

Morphological effects of river improvement works:

Case studies

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Summary

River improvement works can change the capacity of the channel to transport sediment causing areas of deposition and erosion. Morphological effects are often not considered at the planning stage of an improvement scheme, and it is only when the scheme is operational that problems become apparent and may cause large maintenance commitments and costs.

In total, six case studies have been carried out, to assess the morphological effect of river works using a morphological model. All the schemes have involved some improvement works to channel or structures and the morphological model was used to represent the selected stretches of river both for pre and post improvement schemes and to identify the possible causes of deposition and/or erosion problems.

From a study on an idealised channel general conclusions regarding the effects of deepening and widening on a river channels have been made.

Recommendations are made for methods of minimising morphological problems at the design stage.

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1 Introduction

River improvement schemes can change the capacity of the channel to transport sediment causing areas of deposition and erosion. Morphological effects are often not considered at the planning stage of an improvement scheme, and it is only when the scheme is operational that problems become apparent and may cause large maintenance commitments and costs.

A preliminary assessment into morphological effects of river works and a review of current practices was carried out. The results of that study can be found in HR Wallingford (1987) and includes detailed summaries of schemes throughout the UK which were identified by Water Authorities as schemes where morphological problems due to river works have been experienced. A number of these schemes were selected for a more detailed study. These schemes are:

- East Mill improvement scheme on River Colne.
- Duffield scheme on River Ecclesbourne.
- River Stour improvement, Bures to Cornard.
- River Usk, Brecon Improvement scheme.
- Aylesford stream, Ashford, Kent.
- River Sence, Leicestershire.

and they all show problems of erosion and/or deposition.

This report gives details of six case studies carried out to assess the morphological effect of river works using a morphological model. All the schemes have involved some improvement works and the morphological model was used to represent the selected stretches of river both pre and post improvement scheme and identify the possible causes of deposition and/or erosion problems.

The commitment to maintenance required to maintain flood defence standards is costly and time consuming in these situations. The case studies identify alternative solutions to the constructed improvement schemes which may have reduced the deposition and/or erosion problems.

Each case study is summarised in Chapter 4 and a more detailed description can be found in Appendices C-H. Chapter 5 gives alternative solutions which could have reduced some of the morphological problems in the case studies.

A further study on an idealised channel situation was performed and the results given provide more insight into the relative benefits of different types of river improvements. The idealised channel study is summarised in Chapter 3 and a more detailed description of the study undertaken is given in Appendix B.

The overall aim of the study was to produce guidelines for engineers to use so that they can take account of morphological effects at the design stage of river works. By carrying out case studies, the morphological effects of different types of improvement works (eg new structures, widening and deepening of channels), are assessed and suggested methods of alleviating the problems or avoiding the problems in the first instance are recommended.

2 Description of morphological model

The morphological model used in the case studies is a FORTRAN language program which predicts long-term changes in river bed levels caused by engineering works. The details of the model can be found in Bettess and White (1981).

2.1 Sediment transport calculations

Within a river channel, sediment transport is determined by the characteristics of the sediment and the hydraulics of flow. To model sediment transport, therefore, the details of the flow must first be determined. The nature of flow depends on the discharge in the channel and the water level at the downstream limit of the simulated reach; both of these have to be given to enable the calculations to be performed. The geometry of the channel is specified by a number of cross-sections.

From the calculated depths, velocities and slopes it is possible to calculate the sediment concentrations at each section. The overall quantity of sediment entering the modelled reach must be specified as a boundary condition.

The sediment is divided into two size ranges in order to calculate sediment transport since the behaviour of sediments depends partially upon sediment size. For sands and larger sediments ($D > 0.06\text{mm}$) the movement of sediment depends only on the local hydraulic conditions.

For sediment smaller than 0.06mm the sediment transport depends on both the local hydraulic conditions and the sediment supply.

The transport of the coarser sand material is calculated using the Ackers and White sediment transport theory (1973) and the updated theory HR Wallingford (1990). At the upstream boundary three different sand boundary conditions are permitted:

- No sand inflow.
- Sand inflow determined by the friction slope and the characteristics of the sediment.
- Sand inflow at a specified concentration.

For the finer silt fractions some of the material settles out and is dependent on the fall velocity which in turn is dependent on the sediment diameter and is variable with concentration and with flow conditions.

Once the transport rates for each size range have been determined these are added together to obtain the total sediment transport rate at each cross-section. A sediment continuity equation is then applied to determine the change in bed level at each section due to the variations in sediment transport rate along the reach.

The morphological model incorporates the ability to restart a simulation. If this ability is to be utilised the cross section geometries at the end of the previous run of the experiment are required.

2.2 Flow calculations

The model is one-dimensional, that is only variations along the length of the channel are considered and all the quantities calculated are averaged over the cross-sectional area. No account can be taken of variations across the width or through the depth.

The package can take account of any or all of the following features of flow in a natural or artificial watercourse:

- irregular cross-section geometry with river channel meandering in its flood plain;
- afflux at bridges;
- flow through sluices and syphons, and over weirs;
- multiple steady discharges into the channel at the upstream boundary, to/from tributaries along the river and the drainage from the land adjacent to the river;
- flow through bypasses;
- multiple reaches.

The flow calculations are based on the conservation of mass and Newton's principle of conservation of momentum. The application of these principles to open channel flow was first made by the French mathematician de Saint Venant in the nineteenth century. The flow equations used in the model are similar to those of Saint Venant.

2.3 Structure of model

The model consists of a single program which divides into four modules.

- 1 Cross-section analysis; set up a database of parameters which describe the hydraulic properties of the river channel.
- 2 Backwater calculation; calculates the hydraulic characteristics of the river for a given downstream level and discharge value.
- 3 Sediment transport; calculates the variation in sediment concentration along the river channel and the total sediment volume transported between specified points during a time step. A mass continuity equation is applied to determine the quantity of sediment eroded or deposited at each section.
- 4 Cross-section shape update; calculates new cross-section profiles determined from the total quantity of sediment deposited or eroded during a period of time. Distribution of deposition or erosion is specified as a function of the depth of water across the width of the section.

The model requires information on the topography of the river and its flood plain, sufficient information to determine the flow conditions and details of sediment data. A list of data requirements is given in Appendix A.

3 *Morphological changes in an idealised channel*

3.1 Description

A better understanding of the impact of flood schemes may be gained by examining the morphological effects of a simplified flood scheme. This part of the project examines the impact of several hypothetical improvement works where the capacity of the channel would be increased to alleviate flooding.

The study is described in greater detail in Appendix B. A straight trapezoidal channel of length 20 km was represented by 41 cross-sections spaced 500m apart. The original river channel had a width of 10 m and a depth of 2 m with two floodplains sloping upwards from the bank tops at a gradient of 1:50. The longitudinal slope of the channel was 0.001. The channel was in regime. The original design discharge for the channel was 31 m³/s and the bed of the river was represented by sand of 1mm diameter.

The initial model tests were carried out under the conditions given above. The channel was then changed by either widening or deepening or a combination of both in a 5km stretch of the river 10 km from the upstream boundary. The channel was made larger to contain a flood of twice, three times and four times the original discharge of 31 m³/s. The effects on volumes of deposition, rise in water levels and bed levels within the enlarged section and within upstream and downstream sections were considered.

3.2 Results

Of the five schemes investigated, scheme 1 involved simply widening the channel, scheme 5 involved simply deepening the channel with schemes 2 to 4 being a combination of deepening and widening. Scheme 5 showed the largest amount of deposition within the improved section of the channel and scheme 1 the least amount of deposition. These trends were the same for improvement schemes designed to carry 2, 3 and 4 times the original discharge.

There were some interesting effects demonstrated upstream and downstream of the improved reach of the channel. For the schemes where widening was the dominant improvement, schemes 1 and 2, deposition was shown in the sections upstream of the improved part of the channel. The water level rise for the often large rise in minimum bed level was relatively small. The schemes where more deepening work occurred, schemes 3,4 and 5, showed net erosion upstream of the improved reach. Downstream of the improved reach erosion occurred for schemes where widening was the main improvement, schemes 1 and 2. This erosion had no effect on water levels. Deepening of the channel had no morphological impacts downstream of the improved reach.

3.3 Implications

From the morphological model study for the idealised channel we can draw a number of conclusions regarding likely morphological impact of widening and deepening.

- The deepening of a river channel is likely to cause greater deposition than widening the channel to carry the same discharge. A larger width/depth ratio would cause less deposition.

- A large width/depth ratio can cause more deposition upstream of the improved reach as the river makes an attempt to re-adjust after the changes made.
- Erosion is more likely to occur downstream of the improved reach if the width/depth ratio within the improved reach is large.
- Although the results of the study suggest that widening would be favoured over deepening to prevent deposition within the improved reach of the channel, local constraints and the effects upstream and downstream need to be considered carefully.
- Deepening is likely to cause more bank stability problems than widening.

These conclusions can only be qualitative as it is unlikely that a natural river will be a uniform channel. River improvements often involve works in addition to deepening and widening which may compound the morphological problems. In order to determine a more detailed picture it would be advisable to carry out more studies using a morphological model similar to the one used for the idealised channel study.

4 Morphological changes for site specific studies

Six case studies have been carried out on schemes which were identified as having morphological problems after river works were implemented, HR Wallingford, (1987). The details of each case study are given in Appendices C to H. The problems found for each scheme are briefly summarised in Table 1.

4.1 Scheme descriptions

East Mill improvement scheme, River Colne, Colchester

The scheme was originally designed for the provision of a water supply and the river was impounded by sluice gates and the channel widened upstream of the sluice gates. Flood embankments were constructed to protect a nearby residential area. At the design stage sediment problems were not considered. Two years after the scheme was implemented sediment deposition in the areas where the channel had been widened and deepened, upstream of the sluice gates, became apparent. Ten years after the implementation of the scheme 10,000 m³ of silt was removed. The deposition was due to the reduced flow velocities in the river as a result of the three improvements made: deepening; widening; and installation of sluice gates.

River Stour improvement scheme, Bures to Cornard

The River Stour improvement scheme was implemented in 1970 to protect agricultural land between the villages of Bures and Cornard over a distance of 10 km. The scheme proposed to reduce water levels at Bures gate and Henny Mill and to deepen the channel to increase the retained volume available for flood water. The excavated material was used for raising embankments. In order to prevent possible erosion problems caused by the changes, two new weirs were constructed. The new weirs reduced velocities and increased water levels upstream. At high discharges water bypassing one of the weirs caused erosion. Localised erosion and accretion were observed at bends. Weed growth is visible in the channel and this is controlled by regular maintenance.

Duffield improvement scheme, River Ecclesbourne

In the mid-1970s a flood protection scheme was implemented along the River Ecclesbourne to protect Duffield against a 1 in 100 year flood event. The scheme involved removing a side weir controlling flow around a bypass channel at Mill House. The bypass channel was realigned to meet the main river channel as it left the Whitehouse complex. A flood embankment was constructed between the new channel and bypass channel to prevent flood waters from flowing from one to the other. In the central part of the scheme the river was confined to a concrete flume and the channel width upstream of the two road bridges was doubled. This widening caused sediment deposition in the region immediately upstream of the road bridges. Between 300 and 600 m³ of material is removed on a yearly basis and such maintenance is costly and time consuming.

Brecon improvement scheme, River Usk

After a flood in Brecon which caused extensive damage in 1979, an improvement scheme was designed to protect the town of Brecon from a 1 in 100 year flood of peak discharge 685 m³/s. The scheme included regrading of the river bed and widening of the channel upstream of Llanfaes bridge. The channel was also deepened in some places and the invert of Llanfaes bridge was lowered by 0.75 m in order to achieve the required hydraulic gradient upstream and downstream of the bridge. These changes caused deposition of gravel in the reach upstream of Llanfaes bridge. Since 1979 gravel has been removed on two occasions from the river. On each occasion 5-8,000 tonnes of gravel was removed at a cost of £40,000.

Aylesford stream improvement scheme, Ashford, Kent

In 1972 Aylesford stream flooded which prompted the design of a flood prevention scheme. The catchment is mainly agricultural and woodland although development of Ashford since the mid-70's has increased the urban area. The scheme consisted mainly of widening and deepening the channel and the removal or modification of bridges. The river flooded again in November 1986 and after this event remedial works were carried out to remove significant quantities of sediment from the channel. It is thought that the increased bed levels resulting from deposition of sediment and the continued urban development in the post-scheme period (1974-1986) were partly responsible for the flooding in 1986.

River Sence improvement scheme, Leicestershire

As a result of flooding problems on the agricultural land adjacent to the River Sence, a river improvement programme was undertaken in 1973 for the reach of river between Kilby bridge and the bridge adjacent to the mill downstream. The improvements involved deepening, widening and straightening of the channel. In 1990 maintenance work was carried out to remove a depth of approximately 0.5m of sediment from the bed across the whole reach. During the period 1973 to 1990, there was considerable bank instability and vegetation had grown on the slumped banks. These well established terraces were left intact during the maintenance work.

4.2 Problems and implications

Each of these improvement schemes was designed to improve the discharge capacity of the channel to provide protection against floods for the neighbouring town or agricultural land. In most of the cases there had been deposition e.g East Mill, Duffield, Brecon, Aylesford Stream and River Sence. This deposition had caused the capacity of the channel to be reduced and meant that the scheme would no longer prevent flooding at the design condition. The implications of the deposition were costly and time consuming if regular maintenance works have to be performed as at Brecon. If maintenance work is not carried out the efficiency of the designed scheme will be greatly reduced.

Erosion problems were experienced on three of the schemes, Duffield, Bures to Cornard and River Sence schemes. The stability of banks was a major erosion feature as well as the erosion of the bed particularly at weir sites. The possibility of erosion must be considered at the design stage and steps taken to stabilise banks by using vegetation, grass, bushes, trees or man-made protection as detailed in Lewis and Williams (1984) or CIRIA (1990). Hemphill and Bramley (1989) explain the various erosion processes which affect the integrity of unprotected banks and gives guidance on methods of protecting the banks of rivers, canals and drains against natural and man-made causes of erosion.

For each of the six studies described above alternative methods were considered which would reduce the morphological problems experienced. Often these alternatives would involve major changes to an existing scheme which for economic reasons would not be viable. The alternatives are given in the Table 2 and discussed in more detail in Section 5. They may prove a valuable reference for designers when planning new improvement schemes with similar features to those described in the six case studies.

5 *Alternative solutions for minimising morphological problems*

For each case study detailed in Chapter 4 and Appendices C to H some alternative solutions were recommended which are shown in Table 2. Details are given below of some of these alternative solutions.

Where widening and/or deepening is or could be the cause of deposition and/or erosion, different options can be suggested to increase the capacity of the channel. One option would be to design a two-stage channel where the low-flow channel remains as existing and the berm between the river channel and flood banks are widened and lowered, as suggested for the Duffield improvement scheme. The advantages of this solution are that the original channel remains the same whilst the flood capacity of the channel is increased with extra capacity provided on the flood berms. There are some problems with two-stage channels which provide extra capacity on a lowered flood berm. During a flood sediment can be deposited on the berm especially if the berm is heavily vegetated.

The construction of a two-stage channel can be complimented by constructing flood embankments. Often the material taken from lowering the flood berm can be used in the embankment construction. If the embankments are made up from soil an extra freeboard should be allowed as the embankments may settle. For concrete or man-made flood walls or embankments the extra freeboard is not required although this type of protection is not so environmentally acceptable.

Calculating accurately the capacity of two-stage channels is a subject under discussion and research at the moment. Often the capacity at the design stage is overestimated due to the complex interaction of flow at the boundary of channel and floodplain and an incorrect value taken for the roughness of the floodplain. It is recommended that specialist advice is taken on the design of two-stage channels.

Another alternative to widening and deepening of a channel may be to provide a flood relief channel. This is dependent on land availability and would involve the digging of a new channel and construction of new structures. This alternative was not a viable option for any of the case studies due to land constraints. The advantages of relief channel over improvements to the original channel can only be determined by a site specific morphological study as it would be dependent on the sediment size, river type and the operation of the relief channel. It is however an option which may be considered for relieving morphological problems.

Due to land constraints, widening and deepening may be the only viable option of increasing channel capacity. From our research on an idealised channel, see Chapter 3, it appears that to try and minimise the volume of deposition it is preferable to make the width/depth ratio as large as possible i.e to widen the channel in preference to deepening the channel. An additional factor to consider is that deepening is more likely to cause bank stability problems than widening.

In a number of the case studies; East Mill; Duffield; and Brecon improvement schemes part of the alternative solution recommended was to narrow the river to pre-improvement scheme widths thereby leaving the river channel 'in regime'. This suggestion can only form part of an alternative solution and embankments or a two-stage channel or a regular maintenance program recommended for the River Sence need to be considered in addition to retaining the original channel cross-sections.

The morphological problems encountered on a scheme may be caused by particular parts of the scheme. If changes are made to these sections the problems may be greatly reduced. This was demonstrated in the Brecon improvement scheme on the River Usk when results from our morphological model study demonstrated that changes made to the river cross-sections immediately upstream and downstream of Llanfaes bridge could have some effect on the morphological problems encountered. By making changes to a few cross-sections the morphological problems of a scheme could be minimised. The cross-sections which need changing may be identified by calculating the sediment concentration and transport rates. If the sediment transport rates and concentrations are high then morphological problems are likely to occur.

A morphological model study can identify in more detail the areas of the scheme where deposition will occur. Cross-sections can be easily changed in the model to determine the design which minimising sediment deposition and/or erosion whilst giving acceptable flood levels. This was demonstrated in the River Stour, Brecon, East Mill and Aylesford stream case studies.

Sediment traps can be usefully incorporated in a scheme. They can be positioned at the upstream end of a scheme and restrict the volume of sediment entering the scheme by trapping the sediment. The trap is usually an area within the river which is wider and deeper than the normal cross-section. The velocities are reduced within the area of the trap and cause the sediment to settle in the trap. A maintenance commitment is needed to remove the sediment from the

trap at regular intervals to maintain the efficiency of the trap. The advantages are that the sediment will mostly collect in one place and if the trap operates as designed the cost of dredging the trap can be determined at the design stage.

Sometimes the only options available to the designer will involve improvement works which may have adverse morphological effects. In that situation it will be useful to know the maintenance commitment which will result from deposition and/or erosion. The volume and position of sediment deposition can most accurately be determined from a numerical, morphological model study. Details of recommended procedures for determining morphological impacts can be found in a complementary report HR Wallingford, (1992). Due to the diverse nature of rivers, sediment types, improvement works it is difficult to predict without a morphological model study the detailed effects and maintenance commitment required.

6 Conclusions

The six case studies outlined on Section 4 and detailed in Appendices C to H highlight some important aspects of the potential morphological effects of river works. The improvement works included a variety of features including channel widening and deepening, flood embankments, channel re-alignment, new structures and bridge modifications. The model studies suggested ways in which the problems of sedimentation or erosion may be alleviated as detailed in Section 5. These options included reducing the width of the channel, often where it had been widened or deepened, removal of new structures, removal or modification of bridges and introduction of a two-stage channel. These options were based on the best morphological solution and take no account of the cost of making changes to the existing scheme.

The only case study where deposition was considered at the design stage was Brecon. At Bures-Cornard, Duffield and Brecon, erosion was considered at the design stage and measures were taken to prevent bank erosion by using revetments. Bed erosion, however, was not considered. The problems experienced on each of the six case study schemes emphasise the need for engineers to consider the morphological effects early in the design stage of a scheme.

As a result of the six case studies we can make some preliminary recommendations regarding the potential morphological effects of certain aspects of river improvement works.

- 1 Widening and deepening of a channel may cause deposition of sediment especially if the sediment concentrations are high. The problem may be more severe upstream of a structure or bridge where the flow velocities are reduced.
- 2 Channel widening may cause a drop in water level and cause bank instability and erosion.
- 3 Construction of new weirs may cause downstream erosion.
- 4 New structures built to impound water cause reduced flow velocities and the potential for sediment deposition is greater.

- 5 Modifying existing bridges may cause problems of deposition or erosion if the invert of the bridge is lowered or raised.
- 6 If the open area of a bridge is reduced so as to severely restrict the flow and cause a large afflux this causes a rise in water level and deposition upstream of the bridge.
- 7 Even if deposition and erosion is accounted for in a feasibility study the actual constructed works often incorporate only a number of the design feature or different features. The effect on the morphology could be different from that predicted in the feasibility study.

From the case studies it was identified that widening and deepening of a channel may cause deposition of sediment. In an effort to determine which changes, widening or deepening, or a combination of the two, has the most detrimental effect, a model study was carried out for an idealised channel and that study suggested the following points:

- 1 Widening the channel to pass a certain discharge will cause less deposition within the improved reach than deepening the channel to provide the same discharge capacity. The problem of deposition will not be removed but reduced by increasing the width/depth ratio of any improvement made to a regime channel.
- 2 Although widening appears to be the most favourable solution for the improved part of the channel, the effects upstream and downstream of the improved channel may be more serious with a widened channel than with a deepened channel.

The value of collecting field data and modelling morphological behaviour can be seen from the six case studies carried out for this project.

Recommended procedures for determining the morphological impact of river improvement works can be found in a complementary report HR Wallingford (1992).

7 References

Ackers, P and W R White, 1973. Sediment transport: new approach and analysis. ASCE. Journal of the Hydraulics Division, 99, HY11, pp 2041-2060.

Bettess R and W R White, 1981. Mathematical simulation of sediment movement in streams. Proc ICE, Part 2, 71, pp 879-892.

Bramley and Hemphill, 1989. Protection of River and Canal Banks. CIRIA. Butterworths.

CIRIA, 1990. Use of vegetation in Civil Engineering. Butterworth.

HR Wallingford, 1987. HR Wallingford Report SR 116. Morphological effect of river works. A review of current practice.

HR Wallingford, 1990. HR Wallingford Report SR 237. Sediment Transport: The Ackers and White Theory Revised.

HR Wallingford, 1992. HR Wallingford Report SR 300. Morphological effect of river improvement work : Recommended procedures.

Lewis and Williams, 1984. Rivers and Wildlife handbook: A guide to practices which further the conservation of wildlife on rivers. RSPB and RSNC.

Tables

Table 1. Details and morphological problems of river improvement works

Scheme	River	Scheme Details	Subsequent problems	Maintenance
East Mill improvement scheme	River Colne	New sluice gates, channel enlargements, flood embankments	Silt accumulation	Silt removal
River Stour improvement scheme, Bures to Cornard	River Stour	Channel deepening, flood embankments, new weirs	Erosion at weir, weed growth	Grass cutting
Duffield Improvement scheme	River Ecclesbourne	Concrete channel, channel widening, re-alignment, bank protection, flood walls	Sediment accumulation. Erosion of revetment	Sediment removal
Brecon improvement scheme	River Usk	Channel regrading, flood embankments, bridge alterations, widening and deepening of channel upstream of Llanfaes bridge	Silt accumulation upstream of Llanfaes bridge	5,000 to 8,000 tonnes of gravel removed on two occasions since 1979
Aylesford Stream improvement scheme	Aylesford stream	Channel widening and deepening, bridges removed or reconstructed, re-alignment of channels, concrete flume section, flood embankments	Weed growth, deposition of silt and sand	Yearly weed clearance, dredging of sand and silt
River Sence improvement scheme	River Sence	Channel widening, deepening and re-alignment	Deposition of sand and silt. Vegetation growth, bank erosion	Dredging and vegetation clearing

Table 2. Alternative solutions to river improvement schemes

Scheme	River	Scheme Details	Alternative Methods	Potential problems with alternative methods
East Mill improvement scheme	River Colne	New sluice gates, channel enlargements, flood embankments	Narrow channel to half expanded width in selected reaches. Retain existing embankments	Increase in water levels due to narrower channel
River Stour improvement, Bures to Cornard	River Stour	Channel deepening, flood embankments, new weirs	1. Retain deep wide cross-sections of post improvement scheme, with pre-scheme weirs and tailwater conditions. 2. Retain deep, wide cross-sections and tailwater of post-improvement scheme.	No problems identified
Duffield improvement scheme	River Ecclesbourne	Concrete channel, channel widening, re-alignment, bank protection, flood walls	1. Removal of central channel wall. 2. Narrowing channel to pre-scheme width 3. Introduce two-stage channel 4. Widen channel by 2m	1. Introduction of a narrower or two-stage channel would cause an increase in water level which may be unacceptable during floods
Brecon improvement scheme	River Usk	Channel regrading, flood embankments, bridge alterations, widening and deepening of channel upstream of Llanfaes bridge	1. Changes to sections downstream of Llanfaes bridge 2. Changes to sections downstream and remove changes made upstream of Llanfaes bridge 3. Replacement of section immediately downstream of Llanfaes bridge 4. Narrowing sections upstream of Llanfaes bridge 5. Replacing Llanfaes bridge	1. Increase in water levels due to changes made to channel for modified scheme 4. 2. Modified scheme 3 would cause similar sediment deposition to original scheme. 3. Modified scheme 5 would probably be unacceptable on economic and environmental grounds
Aylesford stream improvement scheme	Aylesford stream	Channel deepening and widening, bridges removed or reconstructed, re-alignment of channel, concrete flume section, flood embankments	Retain post-improvement scheme cross-sections, remove or replace three bridges downstream of Crowbridge Road bridge	Some sediment deposition would still occur but the amount would be less than the present situation
River Sence improvement scheme	River Sence	Channel widening, deepening and re-alignment	Routine maintenance vegetation cutting and dredging to be carried out to maintain the original channel roughness and shape	Expensive solution

Appendices

Appendix A

Data requirements

The following data would be required for morphological modelling.

- 1 Information to describe the channel geometry prior to the works. This should include cross-sections and a longitudinal profile.
- 2 Details of the works, including changes in channel geometry and alignment, together with details of any new or modified structures.
- 3 Flow exceedence curve for the reach to be simulated. Ideally this should be for a location within the simulated reach but a site either upstream or downstream should be suitable provided no major tributaries enter the river between the site and the simulated reach.
- 4 Stage-discharge relationship at the downstream end of the simulated reach. If this is not available an approximate relationship can be generated based on geometry data.
- 5 Any details which may exist on channel roughness.
- 6 Stage-discharge relationship for any structures within the simulated reach.
- 7 Any details on the variation of stage with discharge at different points in the simulated reach.
- 8 Particle size distributions for bed samples at three locations within the simulated reach.
- 9 Discharge - sediment concentrations at any points in the simulated reach. If they exist.

