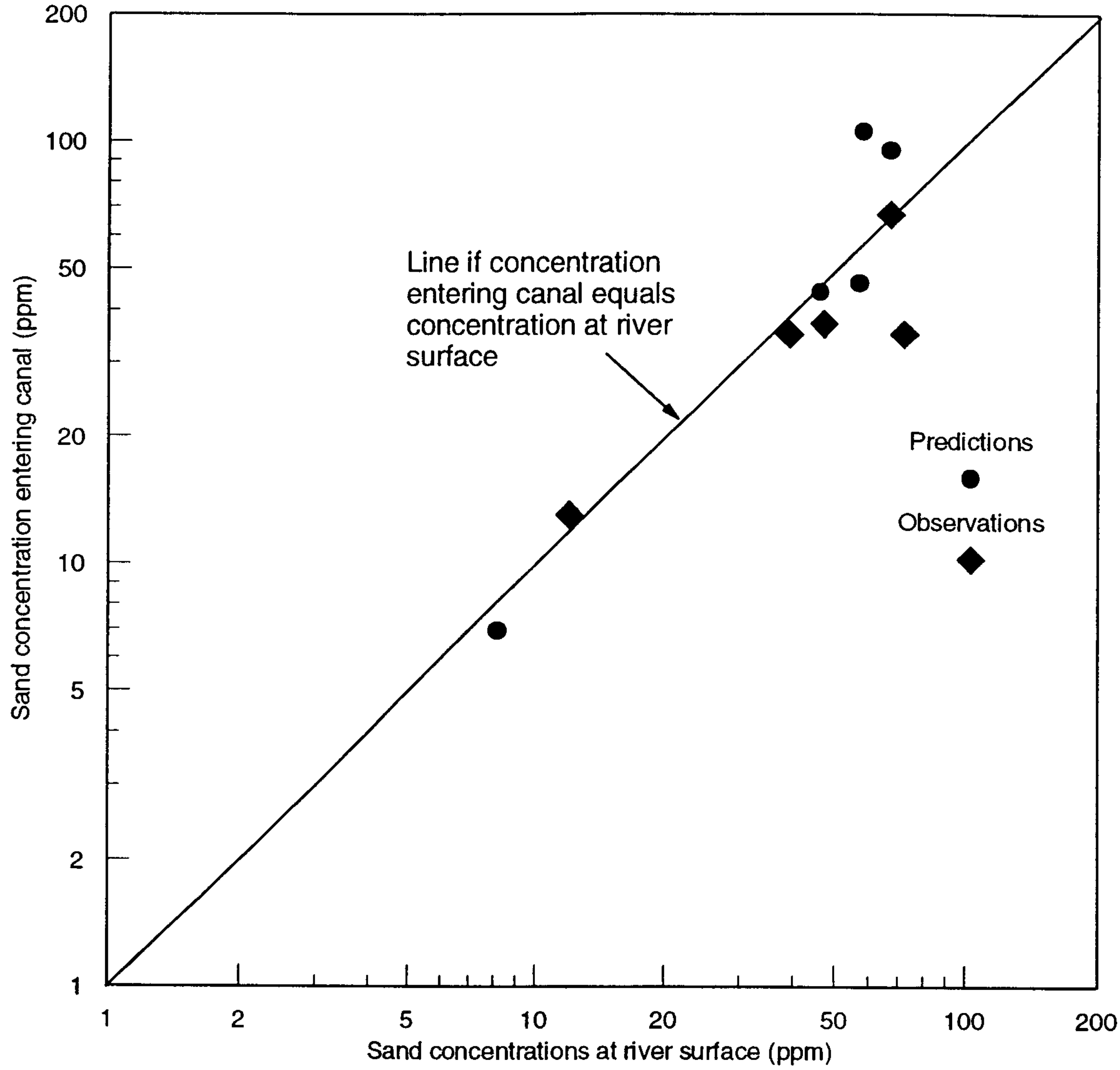


Figure 19 Comparisons between sand concentrations at river surface and entering the canal, Kapunga