Appendix B

Case Study: Idealised Channel

B.1 Introduction

A better understanding of the impact of flood relief schemes may be gained by examining the morphological effects of a simplified improvement scheme. This part of the project examines the impacts of several schemes for improving the capacity of an existing hypothetical river channel in order to reduce flooding.

The initial assumptions for this study are:

- a) A uniform river reach with floodplains is subject to flooding for, on average, one day per year. The reach has a length of 15km.
- b) The river channel has a width of 10m and a depth of 2m, while the two floodplains slope up away from the bank tops at a gradient of 1 (vertical) : 50 (horizontal) Figure B1.
- c) The longitudinal slope of the channel is 0.001. The reach is currently in regime - that is, the channel is in stable equilibrium with bed and water levels showing no tendency to change over a number of years.

Various options are examined for protecting the central 5km of this reach from flooding. These include increasing the depth and/or width of the channel, constructing flood embankments, and realignment of the channel to increase the channel slope. For each option, a 1-dimensional morphological model has been used to predict bed level changes resulting from the improvement schemes. It has therefore been possible to compare the performance of the schemes in terms of maintenance required to preserve their design flow capacity.

B.2 Experimental schemes

B.2.1 Uniform channel geometry

The channel was 15km in length, and was uniform throughout this length as shown in Figure B1. At a later date the channel was extended at the upstream end by 5km to ensure that the improved reach was not affected by boundary conditions. The longitudinal slope of the channel was 0.001.

The Colebrook-White roughness formula was used to calculate channel resistance. Erroneous results can arise if irregular shaped cross sections are analysed using the resistance equation. The flooded cross section was therefore divided into three parts and treated as a compound channel made up from two flood plains and a main channel. The conveyance of the whole channel cross section was then derived from the sum of the conveyances of the flood plains and main channel. The Colebrook-White roughness length for the channel and flood plains was assumed to be 0.1m throughout the reach.

B.2.2 Discharge

Discharge data was supplied in the form of a number of steady discharges, each of which was assumed to occur for a given duration in any year. This discharge sequence was repeated as necessary to simulate the required number of years.

The sequence of discharges was chosen so that, if repeated for a number of years, the following flow statistics would be generated:

- a) The flow should exceed bank-full discharge for an average of one day per year. Bank-full discharge for the channel was 31 m³/s.
- b) The mean annual flood was assumed to exceed the bankfull discharge by 50%.
- c) The growth factor relating the mean annual flood to the maximum modelled flood (return period T = 14 years) was assumed to be 2.

The flow exceedence curve used is shown in Figure B2.

B.2.3 Sediment transport

Sediment transport was calculated using the Ackers and White sediment transport function which relates sediment concentration to the sediment size and properties of the flow. The bed of the river was represented by a uniform sediment of 1.0mm.

B.2.4 Downstream boundary condition

The water level at the downstream model boundary of the model was calculated at each discharge, based on normal flow depth in the channel.

B.3 Experimental results

A model simulation was carried out for a 30 year time period in order to investigate morphological change in the river in the absence of flood protection schemes.

Figure B3 shows bed and water levels along the modelled channel, both at the start of the experiment and after a simulation period of 30 years. The water levels are those resulting from the maximum modelled flow of 90m³/s. The figure also shows the level of the top of the river banks. The depth of flooding over the banks is initially approximately 1m at this discharge. There is practically no long term change to bed or water levels over the 30 year period: the channel is 'in regime', or in a stable state where the sediment transporting capacity throughout the reach is matched to the sediment supply.

B.4 Channel improvements; widening and deepening

This part of the study was designed to investigate the effect on sedimentation in the channel of a number of different flood protection schemes. Each scheme was designed to protect a reach of the channel from flooding by increasing the channel cross section. The schemes all consisted of widening or deepening in varying degrees, of the main channel between chainages 10km and 14.5km from the upstream boundary. In order to avoid sudden changes in cross section, a 1km transition upstream and downstream of the enlarged reach was included.

Parameters such as discharge, sediment size and flow and sediment boundary conditions were the same as used in the initial representation of the channel without flood protection.

Five schemes were investigated, ranging from widening of the channel (scheme 1), combinations of widening and deepening (schemes 2 to 4) to deepening (scheme 5). In each case, the enlarged channel was designed to prevent flooding. The criteria used to design the enlarged cross sections was that the normal depth of flow resulting from the maximum modelled discharge should correspond to the height of the river banks.

Figure B4 shows results for scheme 1, which is the case of only channel widening. The initial water level at $90\text{m}^3/\text{s}$, just after the scheme is implemented, is contained within bank level in the improved or widened area. This shows that the scheme would initially prevent flooding in the improved reach. At the downstream end, however, for a $90\text{m}^3/\text{s}$ flood, the backwater effect from the unchanged channel downstream causes water levels to be slightly higher than bank levels.

After a period of 30 years the bed level within the scheme will have risen up to a maximum of 0.83m for scheme 1 over original scheme bed level. The volume of deposition is greatest in the first ten years. The rate of deposition slows down and for the 10 year period 20 to 30 years the bed begins to erode (Table B.1).

Within the enlarged part of the channel there are differences in the volumes of deposition and subsequent rise in bed level for the five schemes. The largest volume of deposition occurs in scheme 5 where the channel is not changed in width but the extra capacity is gained from deepening the channel. The volume of deposition after 30 years for a channel capacity of $93\text{m}^3/\text{s}$ is $101,410\text{m}^3$ in comparison to $35,395\text{m}^3$ for scheme 1 which was designed for $93\text{m}^3/\text{s}$ but widened instead of deepened. The rise in bed levels in scheme 5 is 1.99m and for scheme 1 is 0.83m with corresponding rises in water levels of 1m and 0.66m. Longitudinal profiles of schemes 1-5 are given in Figures B4 to B8.

Schemes designed for twice and four times the original channel capacity were considered. Schemes 1a and 1b involved only widening and schemes 5a and 5b only deepening and there were a range of intermediate schemes 2a, 3a, 4a, 2b, 3b and 4b. The trends were again the same as designed for 3 times the original discharge, with schemes 5a and 5b, width/depth ratio (B/D) being small, showing the greatest volume of deposition, greatest rise in water bed levels within the improved part of the scheme. The results are shown in Figures B9, B10 and B11, and Tables B2 and B3.

We have demonstrated that the greatest effects are caused by the schemes where deepening is the major method of improving the channel ie when B/D is small. However it is important to determine the morphological effects which the improved scheme may have on those sections of river upstream and downstream of the improved reach.

Figure B12 shows volume of deposition for the sections upstream of the improved reach for different width/depth ratios and for different design discharge capacities. For schemes which are mostly widened there is deposition in the upstream sections. Figures B4 and B5 show this deposition begins just upstream of the improved reach. The bed level rise is relatively large as the slope of the channel adjusts. In reality the channel would erode the banks at high flows in an attempt to gain a stable relationship between width, depth, slope, sediment and water discharge. Therefore the capacity of the channel will not decrease by the amount that the high rise in bed level suggests. The water level rise is quite small, in comparison to the rise in bed level.

For schemes 3, 4 and 5 there is net erosion upstream of the improved reach (Fig B12). The principal improvement for these schemes has involved deepening the improved reach and the slope of the channel upstream is adjusted in an attempt to regain a stable condition, therefore causing erosion.

The erosion downstream of the scheme is shown in Figure B13. The worst erosion occurs when the ratio of width to depth is large ie when the improvement work involves widening. There is no effect on water levels downstream of improved reach.

B.5 Comparison of widening and deepening

The performance of the different schemes is compared in Figure B9. The figure shows the quantity of sediment deposited in the improved reach for each of the schemes over 30 years plotted against the width/depth ratio of the main channel in the improved reach. The figure gives an indication of the quantities of sediment that would need to be removed from the improved reach to maintain the original standard of flood defence. The enlarged channels with high width/depth ratios perform better in this respect than channels with low width/depth ratios. Channel widening may incur less maintenance than channel deepening. The quantities of sediment which would have to be removed for each scheme are given in Tables B1, B2 and B3.

Table B1. Volume of deposition for schemes 1-5 :
Design Discharge = $93\text{m}^3/\text{s}$

Scheme	B/D	Volume of deposition in m^3			
		0-10 yrs	10-20 yrs	20-30 yrs	Total
1	12.7	32008	14658	-11272	35395
2	9.61	36502	19976	-465	56477
3	6.83	44080	15513	13152	72746
4	4.39	57509	15636	7409	80554
5	2.38	77614	17386	6410	101410

Table B2. Volume of deposition for schemes 1a-5a :
Design Discharge = $62\text{m}^3/\text{s}$

		Volume of Deposition in m^3			
Scheme	B/D	0-10 yrs	10-20 yrs	20-30 yrs	Total
1a	9.06	18672	10948	-2624	26996
2a	7.38	21790	7623	5416	34830
3a	5.81	27310	6927	3692	37929
4a	4.39	33421	7768	2987	44176
5a	3.11	43160	8237	3324	54721

Table B3. Volume of deposition for schemes 1b-5b :
Design Discharge = 124m³/s

Scheme	B/D	Volume of deposition in m ³			
		0-10 yrs	10-20 yrs	20-30 yrs	Total
1b	17.06	43639	17106	-7771	52974
2b	12.32	49983	24118	3130	77231
3b	8.14	61730	28406	13045	103182
4b	4.62	77782	24565	12359	114706
5b	1.87	116614	25090	12632	154336

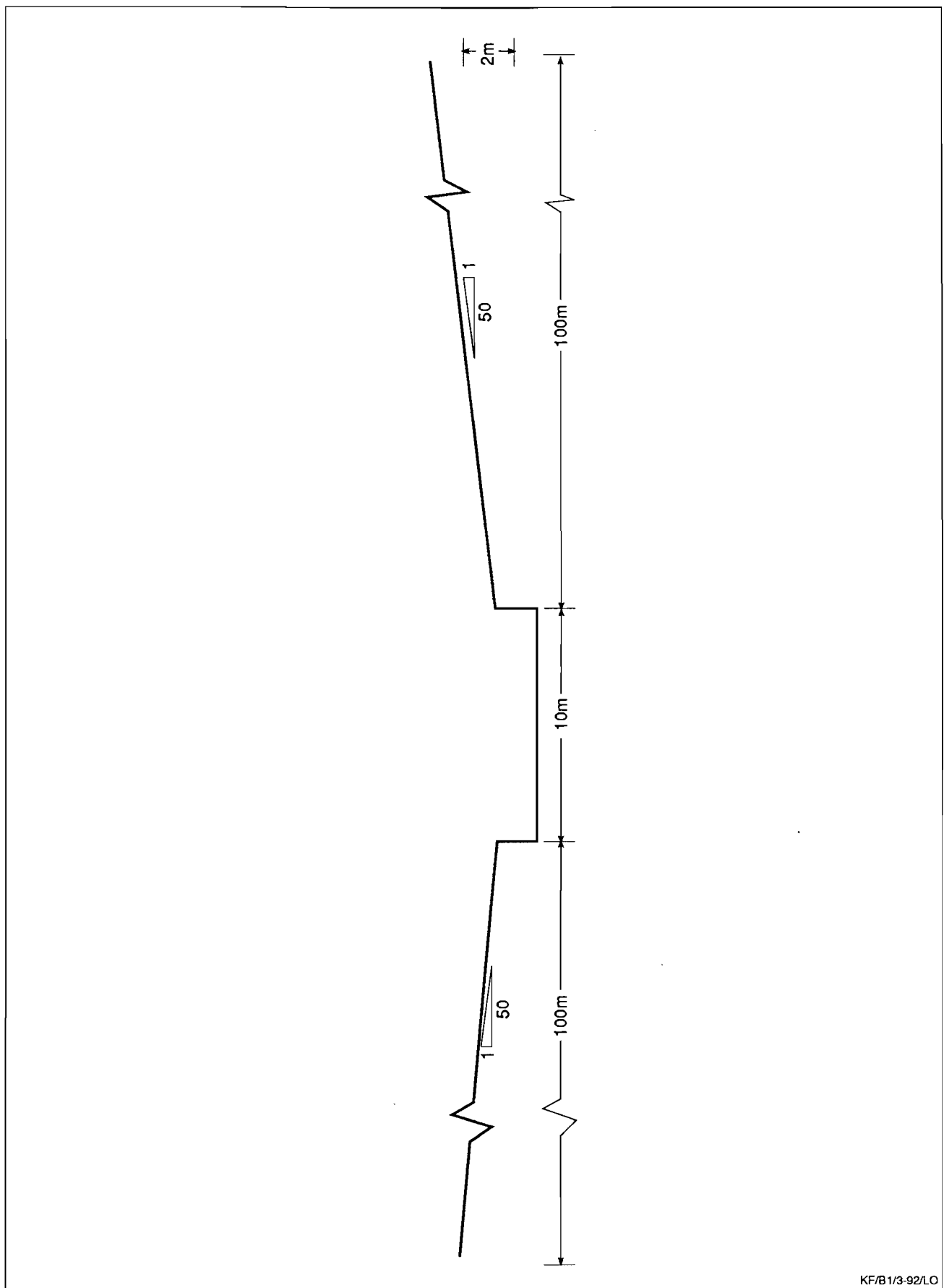


Figure B1 Typical channel cross-section – idealised channel

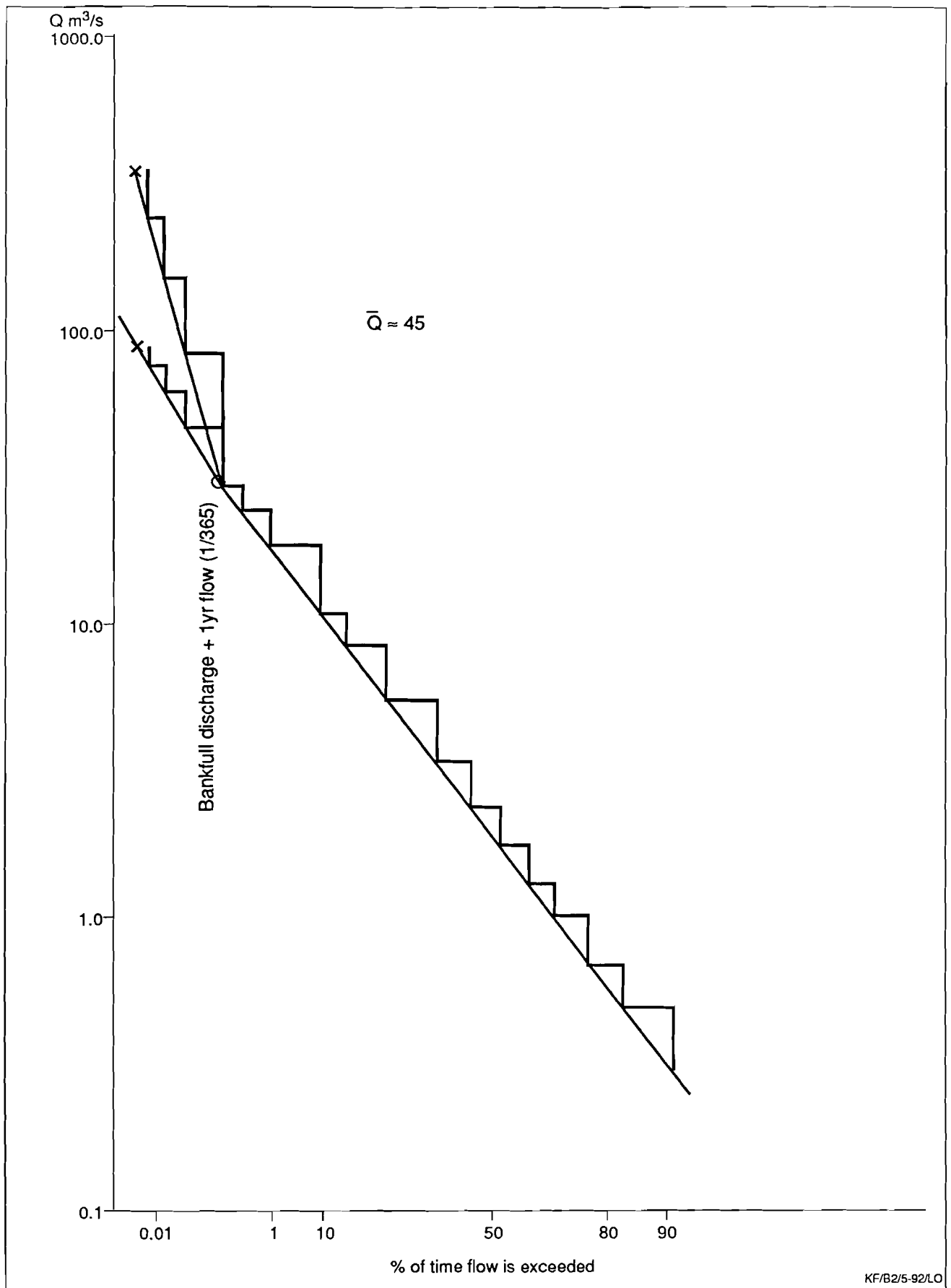
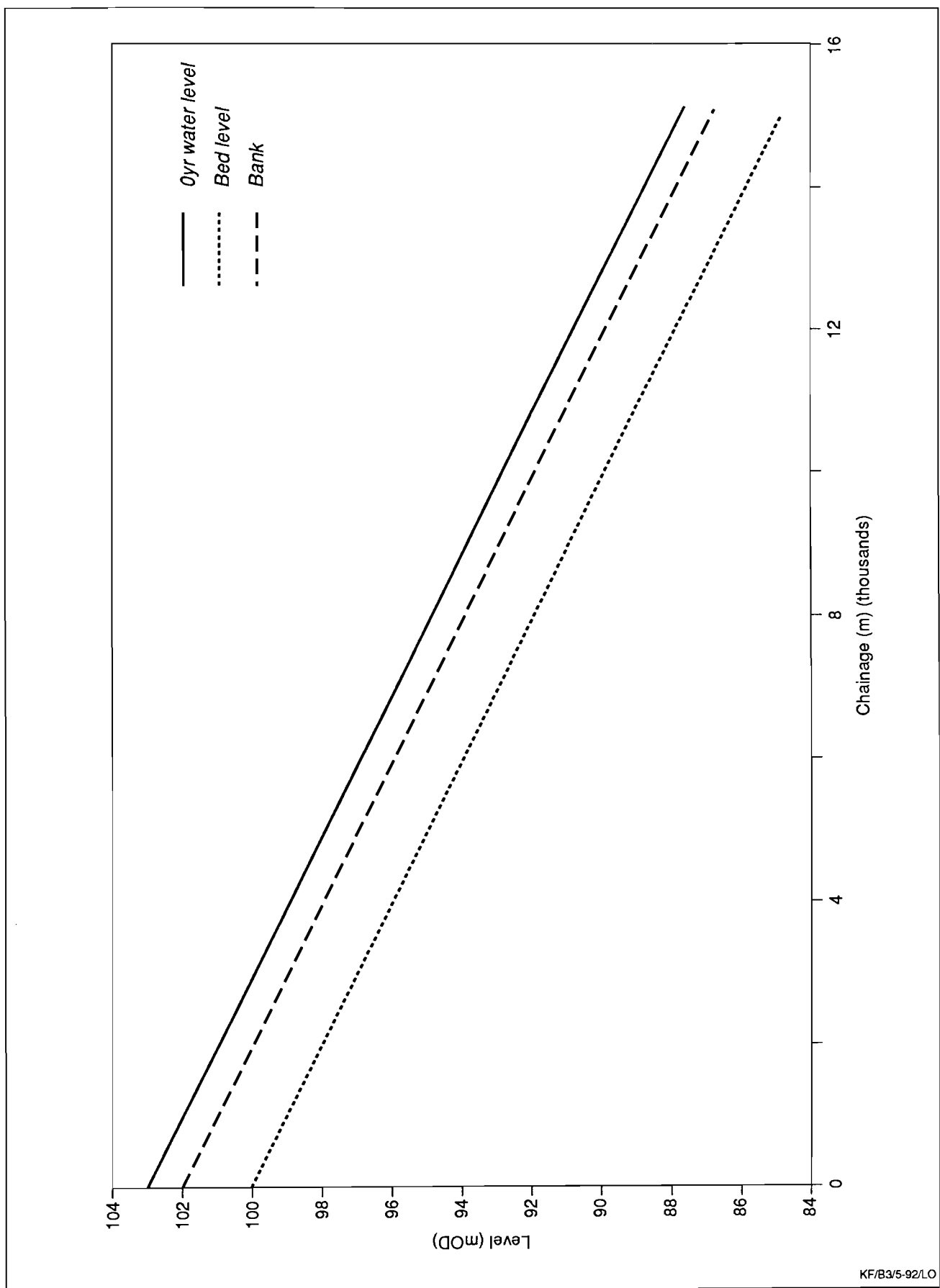
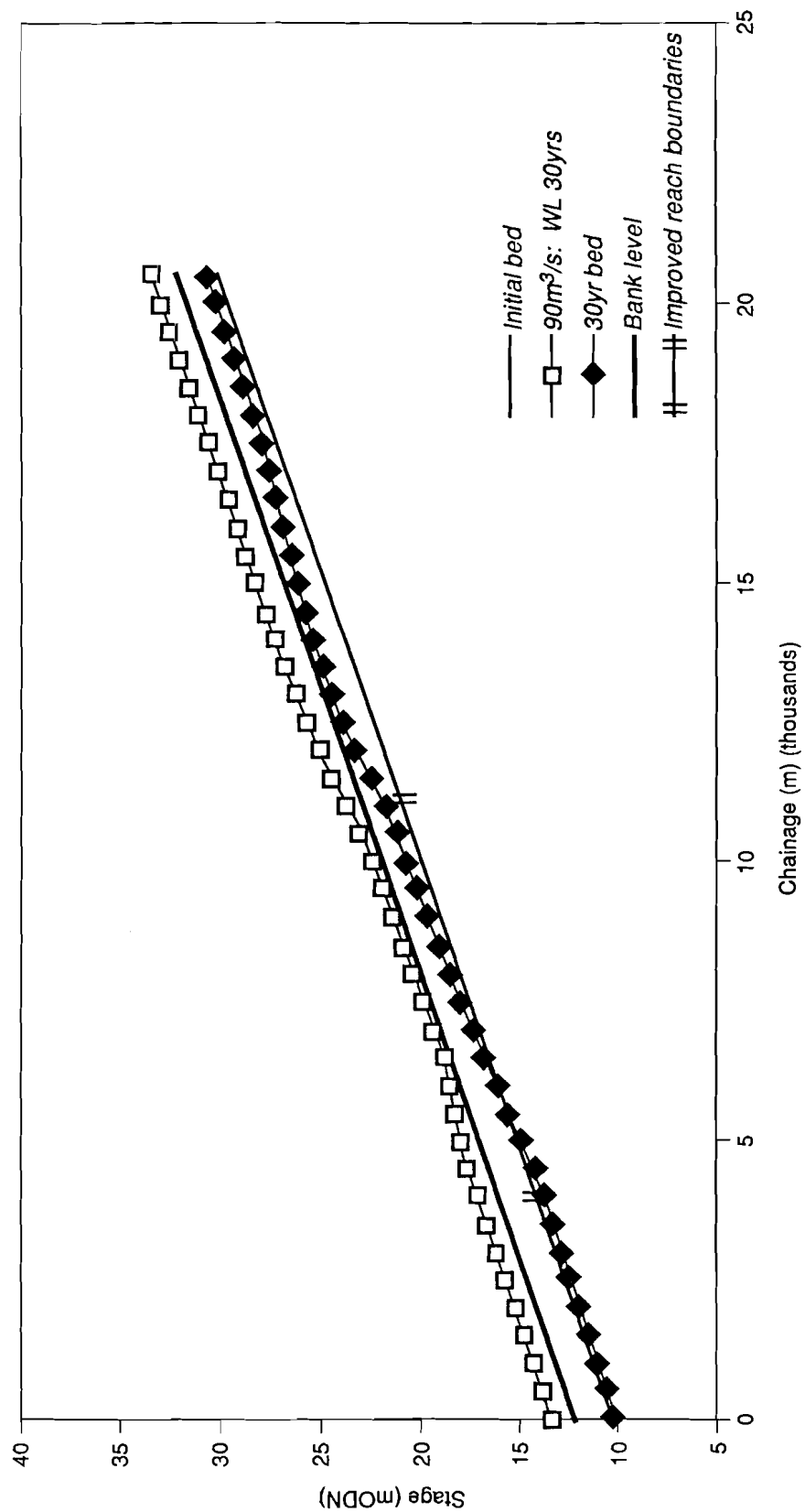