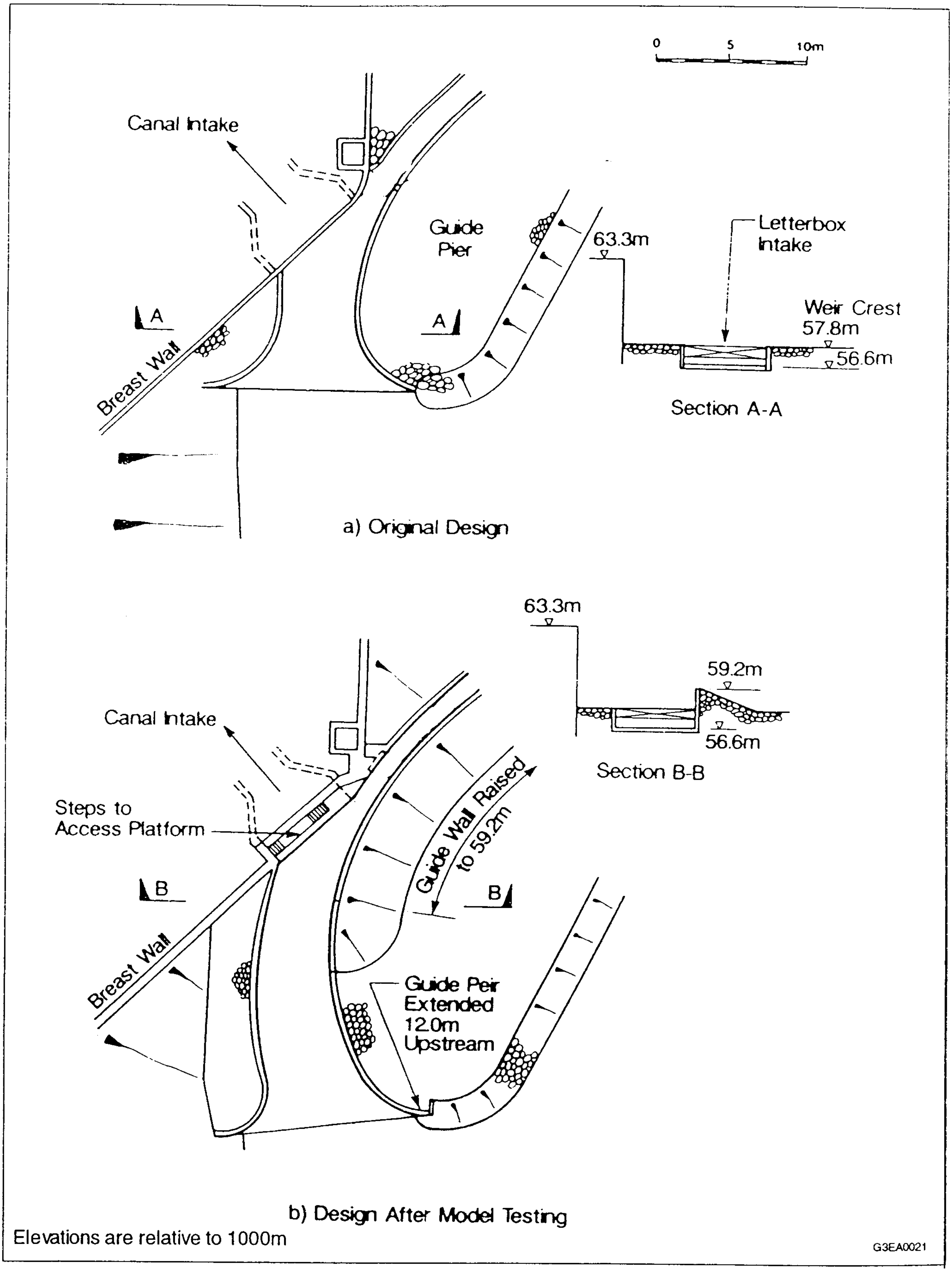


Figure 20 Changes to the design of the Kapunga intake resulting from physical model testing