Figure B2 Flow exceedence curve



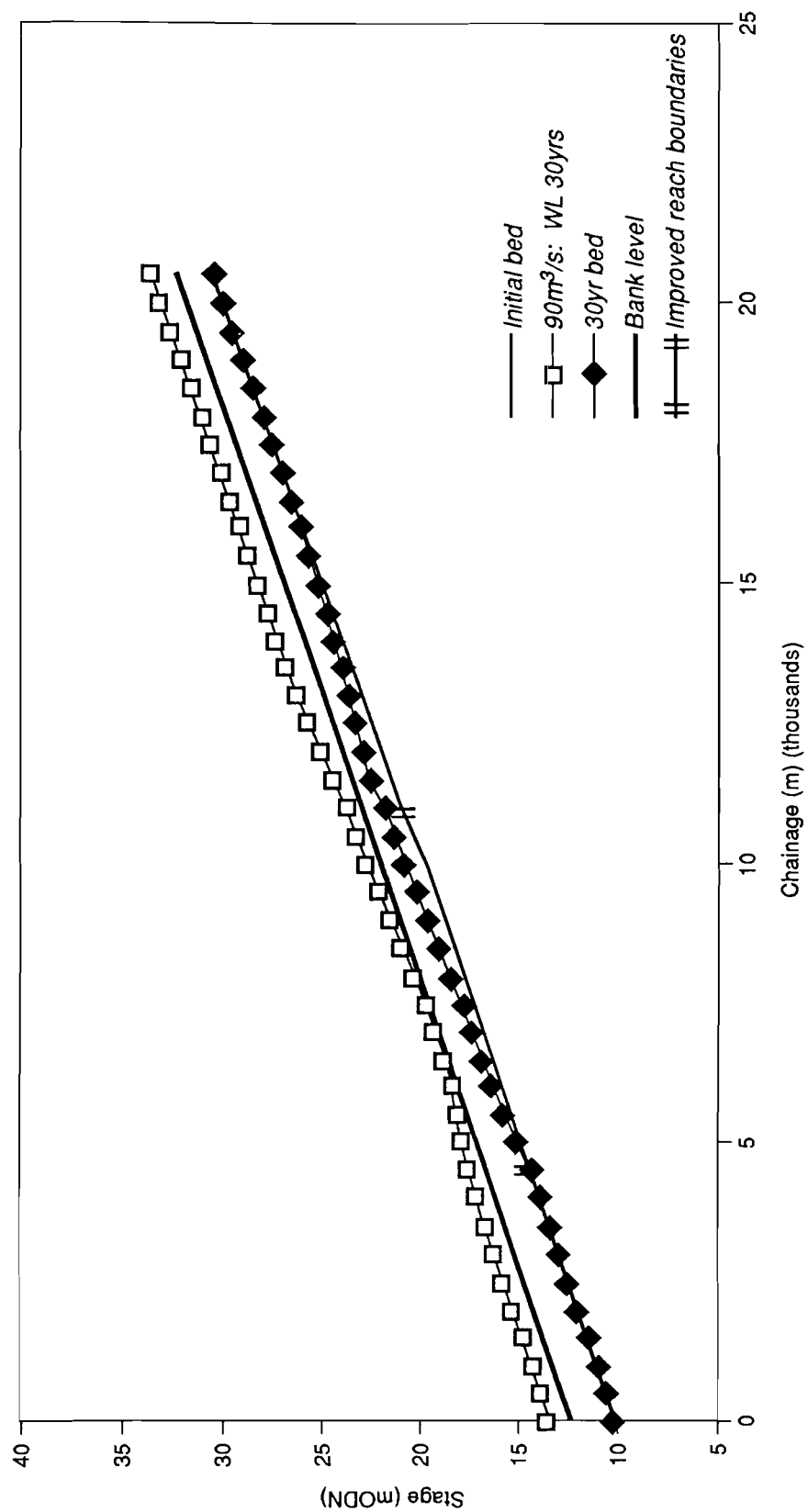
KF/B3/5-92/LO

Figure B3 Pre-improvement scheme bed and water levels



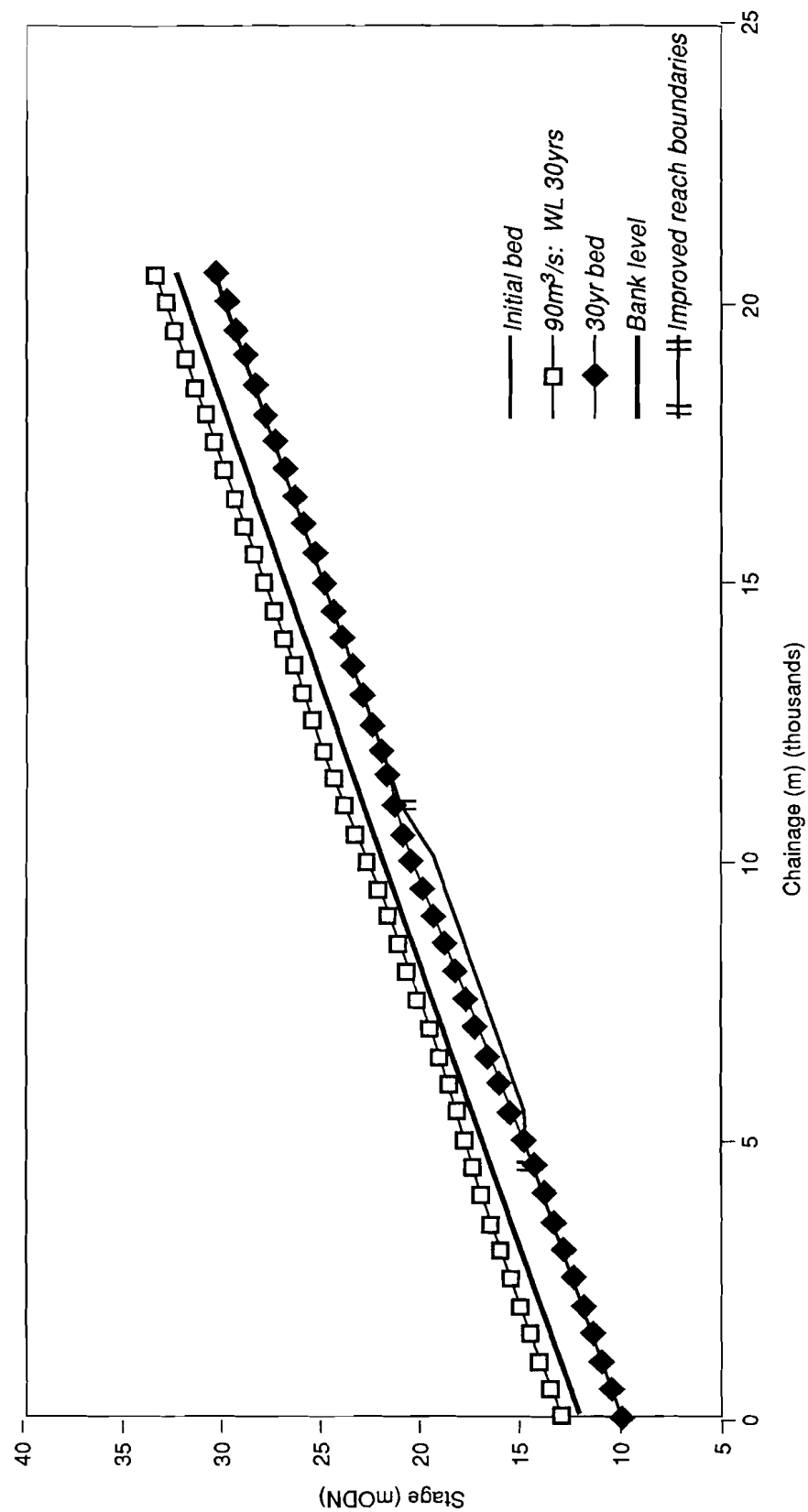
KF/B4/3-92/LO

Figure B4 Long section – Scheme 1: Bed and water levels initially and after 30 years at 90m³/s



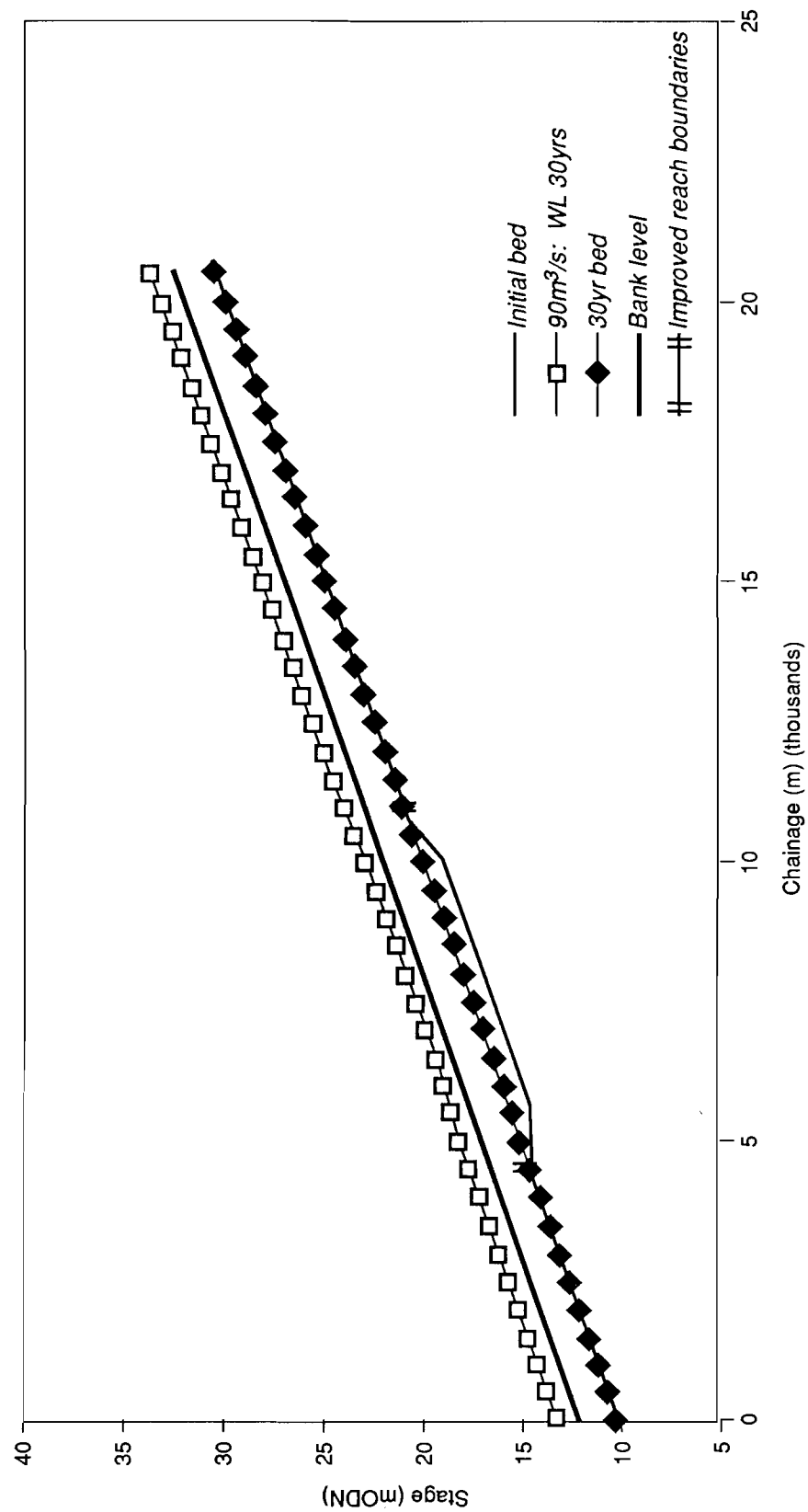
KF/B5/3-92/LO

Figure B5 Long section – Scheme 2: Bed and water levels initially and after 30 years at 90m³/s



KF/B6/3-92/LO

Figure B6 Long section – Scheme 3: Bed and water levels initially and after 30 years at 90m³/s



KF/B7/3-92/LO

Figure B7 Long section – Scheme 4: Bed and water levels initially and after 30 years at 90m³/s

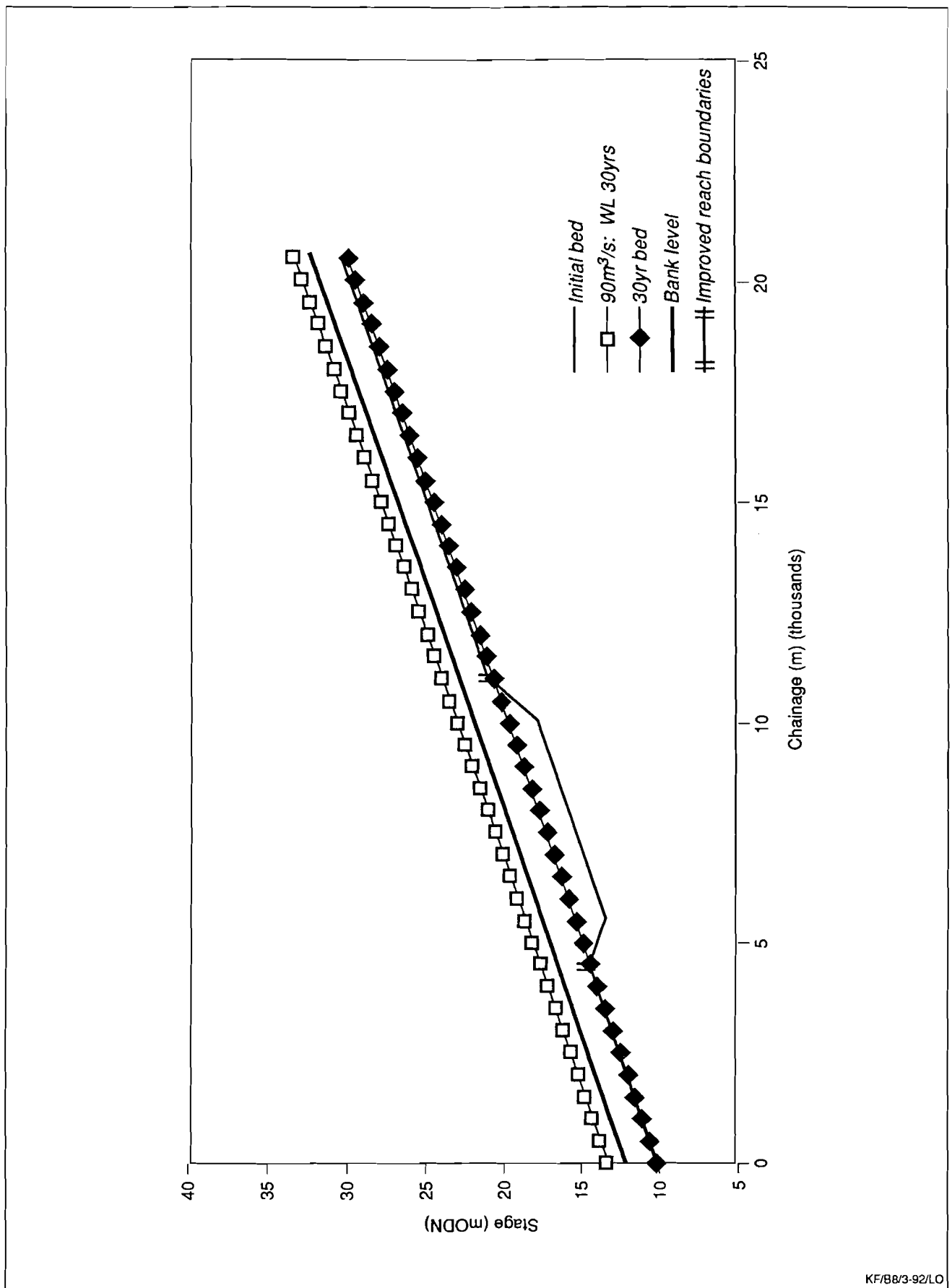


Figure B8 Long section – Scheme 5: Bed and water levels initially and after 30 years at 90m³/s