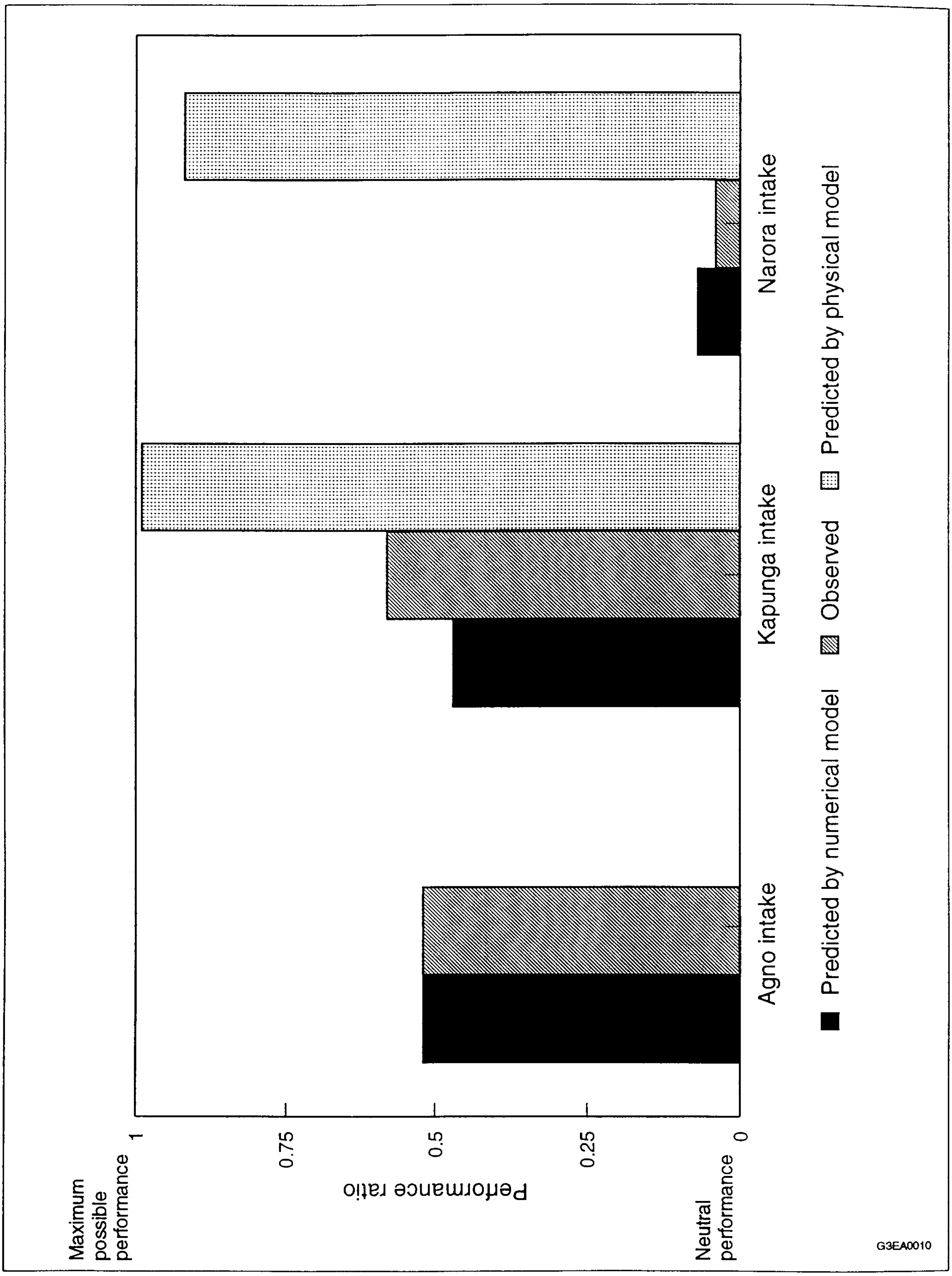


Figure 21 Predicted and observed performance at three intakes

Appendices

Appendix 1

Bed boundary conditions for sediment

Appendix 1 Bed boundary conditions for sediment

(A1) Bed boundary condition with a single sediment size

Initially the case with only a single sediment size is considered. The rate of sediment entrainment from the bed will then be a function of the local bed shear stress, bed roughness and the uniform sediment characteristics.

Garcia and Parker (1991) reviewed the prediction methods for sediment entrainment at the bed of an alluvial channel. Most relations were broadly of the form:

$$E = k\,\tau^{a} \qquad \text{(A1)}$$

where E is the entrainment rate
τ is a bed shear stress, and
k and a are constants.

For the more accurate of the methods reported by Garcia and Parker (1991) values of "a" were in the range of 1.0 to 2.5.

The value of k will be related to the bed material size, the specific gravity of the sediment grains and the bed roughness. A constant value for k is assumed to apply over the model domain when a single sediment size fraction is taken.