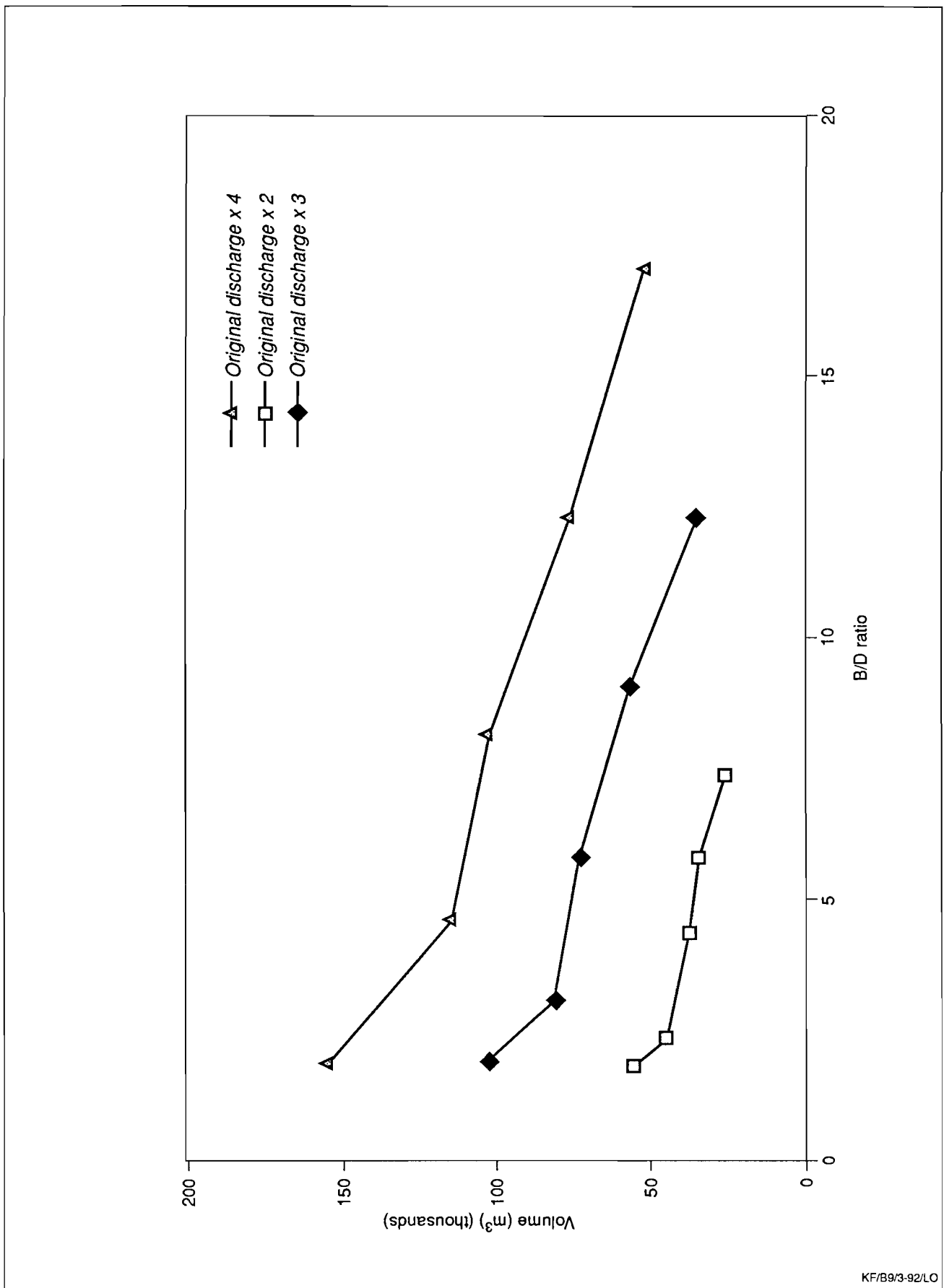


Figure B9 Volume deposited in scheme after 30yrs

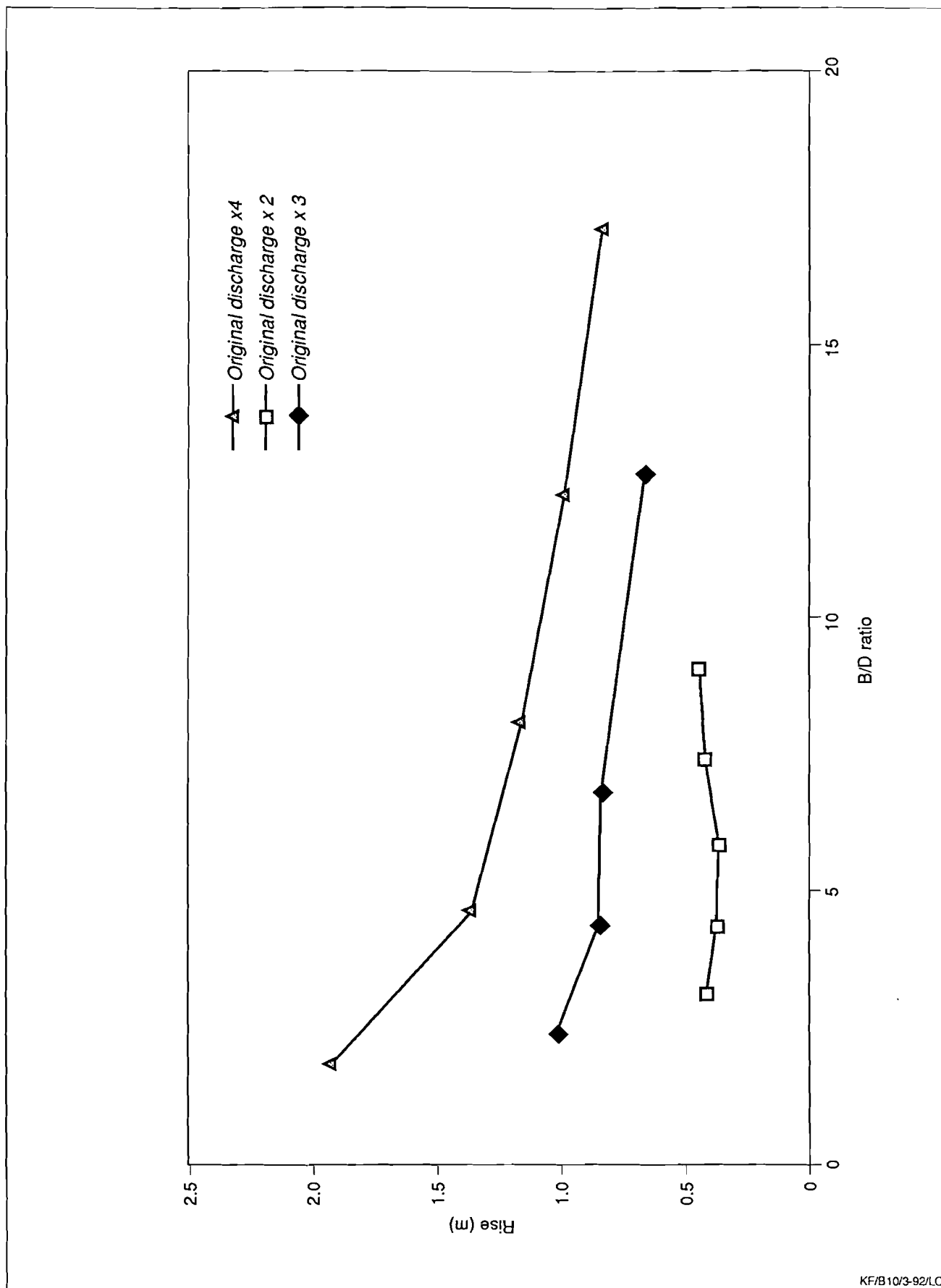


Figure B10 Maximum rise in water levels within scheme: 90 m³/s, 30 yrs

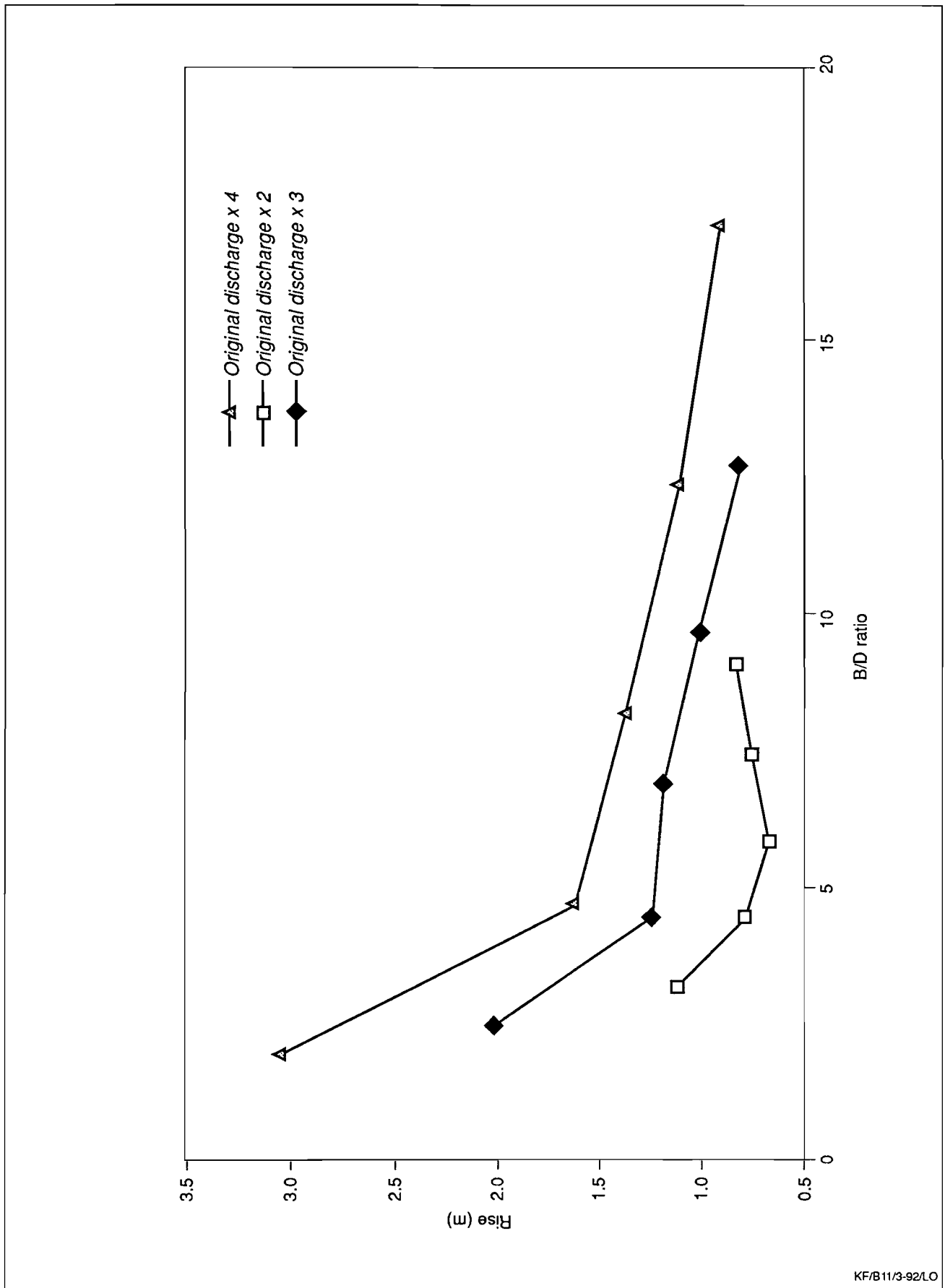


Figure B11 Maximum rise in minimum bed levels: 30yrs

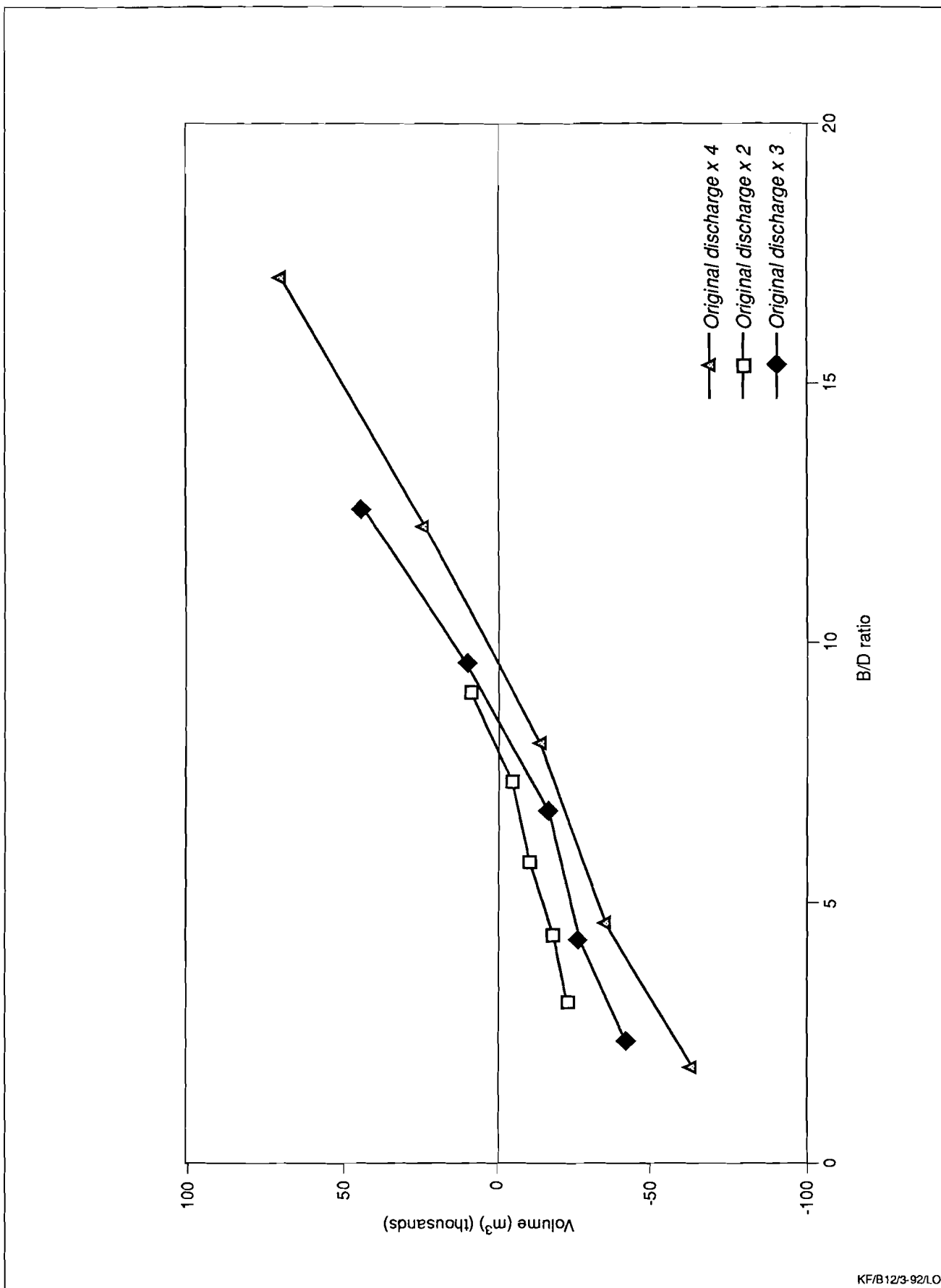
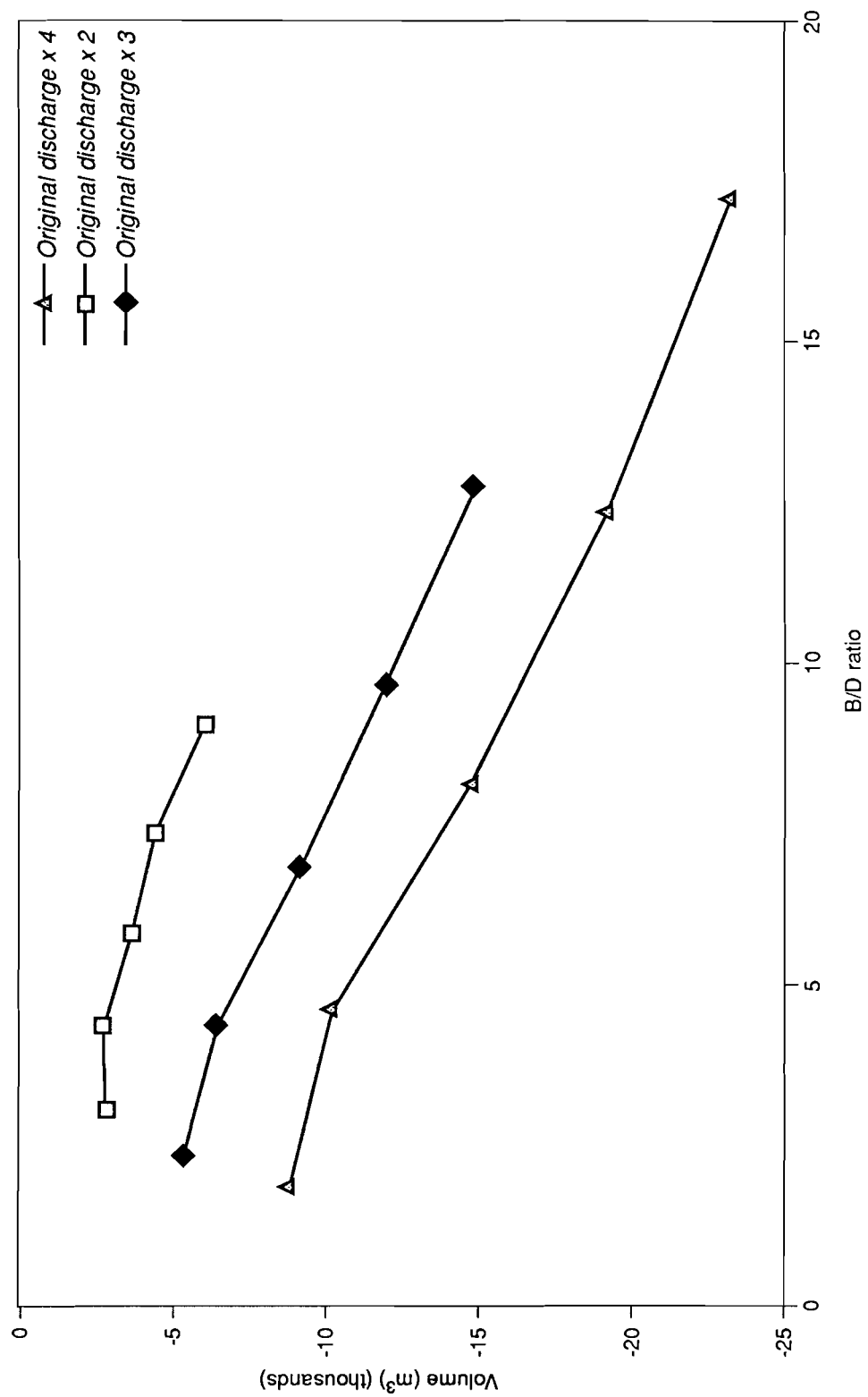


Figure B12 Volume deposited upstream of scheme after 30yrs



KF/B13/3-92/LO

Figure B13 Volume deposited downstream of scheme after 30yrs

Appendix C

Case Study : East Mill Improvement Scheme

C.1 Background

The East Mill scheme is situated at the tidal limit of the River Colne close to the centre of the town of Colchester, Essex. The scheme studied extends from East Mill to Middle Mill upstream, a reach of 1.5km in length.

The scheme was designed originally for provision of a water supply. The river was impounded by sluice gates at East Mill and tides excluded so that water could be extracted and treated for water supply.

The impoundment created severe man-induced flooding problems which had to be alleviated. An improved scheme was designed by Anglian Water and built in 1970 with a wider channel, some embankments upstream of East Mill on the right bank and new sluice gates at the East Mill site. The improved scheme was designed to contain floods up to a discharge of $71\text{m}^3/\text{s}$ with a return period in excess of 100 years.

During the design of the scheme sedimentation problems were not considered despite the fact that the reduction in flow velocities due to river impoundment would increase the potential for deposition. Erosion problems would be less likely to occur with the impoundment so their exclusion from design considerations would be reasonable.

Two years after the improved scheme was built the accumulation of sediment in the areas where the channel had been widened and deepened became apparent. After a period of ten years the channel was dredged for approximately 400m upstream of the sluice gates and $10,000\text{m}^3$ of mainly silt, with a small amount of sand and gravel, was removed.

C.2 Pre-impoundment scheme

East Mill is situated at the tidal limit of the River Colne and the spring, mean and neap tides have been estimated at +2.9m (+9.5ft), 2.29m (7.5ft) and 1.68m (5.5ft) above ODN. Approximately 250 tides every year reach a level of +2.44m (+8ft) ODN, 100 tides reach a level of +2.74m (+9ft) ODN and 25 tides reach a level of +3.05m (+10ft) ODN every year. There is no flooding of the tidal water back into the impounded reach.

The pre-improvement scheme layout is shown in Figure C1. The river was impounded at East Mill originally by three manually operated sluice gates which were under-shot and opened fully in the event of a flood. The two outer gates were adjustable but the centre gate was fixed at a level of 13.16ft ODN. The gates were generally operated to maintain the water level at +3.96m (13ft) ODN in the upstream channel. Each gate was 0.86m wide.

The original course of the river remained and can still be seen on the right bank. This channel was used as a back drainage system for the flood water on the right bank draining into the tidal channel downstream of the sluice gates.

The approach flow to the sluice gates turned through a right angle. Flow was directed under a road and then dropped over a weir into the tidal channel.

The scheme was designed to have a channel capacity of $12\text{m}^3/\text{s}$ (400 cusecs) and the channel had a trapezoidal cross-section. The river channel had a clay base covered with gravel and some sand and was very stable with no problems of erosion or deposition.

The sluice gates at Middle Mill at the upstream end of the reach are operated manually in the event of a flood or to maintain the water upstream at a constant level. There was a significant amount of scour on the left bank just downstream of this structure due to the alignment of the sluice gates. This bank was protected and stabilised.

Subsequent frequent flooding problems on the right and left bank upstream of the sluice gates at East Mill were considered to be a large problem especially when a new housing development was proposed on the right bank. Channel improvement proposals were to widen the channel upstream of East Mill sluice gates, build new and larger sluice gates at East Mill capable of passing greater capacity during floods and build an embankment on the right bank to protect the proposed housing development. These improvements are detailed below.

C.3 Improved scheme

In the downstream part of the scheme the old sluice gates were removed and the channel filled in for a distance of approximately 50m. New tilting, overshoot sluice gates were installed which maintain the water level in the channel upstream at a level of 3.96m ODN when they are in the raised position. In the lowered position the gates allow a flow of $28\text{m}^3/\text{s}$ (1000 cusecs) to pass through at a water level of 3.96m and $71\text{m}^3/\text{s}$ (2500 cusecs) at a level of 4.72m ODN which corresponds to a 1 in 100 year flood. The freeboard allowed was 0.61m for this flood event. The alignment of the new gates was also changed from the previous design and the flow continues along the channel down a drop and into the tidal channel downstream without taking a right angled turn as in the pre-improvement scheme.

The reach upstream of the East Mill sluice gates extends for 1650m to the sluice gates at Middle Mill. For 600m upstream of East Mill the channel was deepened and widened to contain the 1 in 100 year flood event. For the remaining 1000m upstream to Middle Mill there was some widening of the channel to allow flows of $28\text{m}^3/\text{s}$ to pass without flooding. In this region, at a discharge of $71\text{m}^3/\text{s}$, a width of 60m of the right bank would be flooded. For the lower section, East Mill and 650m upstream, an embankment was built on the right bank of the channel to protect the proposed housing development from any flooding effects.

A back drainage channel was a remnant of the original scheme and followed the course of the old river. A siphon which drained the left bank flood water into the back drainage channel was replaced and relocated 80m further upstream. The back drainage system discharges by a new sluice passing through a new tidal defence embankment into the tail water of the outfall structure.

C.4 Morphological problems

In the design of the new scheme no considerations were made for any sediment problems that might occur as a result of the new design.

Widening the channel in the downstream section of the reach would cause velocities to be reduced therefore increasing the potential for deposition. Erosion problems would be less likely to occur.

Approximately two years after the scheme was implemented, sediment began to accumulate just upstream of the new sluice gates and was noticed initially by fisherman. After ten years approximately 10,000m³ of silt was taken out of the river from between East Mill and 400m upstream of the sluice gates.

The deposited material reduces the flow capacity of the channel and it is not easily removed apart from dredging, a costly and time consuming job, especially as the new sluice gates are overshot gates making it difficult to remove the deposited silt, by flushing techniques.

Samples of the bed material at East Mill and upstream near Middle Mill were taken during a site visit and these are discussed in Section C5.1.

C.5 Model details

The morphological model was used to simulate flow conditions in both the pre-improvement scheme and the improved scheme. The aim of the simulations was firstly to describe the sediment movements before the channel was improved and predict areas of accretion or erosion. The second aim was to predict sediment movements and the effects on water level in the improved channel over a period of years. Further model runs were made with a modified channel layout designed to reduce potential sediment deposition.

C.5.1 Model data

The cross-section data used in the model was supplied by Anglian Water and included detailed channel and floodbank cross-sections of the reach between East Mill and Middle Mill. The 15 cross-sections were spaced between chainages of 500m and 1.5km. Highly irregular distances between cross-sections can inhibit the accuracy of the morphological model procedures.

The flow duration relationship was taken from data collected at Lexden, a gauging station 2km upstream of Middle Mill on the Colne River. The flow data for this gauging station was presented as average daily flows over a period of 29 years. From this data a flow exceedence curve was derived which has a peak discharge of 23.5m³/s (see Fig C2).

The flow discharge relationship provides the morphological model with a range of discharges and the length of time during a period of a year over which that discharge is expected.

The sluice gates for the pre-improvement scheme were designed to maintain the water level upstream of 3.962m ODN. The stage discharge relationship for the sluice gates operating in the fully open condition during high discharge or flood conditions

$$Q = C_d \cdot 2/3 \cdot \sqrt{2 \cdot g} \cdot b \cdot (h)^{1.5}$$

Where C_d = coefficient of discharge
 h = depth of water
 b = width of sluice gate

In the fully open operating condition the gates behave as broad crested weirs. Since the flow approaches the structure round a 90° bend the coefficient of discharge will be 1.0. Under normal flow conditions the downstream tailwater is a constant water level of 3.96m ODN.

The rating curve for this structure is given in Figure C3.

The stage discharge relationship of the sluice gates for the post-improvement scheme were designed to maintain the water level upstream at 3.96m ODN but operate in a different way to the pre-scheme gates being tilting gates which allow overtopping but no under flow.

The stage/discharge relationship for the gates during high discharge or flood conditions when gates are laid flat, fully open, is based on that for a short crested weir:

$$Q = C_d \cdot C_v \cdot 2/3 \sqrt{2 \cdot g} \cdot b \cdot (h)^{1.5}$$

The rating curve for this structure is given in Figure C4.

A sample of bed sediment was taken upstream of the sluice gates at the East Mill site. This sediment was fine silt of D_{50} size = 0.003mm as shown in Figure C5. This corresponded well with the type of sediment taken out of this area ten years after the improvement scheme had been built.

Another bed sediment sample was taken from the upstream section of the reach near Middle Mill. This sample was made up to sand and gravel of size D_{35} = 0.5mm. There was no evidence in the scheme of movement of this sand and gravel. The bed at the upstream sections near Middle Mill was very stable with little or no suspended sediment at the time of observation.

C.5.2 Model calibration

Since the only sediment data available was for the improved scheme, the model calibration was carried out with the section and discharge data for this scheme only.

The morphological model operates with either sand or silt input conditions or a combination of both. The sand routine calculates the concentration and movement of sand according to the Ackers-White sediment transport theory. The transport and deposition or erosion of silt and the resulting concentrations are dependent on the shear velocity at the bed. If the shear velocity is less than a certain critical value, deposition only occurs. If shear velocities exceed a certain critical value erosion will occur.

From site observations the type of sediment which appeared to have been transported was mainly silt and the stable bed was made up of sand and gravel. There was some movement of sand along the reach and a small amount of sand deposition at the lower sections of the reach where the channel had been widened and deepened but the majority of the deposition was due to silt.

It was assumed that there would be no input concentration of sand into the reach therefore defining the upstream boundary condition for the sand. The percentage of sand in the bed was defined as being 50%. The size of sand used was D_{50} = 1mm and the roughness height k_s taken as 0.01m.

From the sediment samples taken, Figure C5 it can be seen that some of the sediment in the River Colne is silt. From the silt sample and the known volume of silt deposited upstream of East Mill sluice gates an estimate of the upstream silt input was made. The input concentration of silt into the reach was taken from the silt rating curve relating silt concentrations in ppm to water discharge (m^3/s)

over a period of a year. At the high discharge values, greater than $20\text{m}^3/\text{s}$, the concentration of silt was 1000 ppm, falling to concentrations of 100 ppm at low discharges of below $1\text{m}^3/\text{s}$.

The morphological model was run over a simulated period of ten years with the sediment conditions as described above.

As there was only a limited amount of data available concerning the amount of sediment deposited in the river with the new scheme this calibration could only be considered qualitatively.

C.6 Model results

C.6.1 Pre-improvement scheme

The morphological model was run with the sediment conditions as described above and with the section data and tailwater conditions for the pre-scheme case at East Mill.

The model was run for a simulated ten years and the resultant bed levels over this period are shown in Figure C6. It can be seen that although there is some erosion and deposition in the downstream sections of the reach this corresponds to a depth of sediment of less than 0.2m over ten years. This amount of sediment accumulation over ten years would not cause a significant problem. This result corresponds to the actual behaviour of the river before improvements when there were no significant sediment problems.

C.6.2 Post-improvement scheme

The model results for the post-scheme are shown in Figure C7. The model was run with sediment conditions as described in the calibration run, and with the section data and tail water conditions for the improved scheme case at East Mill.

As before, the model was run for a simulation period of ten years. The resultant bed levels are shown in Figure C7. It can be seen that the model predicts a significant amount of deposition in the lower sections of the reach, 400m upstream of East Mill sluice gates. The sediment deposition has raised the bed by up to 1m and a volume of 6000m^3 has been deposited. Most of this deposit is silt. There is also some deposition in the sections further upstream but this is not a significant amount.

Comparison of these results for the post-improvement scheme with the prototype case shows that the model gives a significant amount of deposition in the sections where the channel has been widened and deepened, as occurred in the prototype. However, the actual volume of sediment deposited in the reach 400m upstream of East Mill in the prototype channel was 4000m^3 more than predicted by the model.

The downstream limit of the morphological model was taken just upstream of the abstraction point for the water supply. This abstraction removes 90% of the water from the river. The velocities in this section of channel, from the final cross-section to the sluice gates which was not modelled, would be severely reduced causing much deposition in this region therefore the difference in the amounts of sediment in model (6000m^3) and prototype (10000m^3) after ten years could be explained by the fact that the model was not used to simulate

conditions in the section downstream of the abstraction point where a large amount of deposition would have taken place.

C.6.3 Flood conditions

The improved scheme was designed for a 1 in 100 year flood. The original bed levels and predicted water levels for this event are compared with predicted bed levels and water levels after a simulated ten years in Figure C8. It can be seen that the water levels have increased due to the sedimentation after ten years. There is little increase in water level at the downstream end but an increase of approximately 0.15m at the upstream sections of the reach. The figure indicates there would be flooding problems on both banks in the upstream sections of the reach and in some areas of the left bank from chainage 1km to East Mill. The embankment constructed on the right bank contains this flooding event both with the original bed level and after ten years.

C.7 Modified scheme

An underlying assumption to this work is that the river was free from sediment problems under pre-scheme conditions. This assumption cannot be verified but no dredging of the river was reported by the Water Authority.

The probable causes of the sedimentation problems were considered to be due to one or more of the following factors:

- widening and deepening of the channel
- modification of the downstream control structure

In the improved scheme the channel was widened in the reach 400m upstream of East Mill sluice gates to the sluice gates and embankments were built on the right bank as a protection against flood water. A modified scheme was run on the model in which the channel was narrowed to approximately half the expanded width between chainages 1100m to 1500m and the embankments retained in situ. The results of this modification can be seen in Figure C10. Deposition in the 400m upstream of the sluice gates was reduced although there was still some accumulation of material over ten years to a depth of 0.4m. Narrowing the channel caused water levels to rise by a maximum of 0.15m at chainage 1km but this was contained within the embankments.

Figure C10 shows the results for the modified scheme after a simulated ten year period. The water levels are slightly increased in comparison to the improved scheme at the 1 in 100 year flood event with the largest increase being 0.27m at chainage 1km, 500m from East Mill sluice gates. The flood is still contained by the embankment on the right bank between chainage 1.1km and East Mill sluice gates. Flooding will occur in the upstream sections of the reach and along most of the left bank at this event.

C.8 Conclusions and recommendations

1. The main causes of the deposition of material was a significant widening of the channel in the area upstream of the sluice gates at East Mill and the installation of new sluice at East Mill which raised the impounded water level.
2. Sedimentation problems could be reduced by narrowing the channel, without causing a significant rise in water levels and retaining the existing embankments. However water levels would be slightly increased at the

narrowed section compared to the improved scheme by a maximum of 0.15m during one year flood flow conditions. For the 1 in 100 year flood event this would cause the water level to rise to within 0.1m of the right bank flood protection.

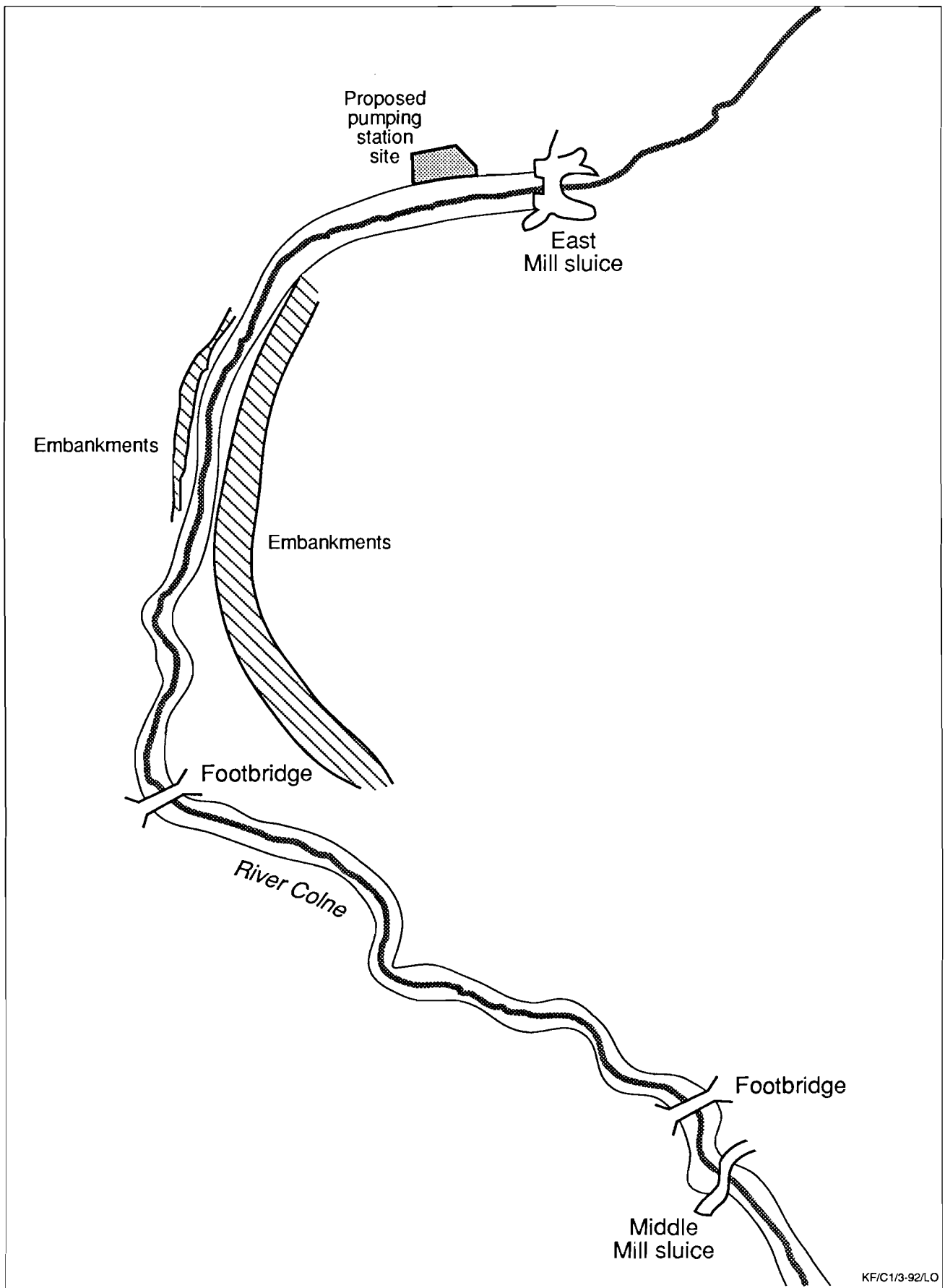
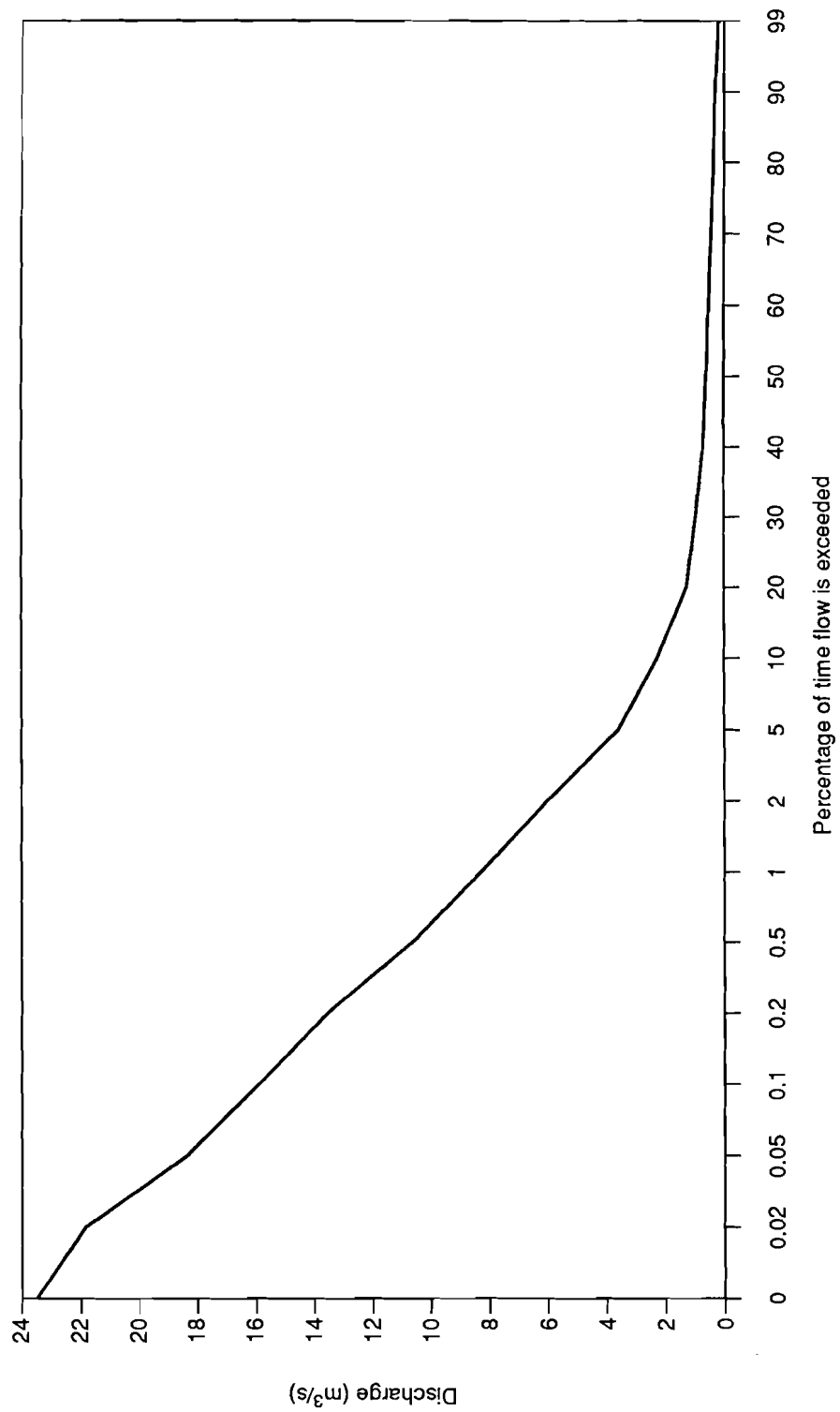


Figure C1 Layout of East Mill improvement scheme



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Figure C2 Flow exceedence relationship

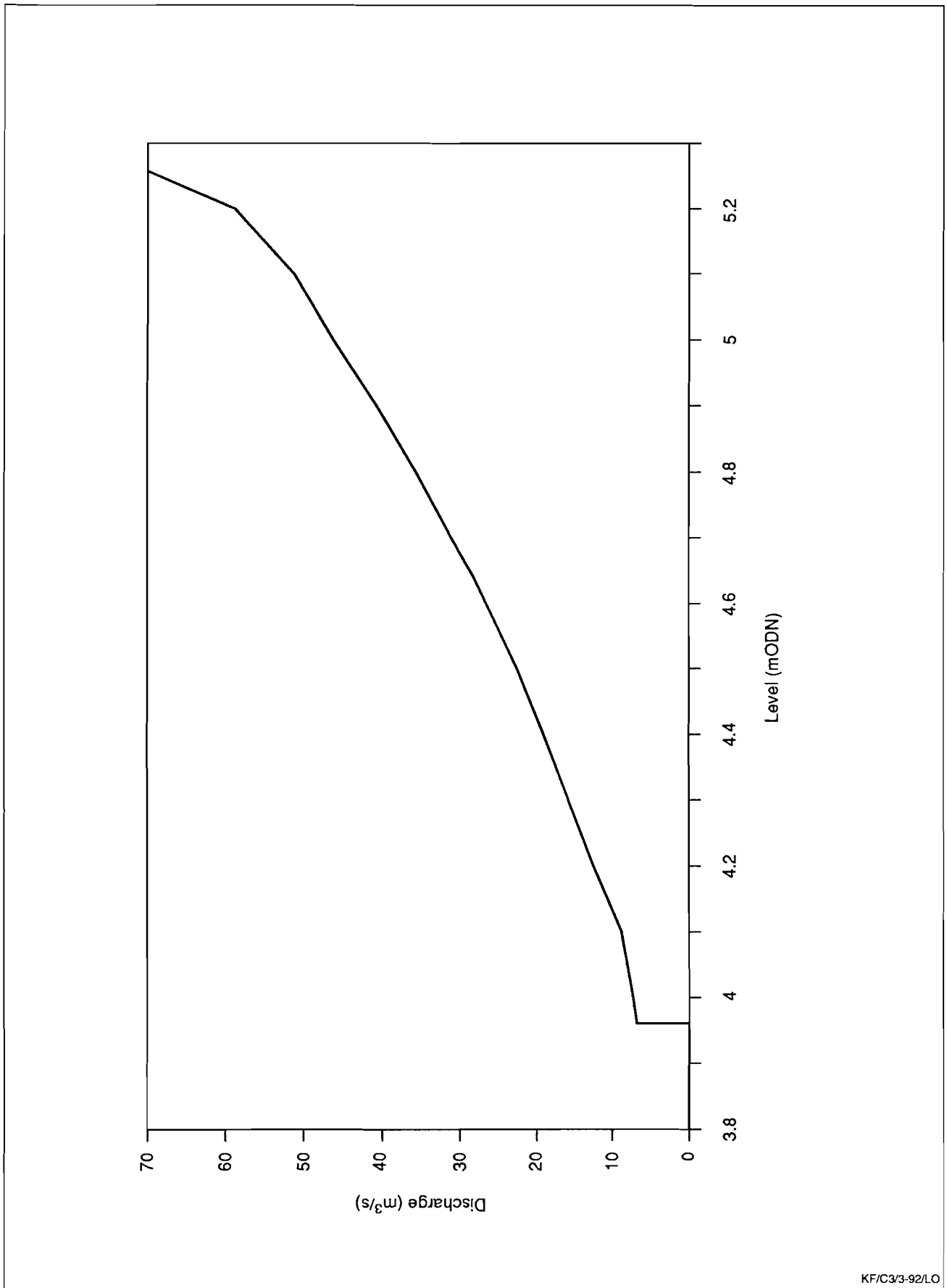


Figure C3 Stage discharge relationship at old sluice gates

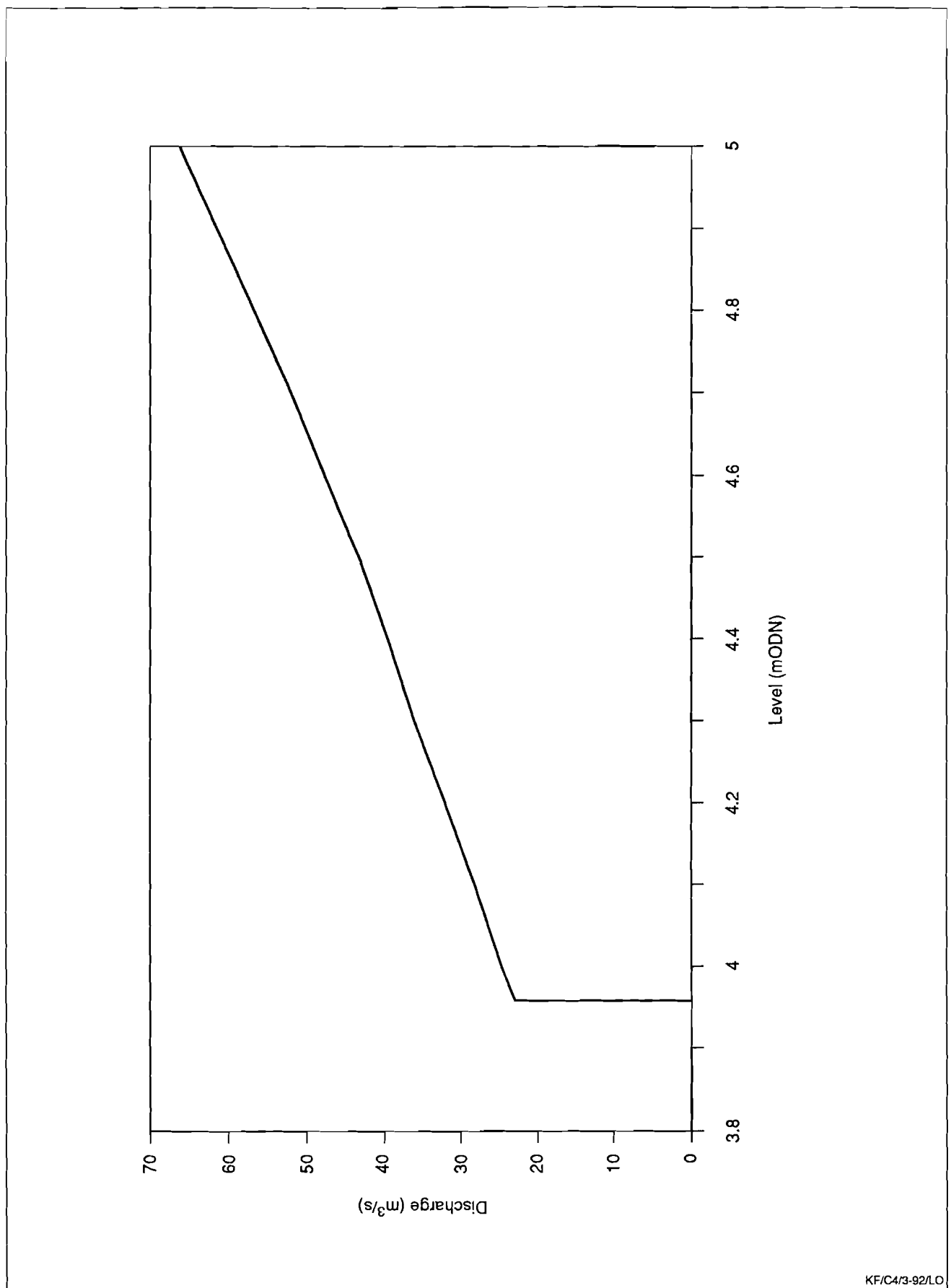


Figure C4 Stage discharge relationship at new sluice gates

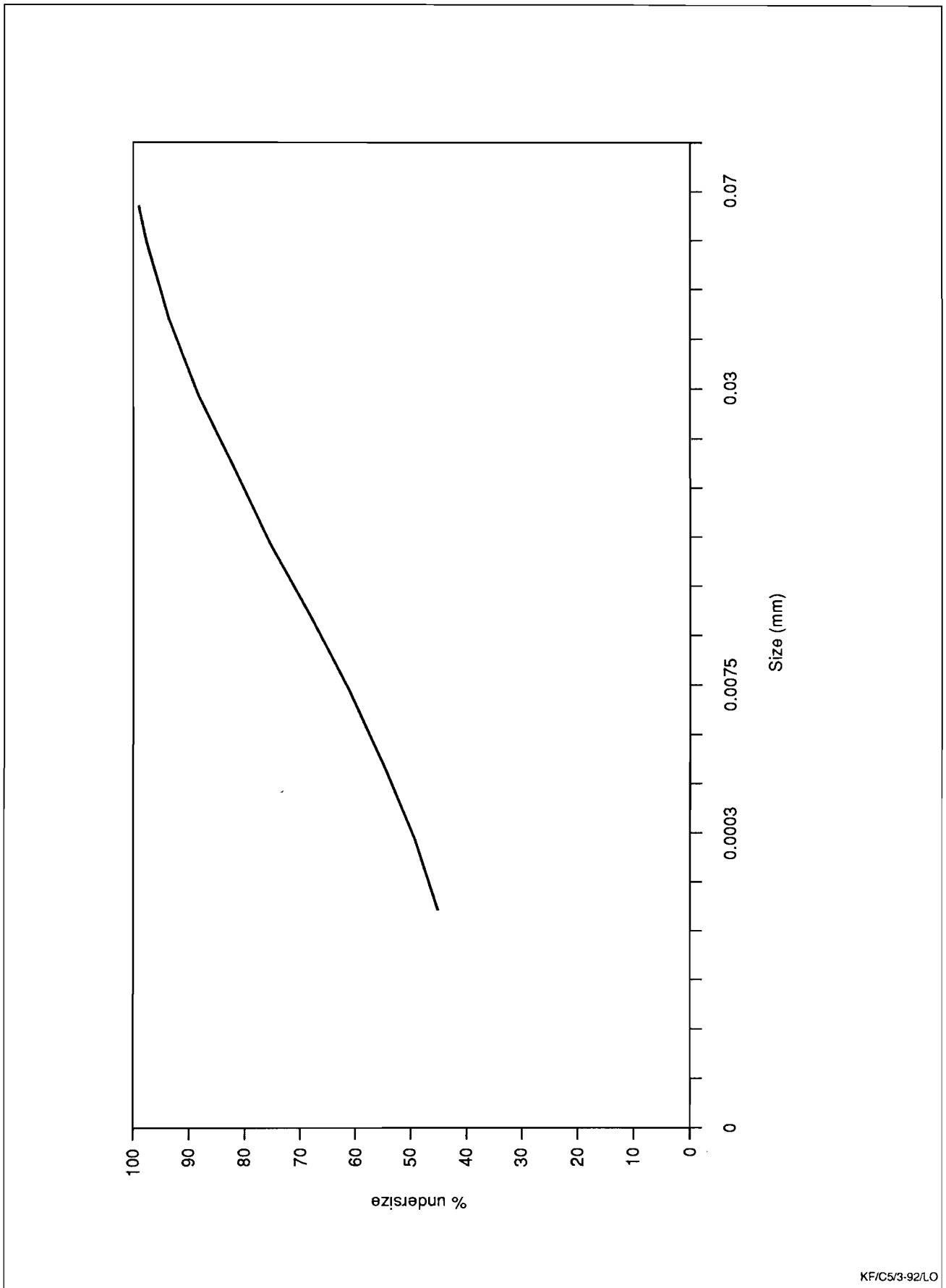
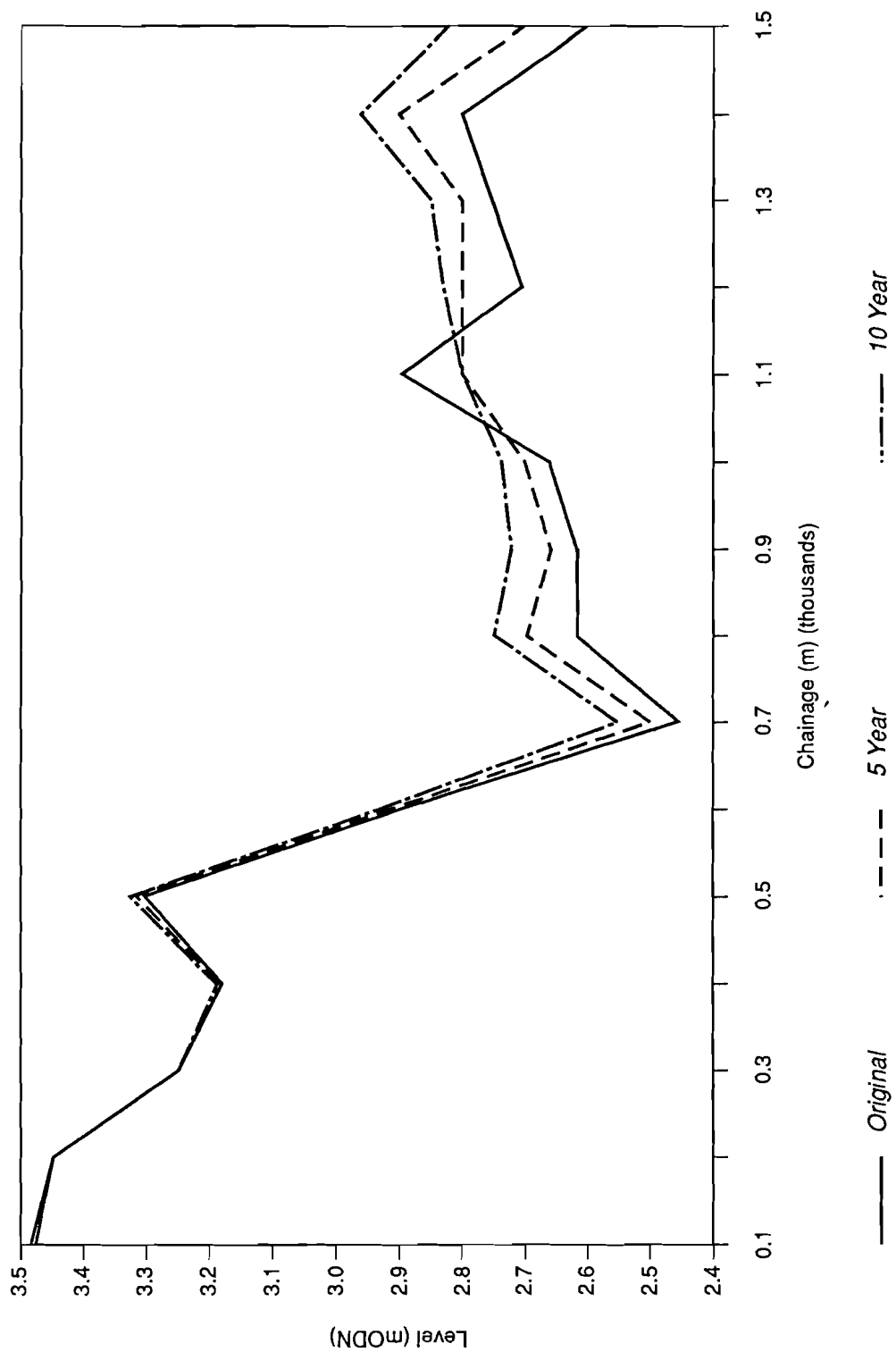
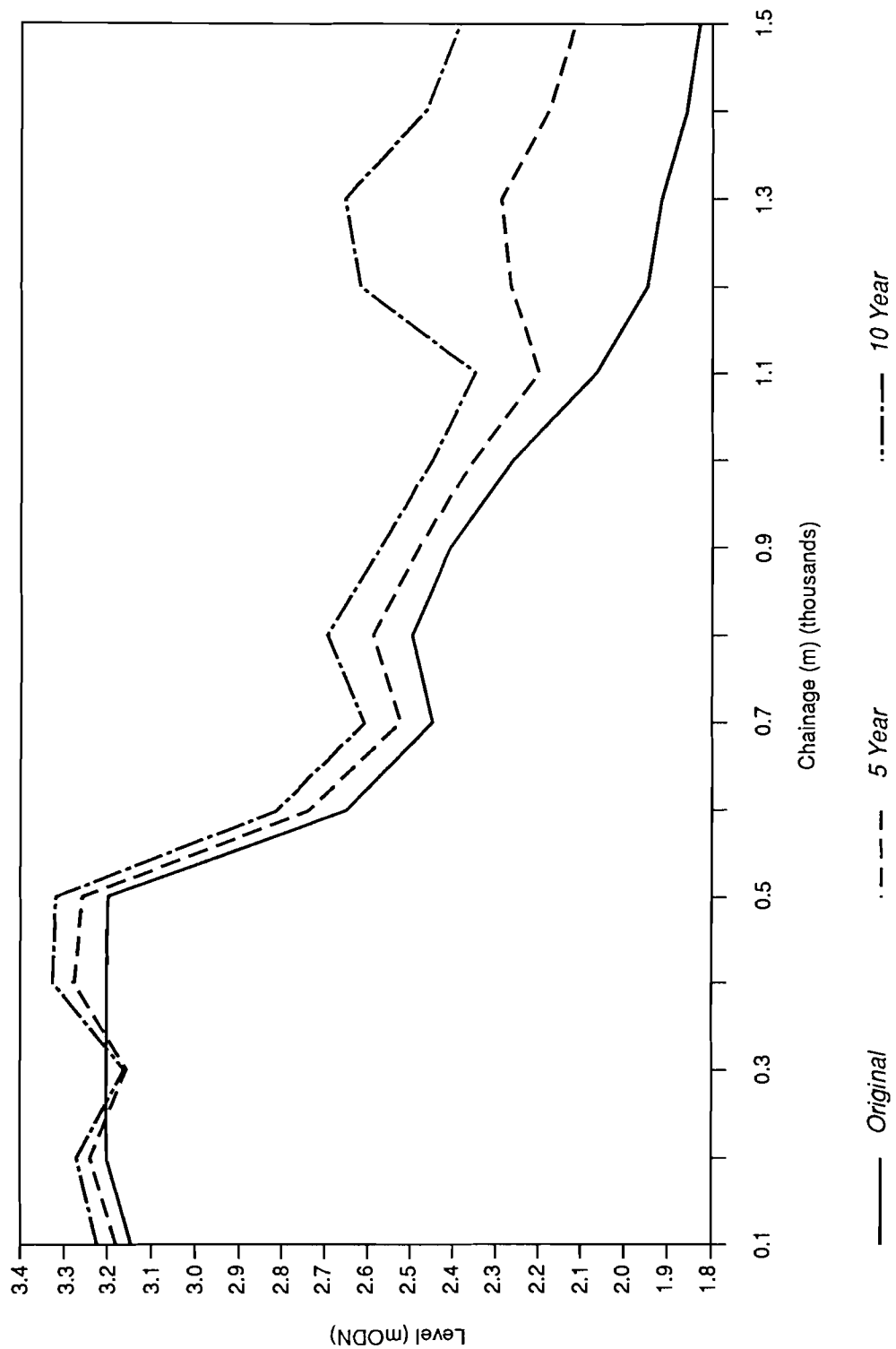


Figure C5 Sediment grading



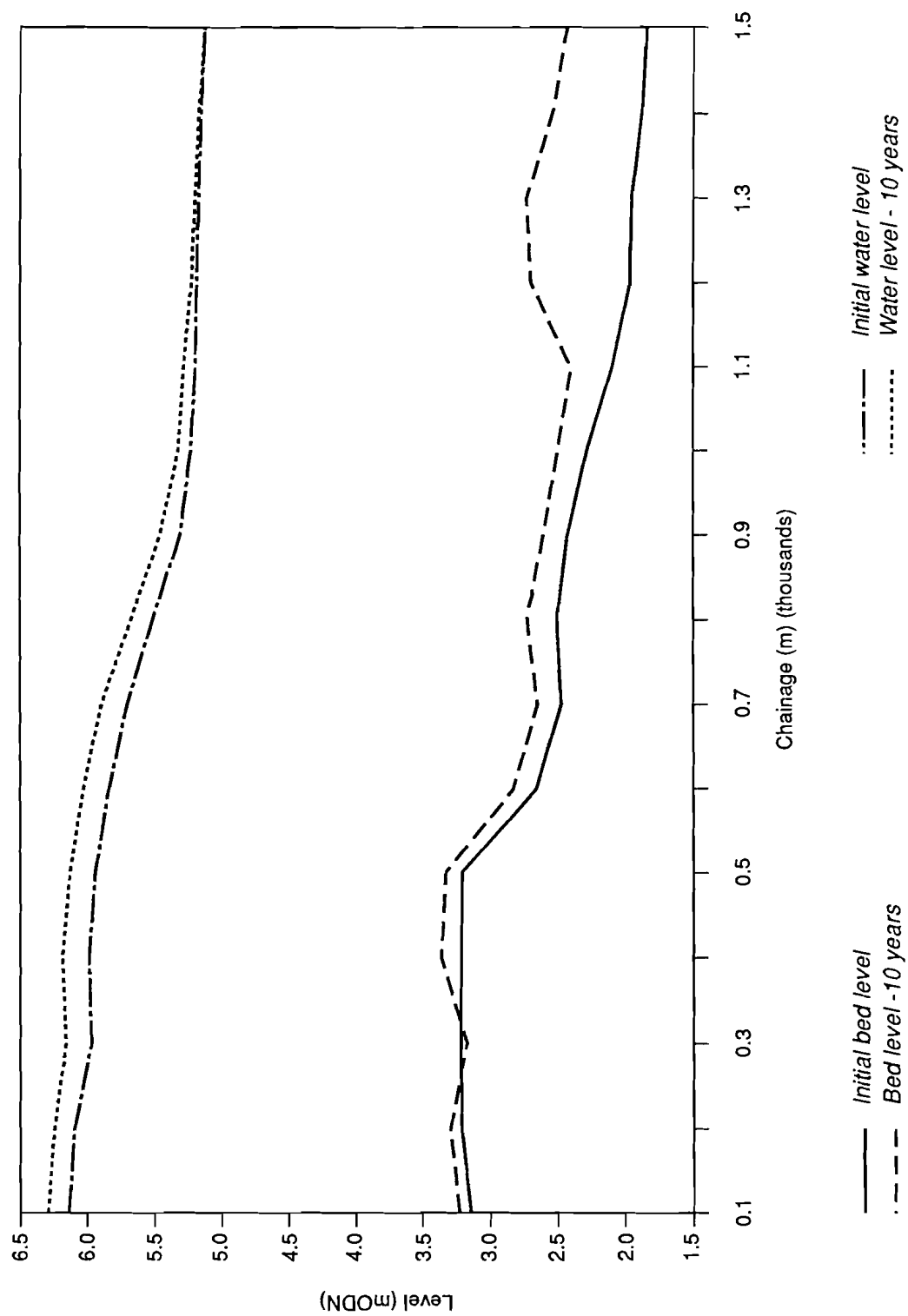
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Figure C6 Pre-improvement scheme: Bed levels