A value of k is determined by calibration. A representative set of conditions (usually those at the upstream boundary) are simulated by the model with equation (A1) used to set the entrainment rate. The constant k is then set so that the mean concentration predicted by the model is equal to the observed mean concentration at those conditions. When no data are available, then a mean concentration predicted by a sediment transport prediction method, such as the Engelund and Hansen (1967) or the Ackers and White (1973) methods, is used.

(A2) Bed boundary condition with many sediment sizes

When many sediment sizes are present the effect of p_{bj}, the proportion of the bed material consisting size fraction j, is included.

The method of Garcia and Parker (1991) for determining entrainment rate for each of a range of sediment size fractions has been used. No other method which could be directly applied in the model was found in the literature. Garcia and Parker's method was developed using data from two reaches of the Rio Grande (with sediment sizes ranging from 0.063mm to 0.5mm), and was tested with data from the Niobrara river (similar grain size distribution). The mean value of the ratio of predicted to observed entrainment rate was approximately 1.7, and scatter was significant (values of the ratio ranged from 0.3 to 25).

In view of the limited data to support the method, and the relatively poor comparison with field data, calibration is used to adjust an empirical constant in the prediction of entrainment rate. Calibration is undertaken in the same manner as the calibration of constant k in Section A1.

(A3) An alternative bed boundary condition based on van Rijn (1984)
The exponents on shear stress and grain size in the Garcia and Parker (1991) bed entrainment function are relatively high when compared with the equivalent exponents in the other methods reviewed. In view of these features of the bed boundary condition used by Garcia and Parker, and its limited comparison with field data, an alternative method has been developed so that results derived by each method can be compared.

The van Rijn (1984) method for predicting reference concentrations at the bed forms the basis of the alternative method. The following assumptions have been taken to apply the van Rijn method as a bed boundary condition for a sediment mixture:

- the transport rate for a size fraction is proportional to p_{bj}, (Einstein (1950) uses this assumption in his derivation of a sediment transport prediction method),

- reference height is constant for all size fractions,

- where the van Rijn method uses a median bed material size, the median size for the relevant size fraction can be used,

- hiding effects only directly influence the shear threshold for sediment motion, u_{*cr}, and

- the method of White and Day (1982) can be used to describe the effect of hiding on u_{*cr}.

Calibration of the method for each field site or flow condition is performed in the same manner as for Garcia and Parker's method.

(A4) Determination of p_{bj} values
Both of the methods presented in Section A2 and in Section A3 require values of p_{bj} for the prediction of grain sizes at each bed cell. The grain size distribution in the bed material, which defines the p_{bj} values, can not be assumed to be the same for every bed cell. For example, at river bends the bed material is coarser near the convex bank. A prediction technique for p_{bj} is therefore required.

The prediction of p_{bj} is made from the relative deposition rates of the sediment fractions at each bed cell:

$$p_{bj} = \frac{dr_j}{tdr} 4 \qquad \text{(A2)}$$

where dr_j is deposition rate for fraction j, and
tdr is total deposition rate for all fractions.

Thus p_{bj} is both a function of deposition rate and a parameter directly affecting deposition. Therefore an iterative procedure is required to determine p_{bj}: initial estimates are taken and deposition rates, dr_j, are predicted, these are then used to derive a new set of p_{bj} values at each bed cell, which in turn yield new dr_j values. This solution procedure was incorporated within the overall iterative structure of the CFD code.

When scour occurs, values of p_{bj} depend on the composition of the material being eroded and therefore on the previous sedimentation history. Prediction of such effects would be difficult. Armouring would also need to be included in the prediction of p_{bj} at many sites.

A simpler approach, which has been adopted, is to assume that armouring prevents scour. This assumption is unlikely to cause significantly incorrect predictions of intake performance, because conditions of scour in rivers are usually only associated with large floods when the intake gates to an irrigation scheme would normally be closed.

(A5) Bed material size distribution

At the end of a model simulation a set of p_{bj} values are predicted for each bed cell. The bed material size distributions are calculated from these p_{bj} values, so that the D_{50} and D_{90} bed material sizes are predicted over the river reach modelled. Other sizes can be calculated if required.

When scour is being prevented in a particular bed cell, then values for p_{bj} are not produced by the computations. In these circumstances, the values of p_{bj} at the bed cell are indirectly set from its predicted concentrations for each sediment fraction.

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