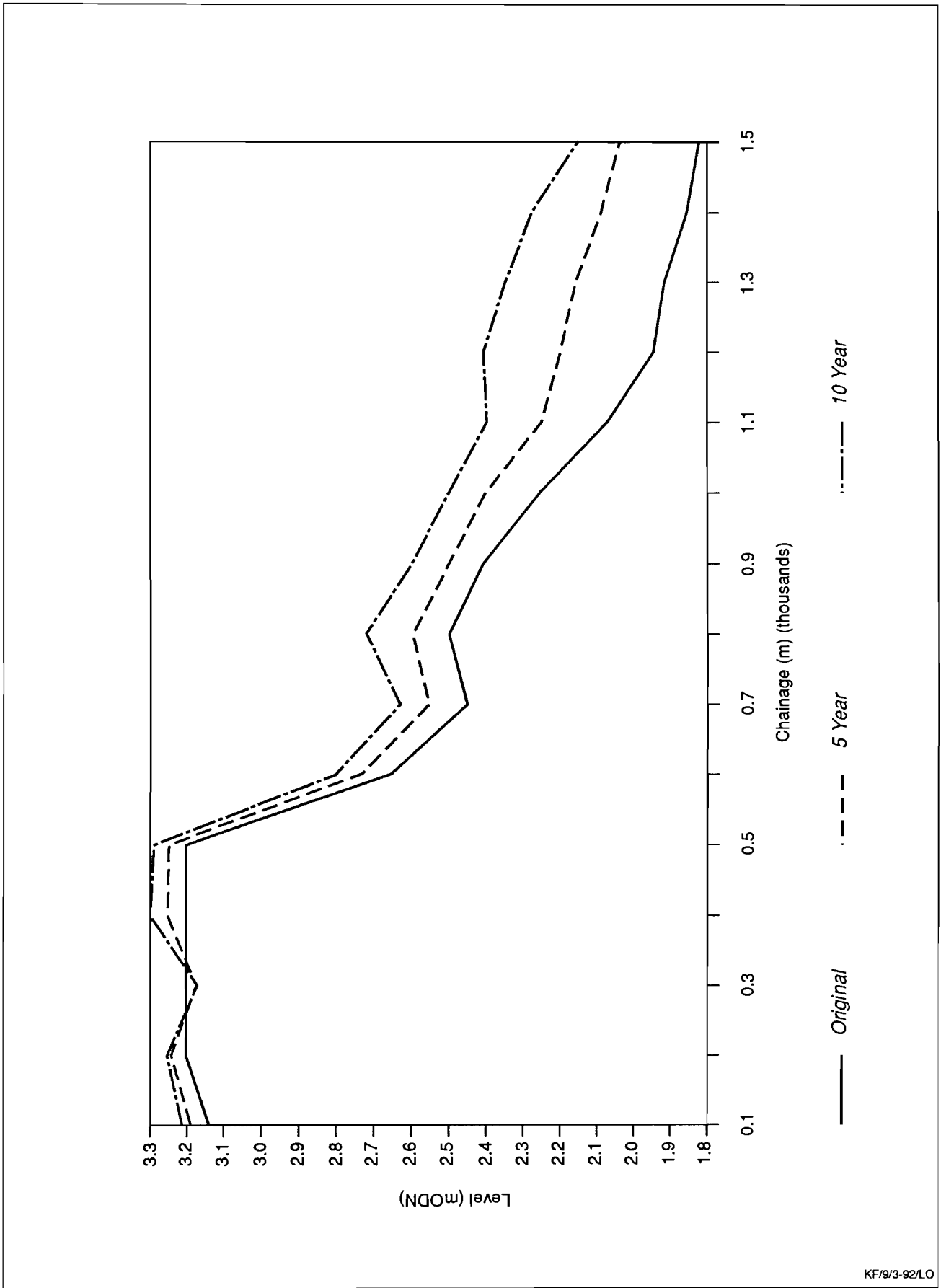
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Figure C7 Post improvement scheme: Bed levels



KF/C8/3-92/LO

Figure C8 Post improvement scheme: Bed levels and water levels
 $Q=70.7\text{m}^3/\text{s}$



KF/9/3-92/LO

Figure C9 Modified scheme: Bed levels

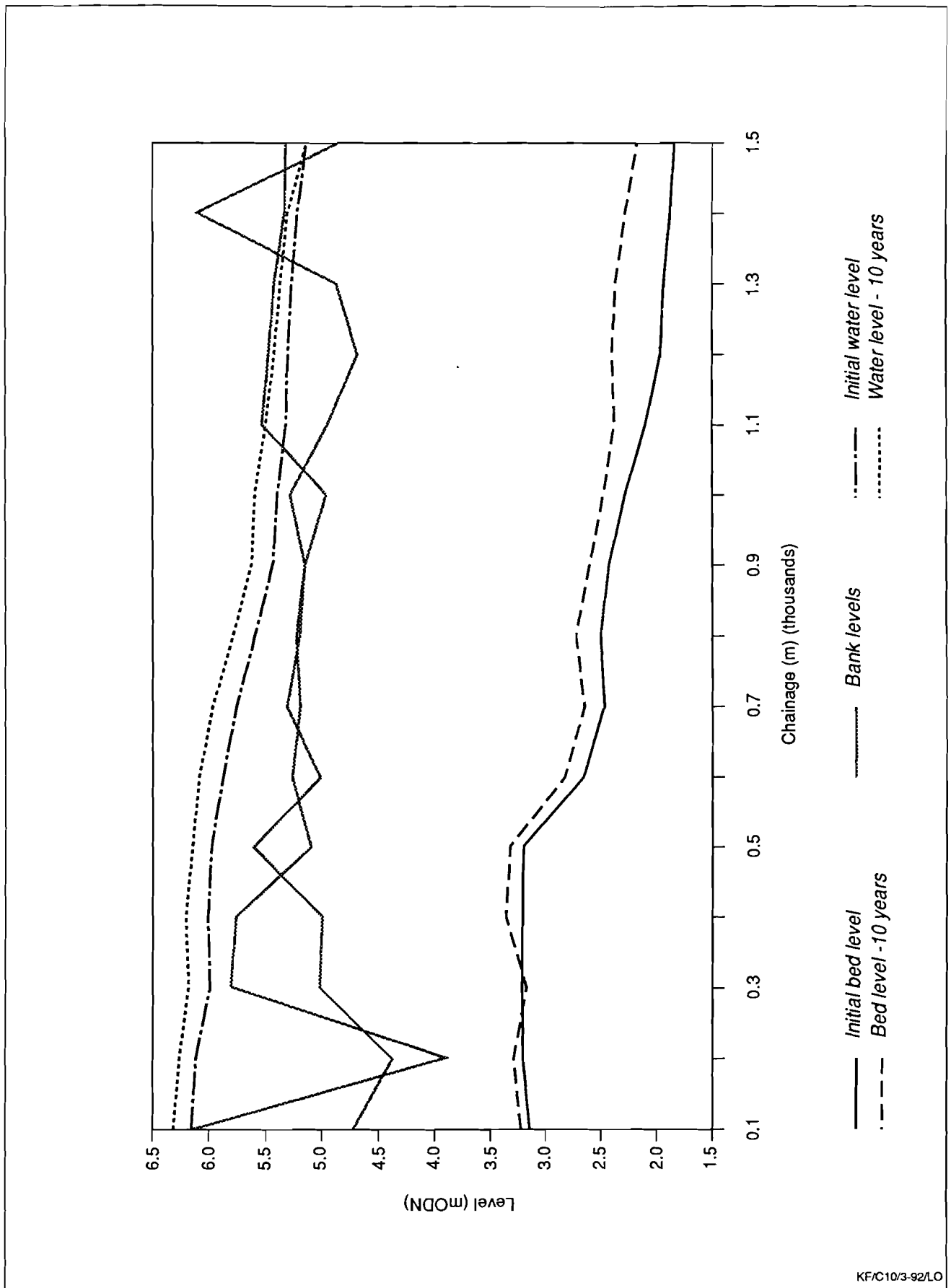


Figure C10 Modified scheme: Bed and water levels. $Q = 70.0\text{m}^3/\text{s}$

Appendix D

Case Study : Bures-Cornard Improvement Scheme River Stour

D.1 Background

The River Stour Improvement scheme was implemented in 1970 to protect agricultural land between the villages of Bures and Cornard over a distance of approximately 10km. The river between Bures Mill and Cornard Mill upstream has a long history of drainage difficulties which was attributed to the retention levels at the mills and the generally low discharge carrying capacity of the channel.

D.2 Pre-improvement scheme

The overall layout of the scheme can be seen in Figure D1. The length of river between Bures and Cornard originally contained two weirs at Pitmire and Henny. Pitmire weir is a sheet pile structure and Henny weir is of the broad crested type. At the downstream end of this stretch of river, Bures Mill, there are radial gates constructed in the 1930's and for a large discharge of $40\text{m}^3/\text{s}$, the gates at Bures Mill would be capable of maintaining the design retention level of 60ft. The gates at the upstream end of the model reach at Cornard are built to the same design as the radial gates at Bures Mill. This 10km length of the Stour is crossed by a road bridge at Bures, a rail bridge at Pitmire and a number of footbridges.

The bank full capacity downstream of Pitmire weir to Bures bridge was low and the flood plain frequently flooded. The drainage difficulties on this stretch of the reach caused conflicts. The local farmers required lower water levels and a reduction in the frequency of flooding, whilst other local residents wished to maintain the existing water levels and preserve water quality, retain the existing river for fishing and maintain the natural beauty of the area.

There were no records of any sedimentation problems, erosion or deposition, in this length of river prior to the improvement scheme. There were many indications however of impeded drainage on grasslands, waterlogging on sub-soils and a lack of soil structure due to the frequency of flooding.

D.3 Improved scheme details

A proposed solution to the flooding problems in the 10km length of river between Bures and Cornard was suggested in 1970. The suggested scheme merged with the Sudbury flood alleviation scheme at the upstream end, Cornard Mill, and was designed to carry discharge of $50\text{m}^3/\text{s}$, a 1 in 25 year flood which subsequently has been shown to have a return period of 5 to 10 years. The design water levels at Bures gate were to be reduced by 0.61m and at Henry Mill by 0.31m. This reduced level would have led to a reduction in retained volume and it was proposed to deepen the channel and use the excavated material to raise the embankments, the highest embankment being 1.55m in the middle section of the length of river. The proposed levels of the improvement scheme from Bures Gate are shown in Figure D2.

In order to maintain existing water retention levels held by Bures Gate a new weir at Lamarsh was built. The proposal suggested raising existing water levels downstream of Henny Mill and second new weir was constructed. The positions of the new weirs can be seen in Figures D1 and D2.

Sedimentation problems were not considered at the design stage but potential erosion problems were anticipated and the new weirs constructed so as to reduce flow velocities and erosion.

D.4 Morphological problems and maintenance

Erosion has occurred upstream of Henny weir where the river by-passes the weir at high discharges. Other localised erosion and accretion has been observed at bends, for example upstream of Shalford weir but this is typical of a meandering river and cannot necessarily be associated with the implementation of the scheme. Erosion downstream of Bures bridge has been protected by revetments. Some weed growth is visible within the channel at Henny and grass cutting maintenance has been carried out on flood embankments.

D.5 Model details

D.5.1 Model data

The cross-section data used in the model was supplied by Anglian Water and include detailed channel and floodbank cross-sections of the length of river between Cornard Mill, at the upstream limit of the model and Bures Mill at the downstream limit. The pre-improvement scheme data was collected during several surveys over a number of years. The 38 sections in both the pre and post-improvement scheme modes represent a total distance of about 10km giving an average spacing of approximately 250m. Originally 59 cross-sections were supplied by Anglian Water but the spacing of some adjacent sections was small and caused numerical instability in the model.

The modelled length of the River Stour from Cornard Mill to Bures Mill is crossed by a road bridge at Bures, a rail bridge at Pitmire and a number of footbridges. It was considered that none of these structures was likely to cause significant obstruction to flow for the conditions tested and so they were not included in the model.

It was assumed for the purpose of this study that the input flow conditions remain unchanged for pre and post scheme. The weir constructed as part of the improvement scheme at Lamarsh is rated and used as a gauging station to monitor flow conditions. The flow duration curve for this weir was used as the basis for the model flow conditions, see Figure D3, with the curve for the high discharges being altered slightly to give a more conservative estimate of flow.

There are no significant tributaries or by-pass channels in the modelled length of river.

D.5.2 Model calibration

There is no sediment data available for the river before the improvement scheme. The model calibration was carried out using the cross-section and discharge data. The upstream sediment input was calculated using the bed slope and an estimated bed sample D_{35} size of 20mm resulted in virtually no movement of sediment in the model. In order to obtain more realistic transport rates the sediment size was reduced to 0.3mm. For this sediment size the model generated an upstream sediment input of 85 tonnes/year, approximately 0.2 tonnes/km²/year, an acceptable figure for a river of this character.

The morphological model was run over a simulation period of fifteen years to assess the bed stability using the sediment conditions as described above. The global roughness used in the calibration run was $k_s = 0.5\text{m}$ with the discharge threshold for sediment transport being 1.5 cumecs. With these parameters the calibration demonstrated that the channel was stable which is the assumed condition before the improvement scheme was implemented.

The results of the change in bed levels for the pre-improvement scheme situation over the 15 year period are shown in Figure D4 and apart from some local changes close to Pitmire and Henny weirs, the bed levels remained reasonably stable during the 15 year run, indicating that the pre-scheme model was operating satisfactorily. Bed instability near the weirs will be a result of difficulty in modelling the performance of weirs accurately for all flow rates.

D.6 Model results

D.6.1 Post improvement scheme

The upstream sediment inflow for the post-improvement scheme model was specified using a power law relationship to ensure transport rates comparable with the pre-scheme model. The resultant average sediment inflow was 87 tonnes/year and the global roughness used was k_s value of 0.5. The D_{35} sediment size used was 0.3mm.

The post-scheme model was run for a period of 15 years and the model results showed significant erosion downstream of the post-improvement weir built at Lamarsh, as seen in Figure D5. There has been no reported significant erosion downstream of this weir but it is unlikely that it would be detected unless a detailed survey of the river was carried out. This reach of river in fact was subjected to dredging works in the early 1980's to obtain material for the repair of embankments. Any changes in bed formation may have been obscured by this maintenance activity. A comparison of pre and post-improvement scheme bed levels is given in Figure D6.

D.6.2 Modified scheme

To investigate the model predictions more fully a series of subsequent tests were run under different conditions for a simulation period of 15 years. These tests observed the effect of:

- pre-improvement scheme tailwater conditions and weirs but post-improvement scheme cross-sections (case 1).
- pre-improvement scheme weirs but post-improvement scheme cross-sections and tailwater conditions (case 2).
- post-improvement scheme weirs and tailwater conditions but pre-improvement scheme cross-sections (case 3).

The results from these tests indicate that by replacing the pre-improvement scheme sections with the post-improvement sections (as in case 1) the freeboard of the river is increased for a given discharge, over the pre-improvement scheme case, without introducing any erosion, see Figure D7 and D8.

The post-improvement scheme as it exists gives a greater freeboard than the modified case 1 but as was detailed previously caused some erosion. By using

the pre-improvement scheme sections with the post-improvement scheme weirs and tailwater levels (case 3) there is an increase in freeboard over the pre-improvement scheme conditions but erosion downstream of Lamarsh weir is still significant.

D.7 Conclusions and recommendations

1. The model indicates that some erosion is likely downstream of a new weir constructed as part of the improvement scheme. This may not have been detected due to dredging work carried out in that reach of the river since the initial improvement works in 1970.
2. The erosion problems could have been reduced if the weirs and tailwater conditions of the pre-improvement scheme had remained and the cross-sections deepened and widened as in the present post-improvement scheme situation. This modified scheme would cause a slight increase in the freeboard available.

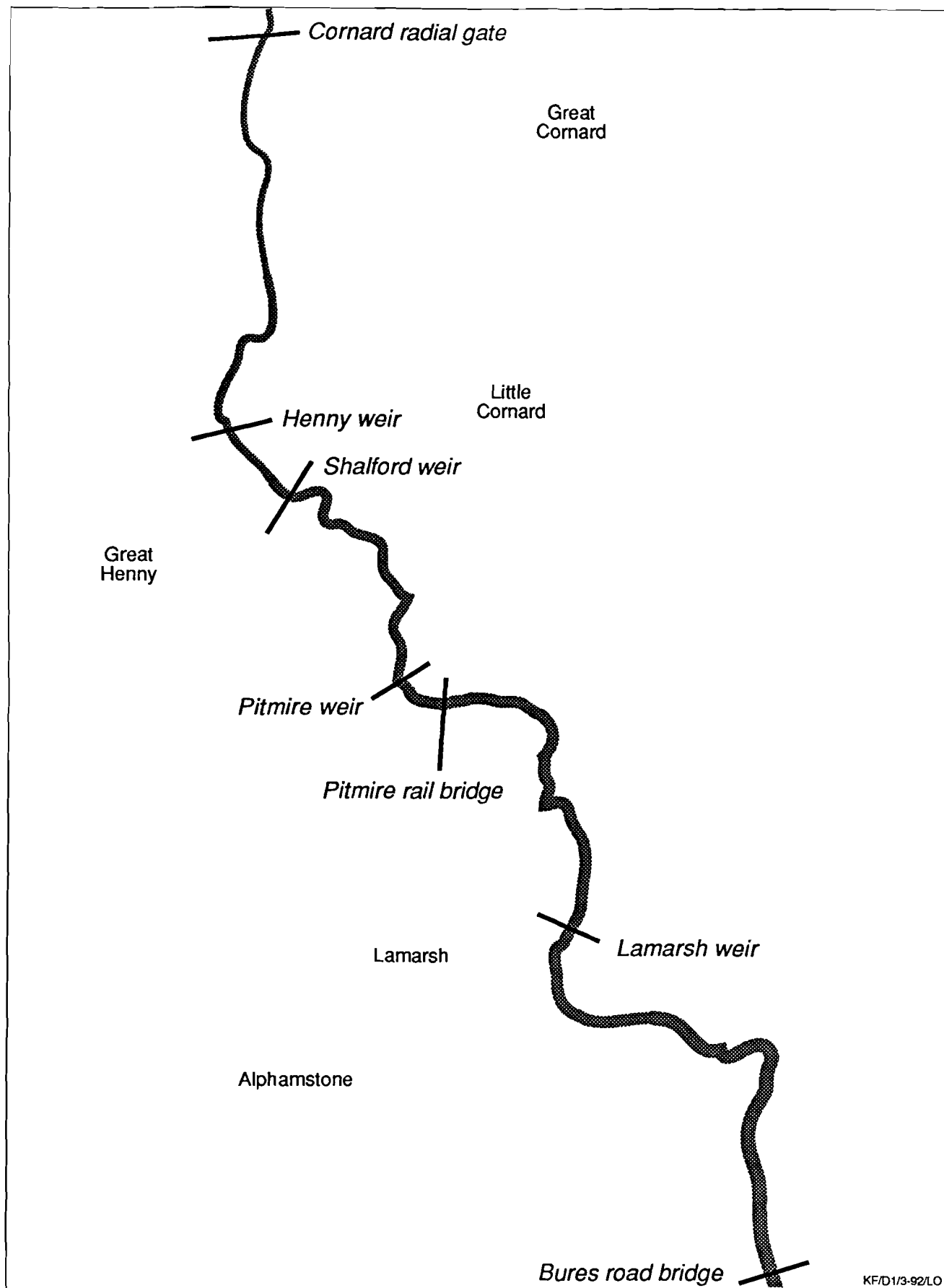


Figure D1 Layout of River Stour – Bures to Cornard

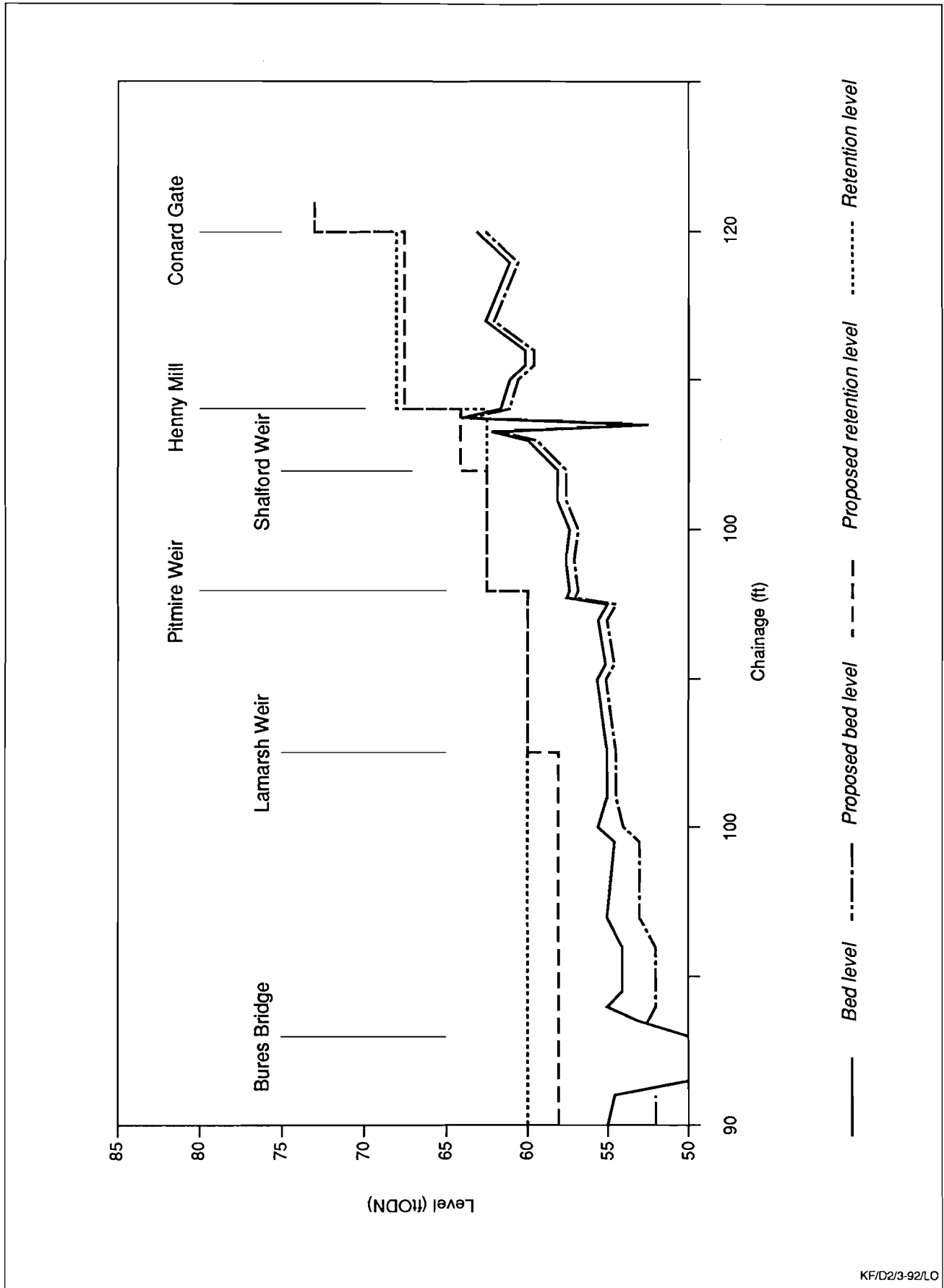
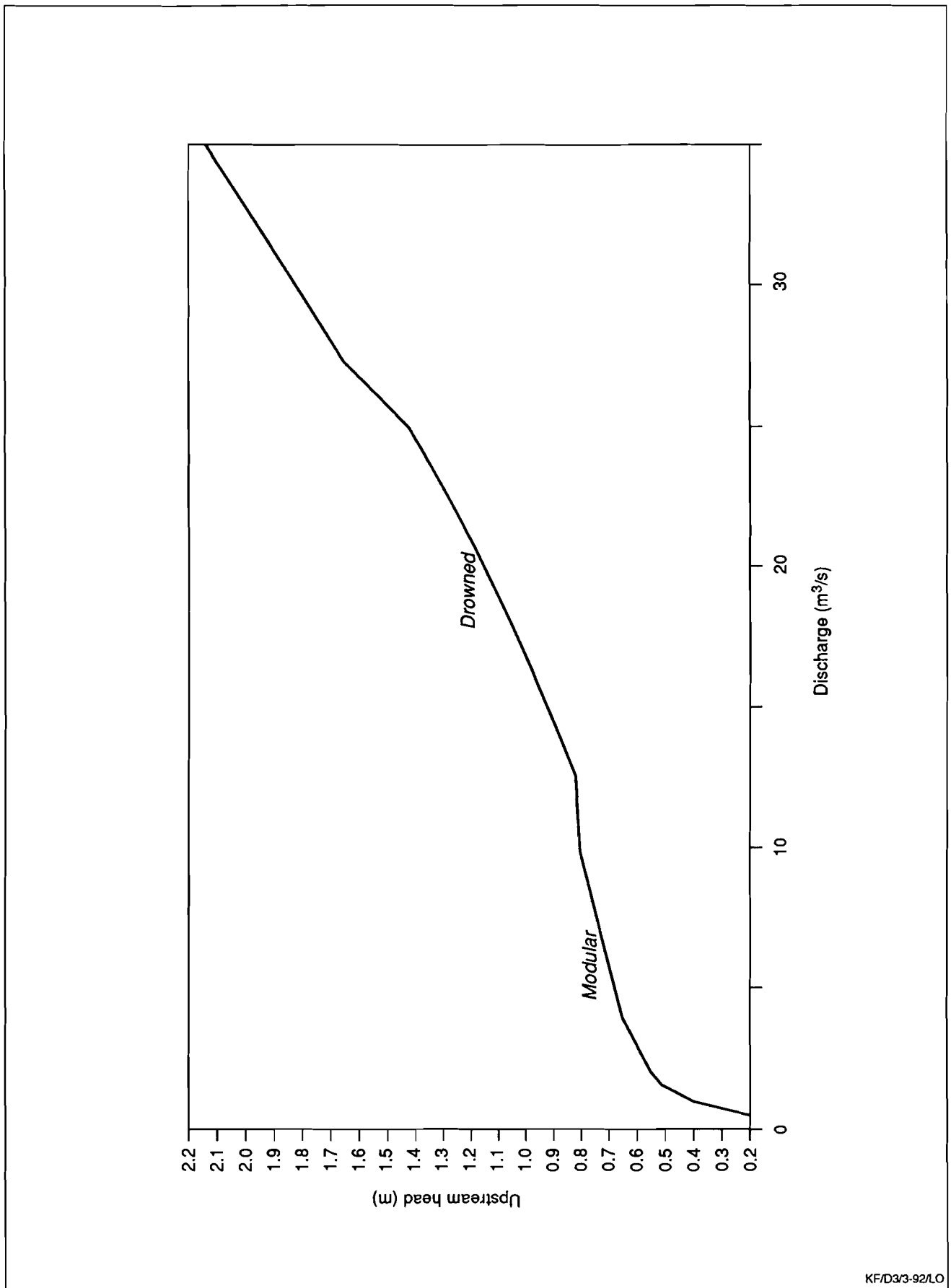
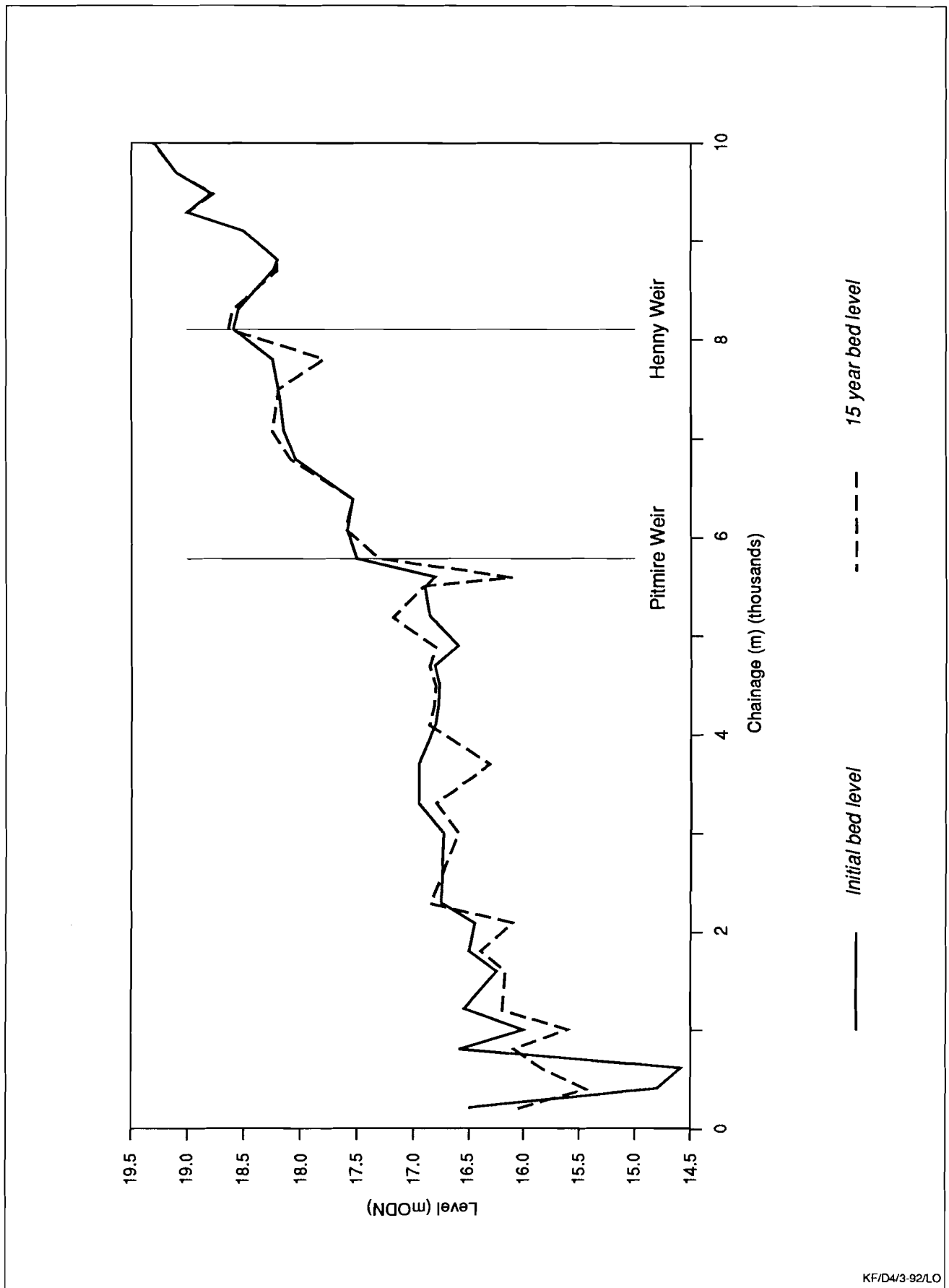


Figure D2 Comparative water levels and depths



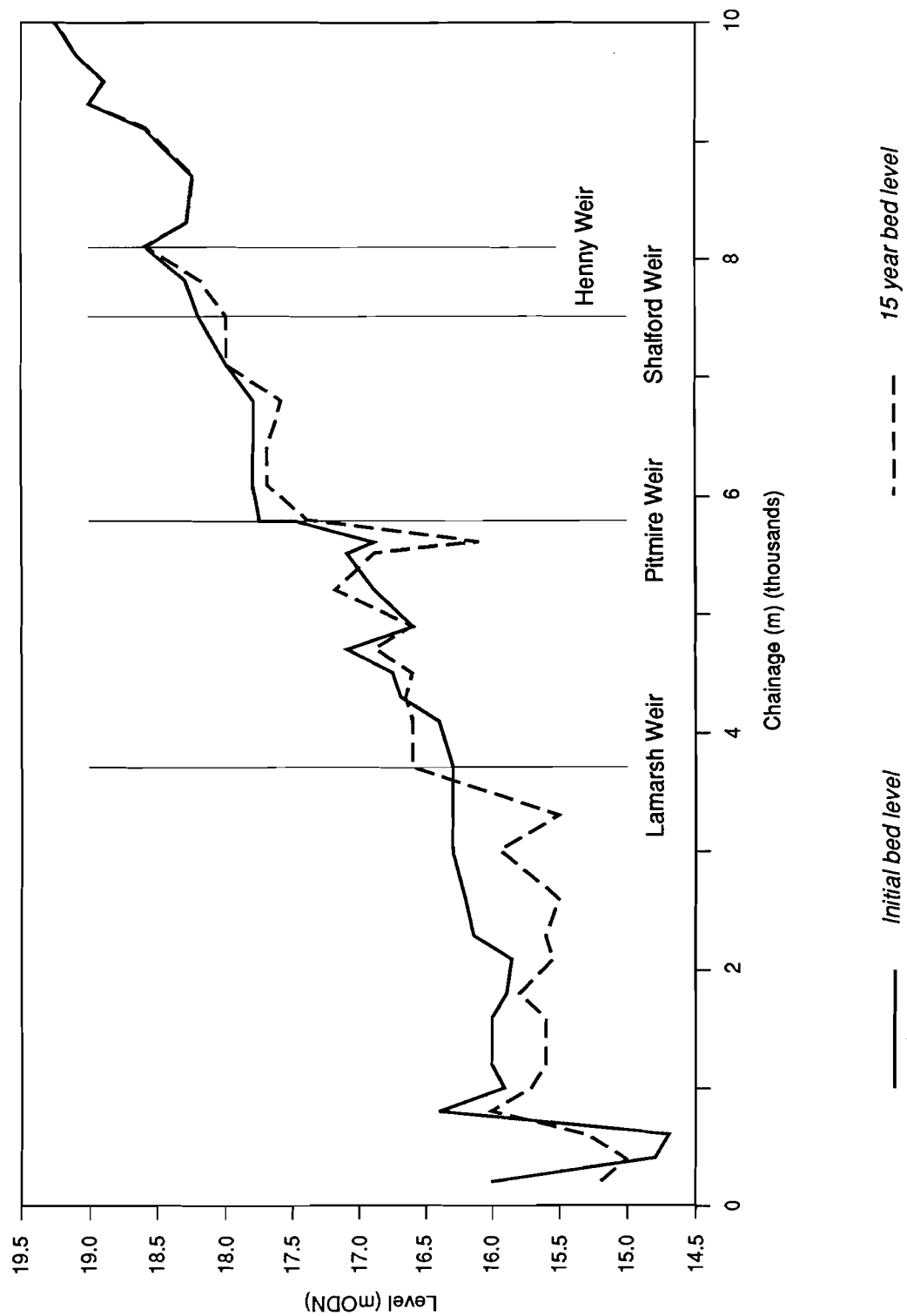
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Figure D3 Stage discharge relationship



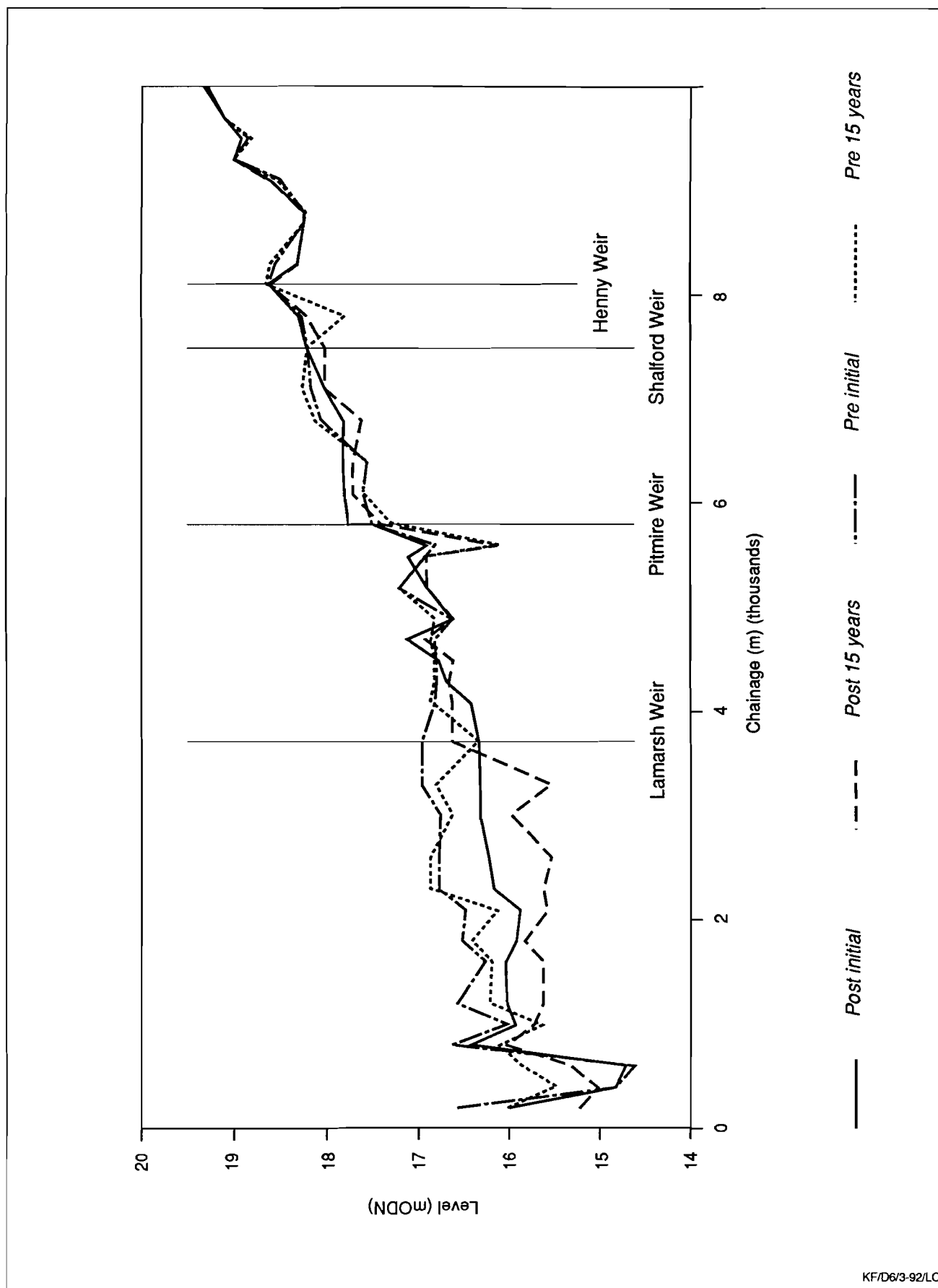
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Figure D4 Pre-improvement scheme: Bed levels



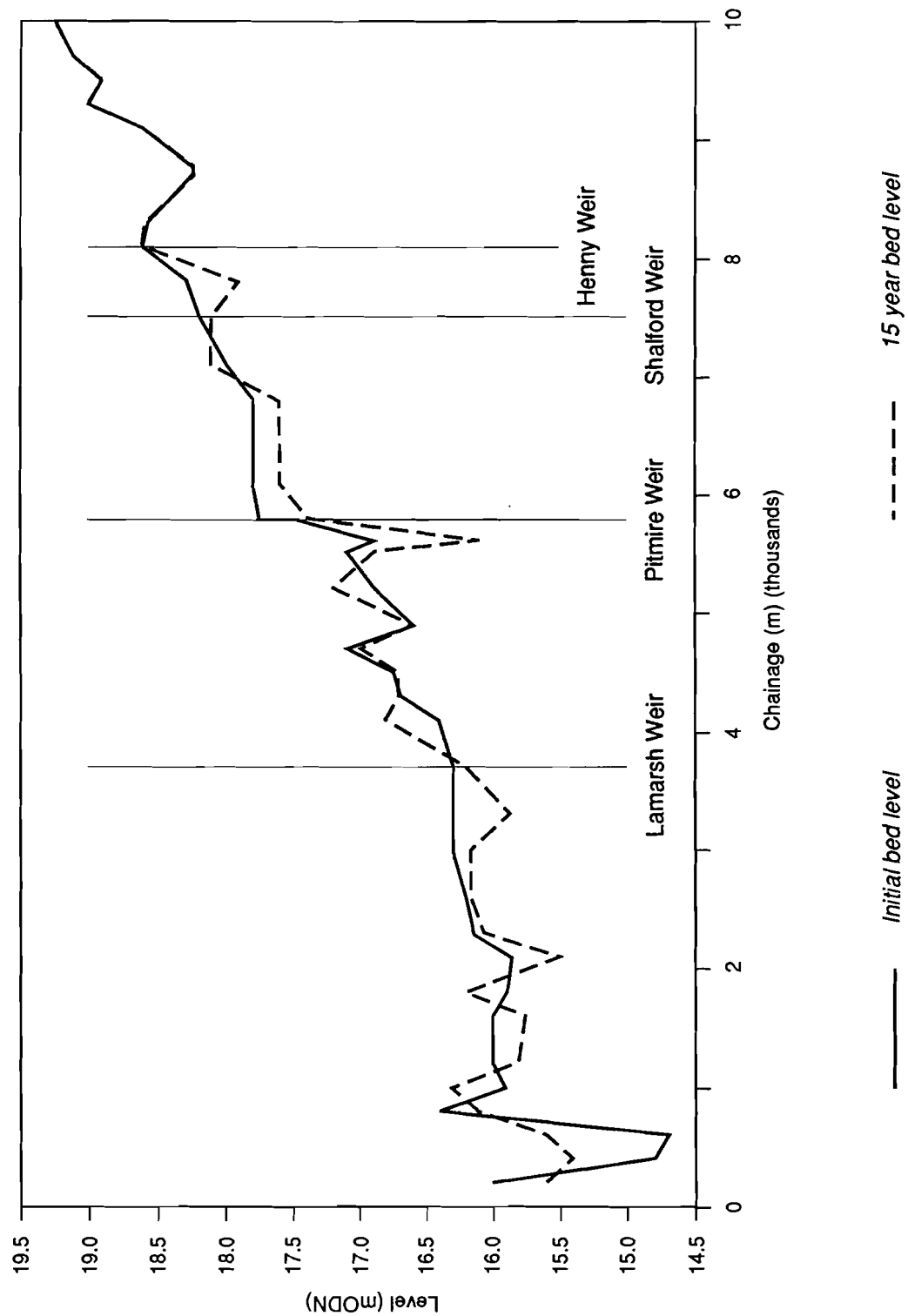
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Figure D5 Post improvement scheme: Bed levels



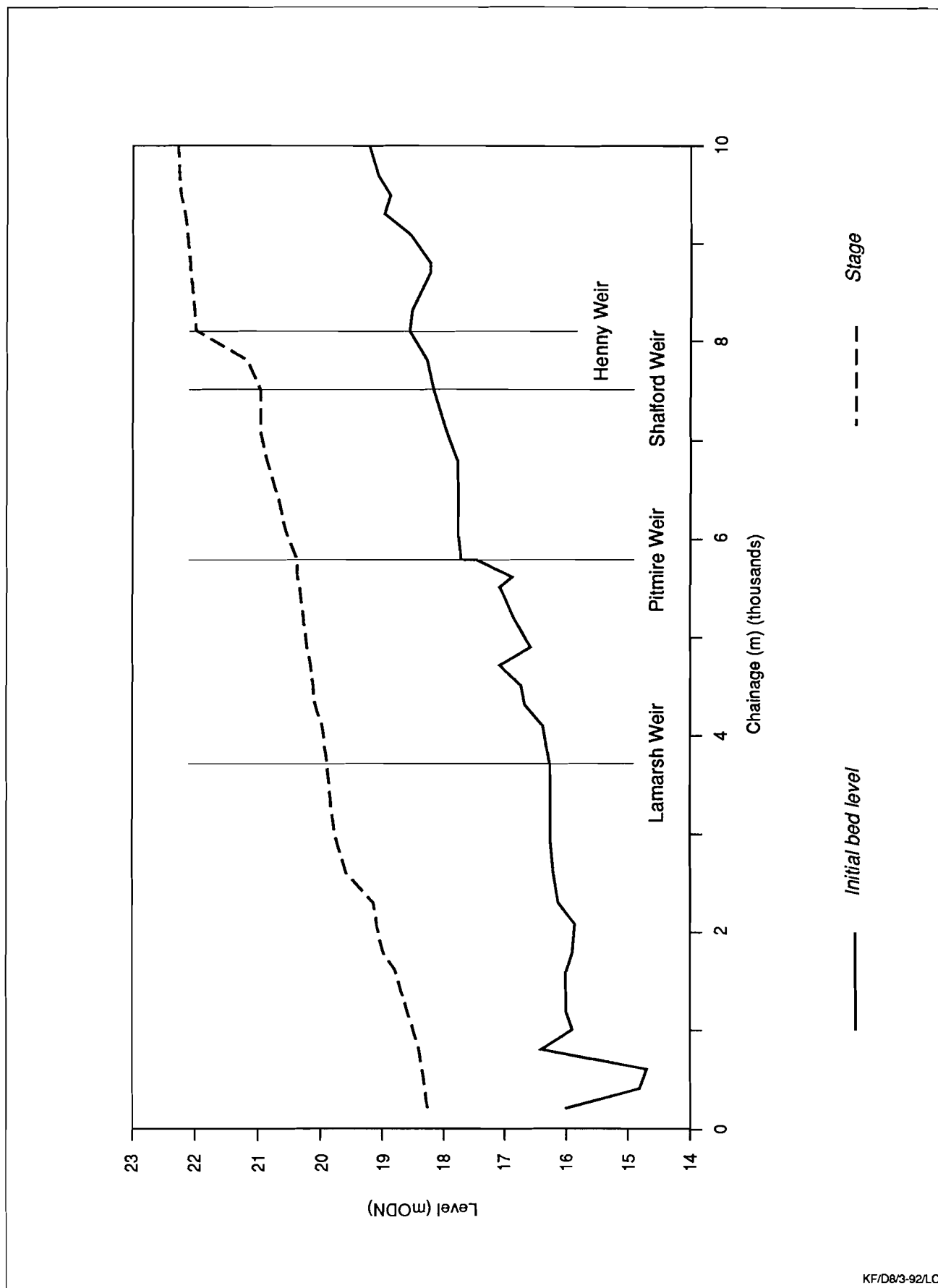
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Figure D6 Comparison of bed levels – pre and post improvement scheme



KF/D7/3-92/LO

Figure D7 Bed levels for modified scheme



KF/D&3-92/LO

Figure D8 Modified scheme – bed and water levels

Appendix E

Case Study : Duffield Flood Protection Scheme

E.1 Background

Duffield is on the River Ecclesbourne, Derbyshire approximately 1km upstream of its confluence with the River Derwent. In the mid 1970's a flood protection scheme was implemented along the river to protect Duffield against a 1 in 100 year flood. During the design phase of the scheme bank erosion problems were considered, and extensive bank protection was planned as part of the scheme, but erosion/deposition along the river bed were not taken into account.

E.2 Original layout

The pre scheme layout of the river reach is shown in Figure E1. The main river passed through an old mill complex at White House, where the side weir on the right bank controlled flow around the bypass channel. Downstream of White House, the main river channel passed through a second mill complex, via Mill House sluices. In the town centre there are two road bridges in close proximity, Town Street and Chapel Street, after which the channel narrowed and turned to follow the railway embankment. Turning through an 'S' bend, the river passed under the railway bridge before joining the River Derwent.

E.3 Post-improvement scheme layout

The post scheme layout is shown in Figure E2. In the upper part of the scheme the side weir controlling flow around Mill House bypass channel was removed and the channel realigned to meet the main river channel as it left the White House complex. A flood embankment was constructed between the new channel and the bypass channel to prevent flood waters flowing from one to the other. At Mill House, two new weirs were constructed to replace the old sluice gates and the main river channel, formerly the bypass channel, regraded to approximately 1 in 290. At the footbridge where the river flows alongside Snake Lane a new bridge and drop structure were constructed which control the water level in the upper reaches.

In the central part of the scheme the river is confined to a concrete flume with vertical side walls. The channel width upstream of the two road bridges has been doubled. The road bridges have been strengthened by underpinning. Immediately downstream of the bridges there is another short length of concrete flume. Further downstream there is a section of open channel in which a meander bend has been straightened followed by another short reach of concrete channel with sheet piling walls. The old footbridge has been replaced by a modern single span concrete bridge. In the vicinity of the railway bridge revetment work has been carried out to stabilise the banks and the channel has been regraded to 1 in 290.

The initial design of the concrete channel upstream of the road bridges contained a low flood wall down the centre of the channel which separated the channel into two low flow channels. The wall was demolished for aesthetic reasons although remains of the sheet piling can be seen at low flows.

E.4 Morphological problems

Since the scheme was implemented sediment has been accumulating in the region immediately upstream of the road bridges. Large deposits of material can be observed at the entrance to the left hand arch of Town Street bridge which, under low flow conditions, restricts flow to the right hand arch only. The deposition continues through the left hand arch and along the left hand bank between Town Street and Chapel Street bridges. Grass has grown on the surface of the deposited material which suggests that the deposition is well established and is a semi-permanent feature of the river channel. It is not known if the banks are eroded during high flow floods.

The deposited material reduces the flow capacity of the channel and decreases the effectiveness of the flood relief scheme. The deposited material is removed on a yearly basis when approximately one metres depth of material is removed (spread evenly across the channel) up to 80m upstream of the road bridges. This represents approximately 300-600m³ of material a year and such a maintenance commitment is both costly and time consuming.

Samples of sediment were taken from the river bed during a site visit and the grading curves from these samples are shown in Figure E3. The deposited material is a sand/gravel with a D₅₀ size of 2mm.

E.5 Model details

E.5.1 Cross-section data

A total of 24 cross-sections were used in the model, for both pre and post-scheme covering a 2.2km stretch of the river. At the upper end of the scheme the model was extended by 0.5km upstream of the upper limit of the scheme, the White House complex, to enable the sediment input to be modelled accurately. The sediment input was a function of the water surface slope but because there were draw-down effects due to the controlling weirs at White House this gave an over-estimation of the sediment input. By extending the model further upstream a more representative water surface slope was attained at the upstream boundary.

E.5.2 Pre-scheme representation of the bypass channels

One of the important features in modelling the river reach was to accurately represent the bypass channels and to assess their implication on the transport of sediment.

Under pre-scheme conditions the flow around White House was controlled by a side weir which was represented as a broad crested weir. The governing flow equation was given by:

$$Q = \frac{2}{3} H \sqrt{\frac{2}{3} g H}$$

Where H is upstream head

Flow down the main river channel was formerly controlled by a weir under the old bridge which had become damaged leaving only half the weir intact. A rating

equation was established for this section by considering it as a channel section of half the full width and a broad crested weir of half the full width.

These rating equations were used to calculate the stage at the upstream section of the bypass channel. The sediment transport through the White House complex was assumed to be down the main river channel; as the main weir had crumbled there was no restriction on its movement.

At Mill House complex a restriction on the sediment movement was assumed. No sediment was assumed to pass over the side weir and sediment transport through the sluice gates only occurred when the sluices were opened. Assuming that the sluices were operated to a constant upstream head sediment could only pass through the complex at discharges in excess of 2 cumecs. At discharges below this the upstream head was governed purely by the side weir which was modelled as a broad crested weir.

E.5.3 Post-scheme representation of the bypass channel

In the post-scheme conditions there is a single bypass channel. Water levels upstream are controlled by a combination of the side weir and the crest level at the drop structure. The 1 in 100 year flow division is designed to be one third down the main river channel and two thirds down the bypass channel.

The side weir has a vertical upstream face which was assumed to negate any sediment movement over the crest. All the sediment therefore passed down the main river channel and was assumed to pass through the drop structure.

E.5.4 Representation of bridges

In both the pre and post-scheme conditions five bridges were included in the model. Afflux tables for the bridges were calculated using the micro-FLUCOMP package which uses the USPBR method of calculating afflux which was specifically designed for use with box-girder type bridges in the US. The bridge affluxes were increased by a factor of two and this gave more satisfactory, and stable results during the calibration. For compatibility, this was retained for the post-scheme conditions.

E.5.5 Flow model calibration

Flow data for the River Ecclesbourne was presented as average daily flows over a 10 year period recorded at the Puss in Boots gauging station located 3km upstream of Duffield. From this data a flow exceedence curve was derived which had a peak discharge of 26 cumecs, see Figure E4. No stage/discharge measurements were available along the reach of interest so it was not possible to specify the tailwater conditions at the downstream section. It was known that the levels in the River Derwent caused a backing-up effect along the Ecclesbourne but without additional data on levels in the River Derwent this could not be taken into consideration. The downstream stage/discharge rating is shown in Figure E5.

Initial estimates of the Colebrook-White roughness length were between 0.1m and 0.2m although these were modified during calibration of the morphological model to give a roughness length of 0.6m.

E.5.6 Morphological model calibration

A single representative sediment size of 0.002m was used in the morphological model. Based on sediment samples taken during a site visit this corresponded to a D_{50} representative sediment size. Sediment input at the upper end of the model was taken as a function of the water surface slope and this resulted in a yearly sediment input of approximately 2.9 tonnes/km² of catchment area. Full equilibrium of sediment input/output balance could not be achieved although the balance was within 1% of the total volume transported into the model.

Running the model over a five year period indicated an accumulation of sediment at the head of the Mill House bypass channel and a net erosion in the reach from Mill House to the road bridges (see Figure E6). However, these transient bed conditions were relatively minor, less than 0.1m per year, and the results were considered acceptable for the purposes of this study. The calibrated values of sediment input, representative sediment size and roughness length were then used in the post scheme model. The latter of these, roughness length, would in fact vary longitudinally along the post scheme river conditions because of the introduction of concrete flumes. The surface roughness of the flumes would be lower than that in the natural channel. However, as sediment is deposited on the concrete beds the roughness will increase. A global roughness length of 0.6m was therefore retained because this represents the worst case condition for deposition.

E.6 Model results

E.6.1 Post improvement scheme conditions

The results from the post improvement scheme model are shown in Figure E7. An accumulation of sediment is to be found in the region immediately upstream of the road bridges which corresponds to the area in which the sedimentation problems exist in the prototype. The model is relatively stable throughout the rest of the reach although a large scour hole is predicted downstream of the railway bridge. During the site visit it was not possible to identify whether such a feature was present along the river bed. However, because it is downstream of the area of interest it has no influence on the build up of sediment at the road bridges and was therefore of little importance to the present study.

E.6.2 Assessment of morphological effects

When comparing the long term post improvement scheme results with the original bed profile it can be seen that the bed appears to be moving back to its original profile, see Figure E8. The majority of the sediment movement took place during the first year of operation whereas during the subsequent years the rate of change of the bed profile decreases and appears to be settling towards an equilibrium condition once more.

Comparison between the pre and post improvement scheme results suggests that more sediment is being transported through the river reach upstream of Mill House and due to the channel modifications is being deposited upstream of the road bridges.

The bed levels throughout the majority of the river reach are stable and fluctuate by relatively small amounts over the period of the simulation. Upstream of the road bridges however, the accumulation of sediment is continuous throughout this period. In the prototype there is a left hand bend in the channel at this point.

The deposited sediment would therefore collect around the slack water on the inside of the bend which is the condition observed during the site visit.

It appears that the sedimentation problems are a combination of large scale transportation effects due to the river works and also due to localised conditions which cannot be modelled using the present 1-dimensional model.

E.7 Alternative methods of alleviating problems

E.7.1 Probable causes of the sedimentation problems

An underlying assumption to this work is that the river was free from sediment problems under pre-scheme conditions. This assumption cannot be verified but no dredging of the river was reported by the Water Authority's staff.

The probable cause of the sedimentation problems were considered to be due to one or more of the following factors:

- modification of the channel dimensions
- regrading of the bed
- modifications to the bridge
- modifications to the control structures
- changes in sediment input conditions

E.7.2 Removal of central channel wall

The central channel flood wall was removed for aesthetic not hydraulic reasons. A simulation was undertaken to investigate the implications of this feature on the deposition of sediment.

It was assumed that the flow was split equally down each side of the channel. The results indicated that the deposition would be slightly worse in the region upstream of the road bridges because the depths of flow were increased thereby decreasing the flow velocities. A dual channel effectively increases the wetted perimeter for a given flow depth which results in a decrease in the hydraulic radius and overall increase in the friction function. The channel conveyance is therefore reduced and this results in an increase in the depth of flow.

E.7.3 Narrowing channel to its pre improvement scheme width

The result of narrowing the concrete flume channel to its original width is shown in Figure E9. The narrowed channel was introduced between chainage 600m to 1100m. Deposition immediately upstream of the road bridges is reduced although there is still a net accumulation of material. Most notably, there is little change in the accumulation at chainage 756m. At the point where the narrowed section ends, chainage 1100m, large amounts of material are deposited. A large scour hole is developed at chainage 1300m which is just downstream of the railway bridge. This feature was present in all the post-scheme runs. It is not known whether there is scouring in this region in the actual river reach but because it is well downstream of the reach of interest it is not an important feature with regards to this study.

E.7.4 Introduction of two stage channel

An alternative to a narrowed channel is to introduce a two stage channel. During low flows the flow is restricted to the main channel and only during high flood

flows does the water spill out on to the flood berms on the second stage of the channel. Flow velocities are kept relatively high whilst the flow is in the low flow channel but the overall channel retains its capacity to pass flood flows.

A two stage channel was introduced from chainage 600m to 1100m. The low flow channel was approximately 3m wide and 1.25m in depth. The channel then opened out to its full width as given by the post-scheme design. The variation in bed levels over a five year period are given in Figure E10 and show that there is very little deposition of material upstream of the road bridges. The channel is also found to be very stable over this time period.

There are two disadvantages to this scheme. Firstly, there is a considerable amount of deposition where the two stage channel feeds back in to the original channel, at chainage 1100m. The deposition problems have therefore only been transported to a location further downstream. Clearly it would be possible to design a channel which could carry the sediment out of the River Ecclesbourne but this would only transfer the problems to River Derwent. Secondly, the two stage channel reduces the flood flow capacity of the River Ecclesbourne unless the bed levels were sufficiently lowered to account for the reduced flow area.

E.7.5 Widening channel by 2m

The impact of increasing the width of the channel by 2m immediately upstream of the road bridges is shown in Figure E11. The results indicate that deposition of material would be reduced in the region of the road bridges although a slight scour hole is developing downstream of the bridges at chainage 875m. The combination of these two factors suggests that the sediment transport is comparable at each of the widened cross-sections but that it has been reduced from the actual post-scheme channel design. A reduction in the amount of deposition when the channel is of uniform width reinforces the view that the deposition problems in the post-scheme layout are due to a sudden change in channel width.

E.7.6 Effect of bridge affluxes

In the region where the sedimentation problems exist, upstream of the road bridges, there is an expansion of the channel width to approximately twice its original width. This causes a fall in the flow velocities and a reduction in the transport capacity of the channel which results in sediment being deposited. This situation could also be made more acute if water levels upstream of the bridges are increased due to the bridge affluxes thereby reducing the flow velocities further. In order to investigate the importance of the bridge structures a run was undertaken in which the downstream bridge was removed. The results, Figure E12, showed that the deposition remained of the same order thereby indicating that the bridge affluxes are not a major factor in the deposition process in the post scheme design.

E.8 Conclusions and recommendations

- 1 The main cause of the deposition of material was a significant widening of the channel in the area upstream of the road bridges.
- 2 The sedimentation problems can be eased by replacing the design channel with either a narrowed section channel or a two stage channel.

- 3 Due to the constricted channel space through Duffield, undertaking either of the options given above will increase flood water levels beyond acceptable limits but at other sites this may prove to be a workable solution.
- 4 The bridge affluxes only become important features when the downstream water levels are significantly increased.

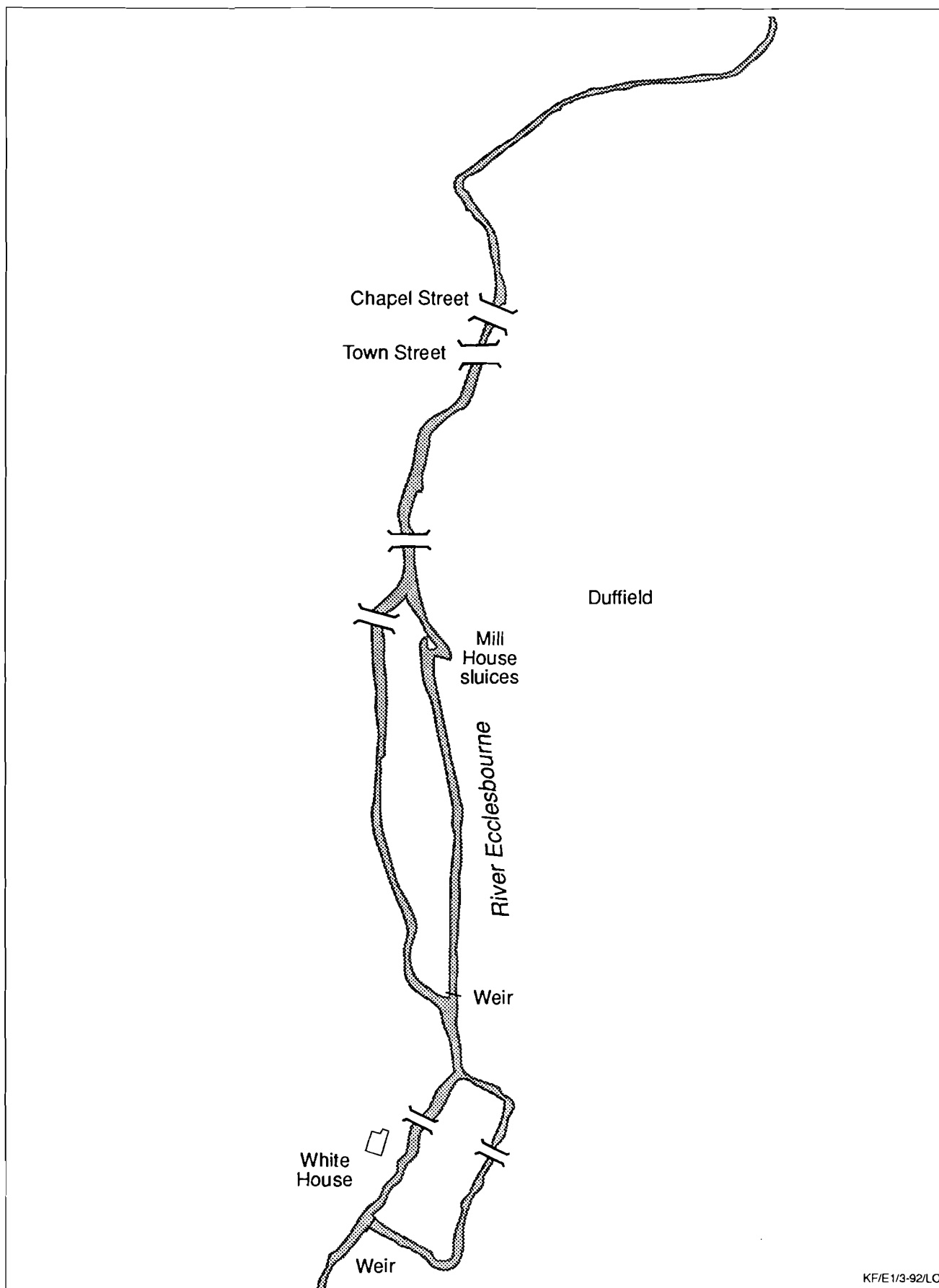


Figure E1 Duffield: River Ecclesbourne – original layout

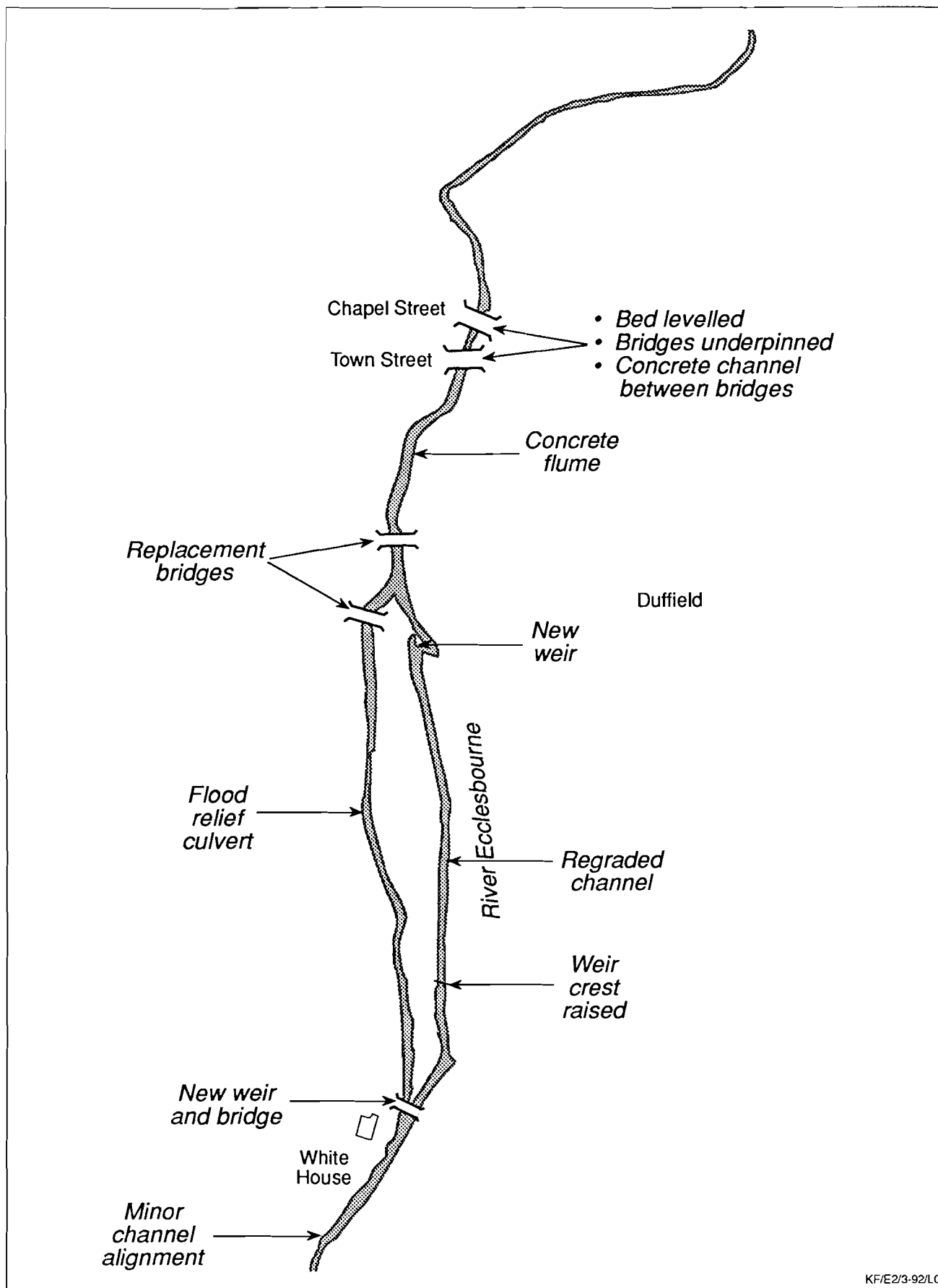
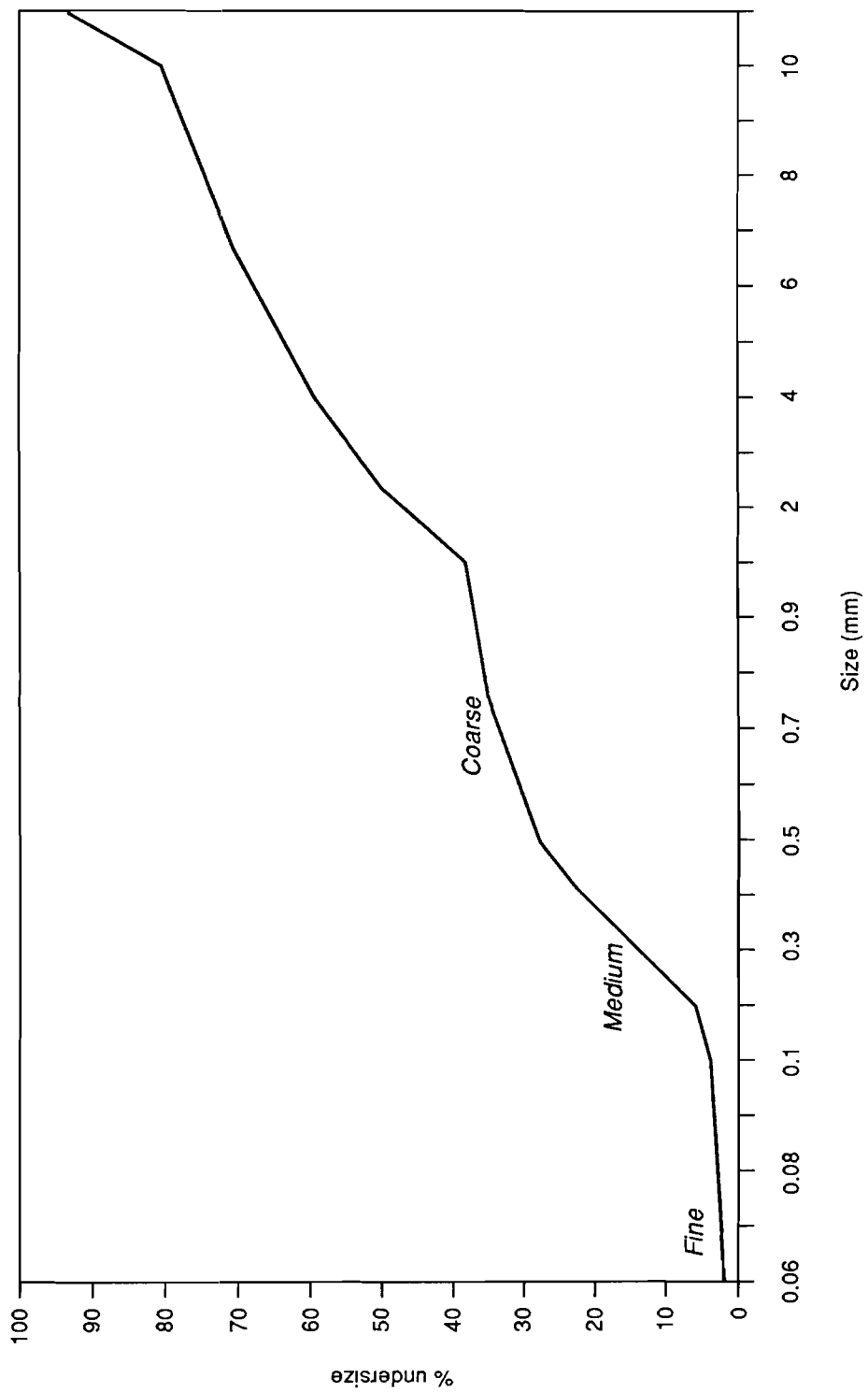
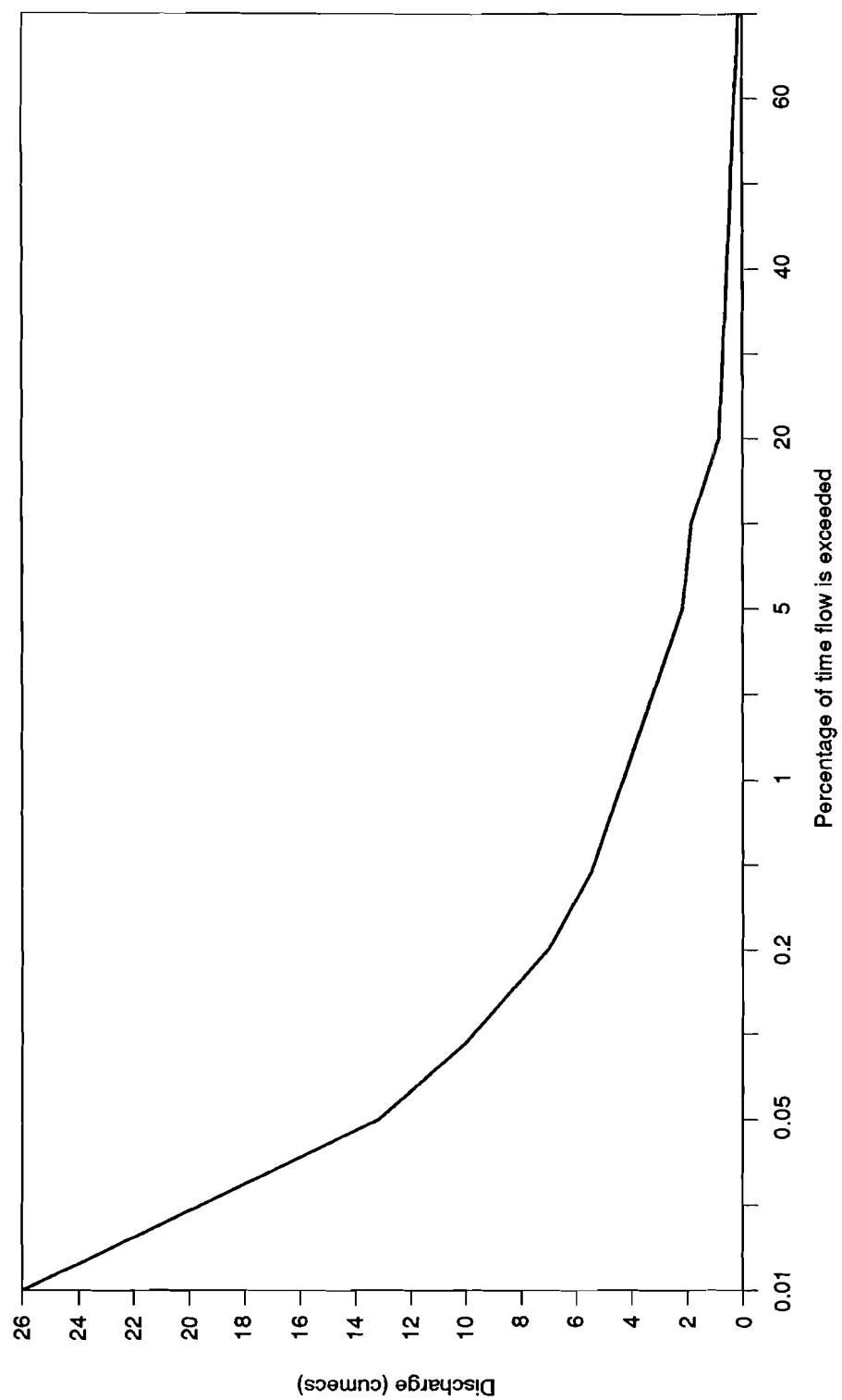


Figure E2 Duffield: River Ecclesbourne – post-improvement scheme layout



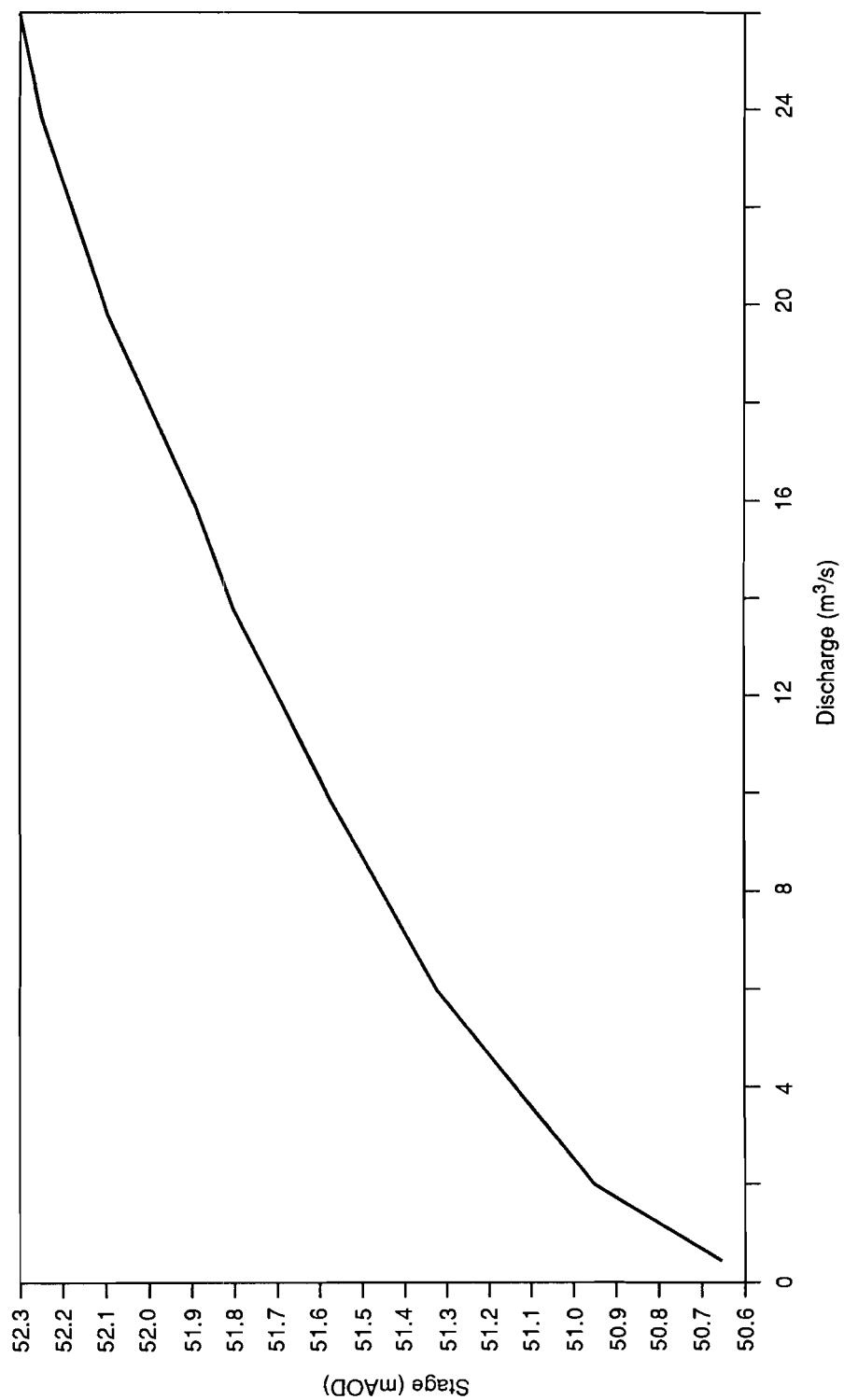
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Figure E3 Sediment grading



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Figure E4 Flow exceedence curve



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Figure E5 Downstream stage/discharge rating

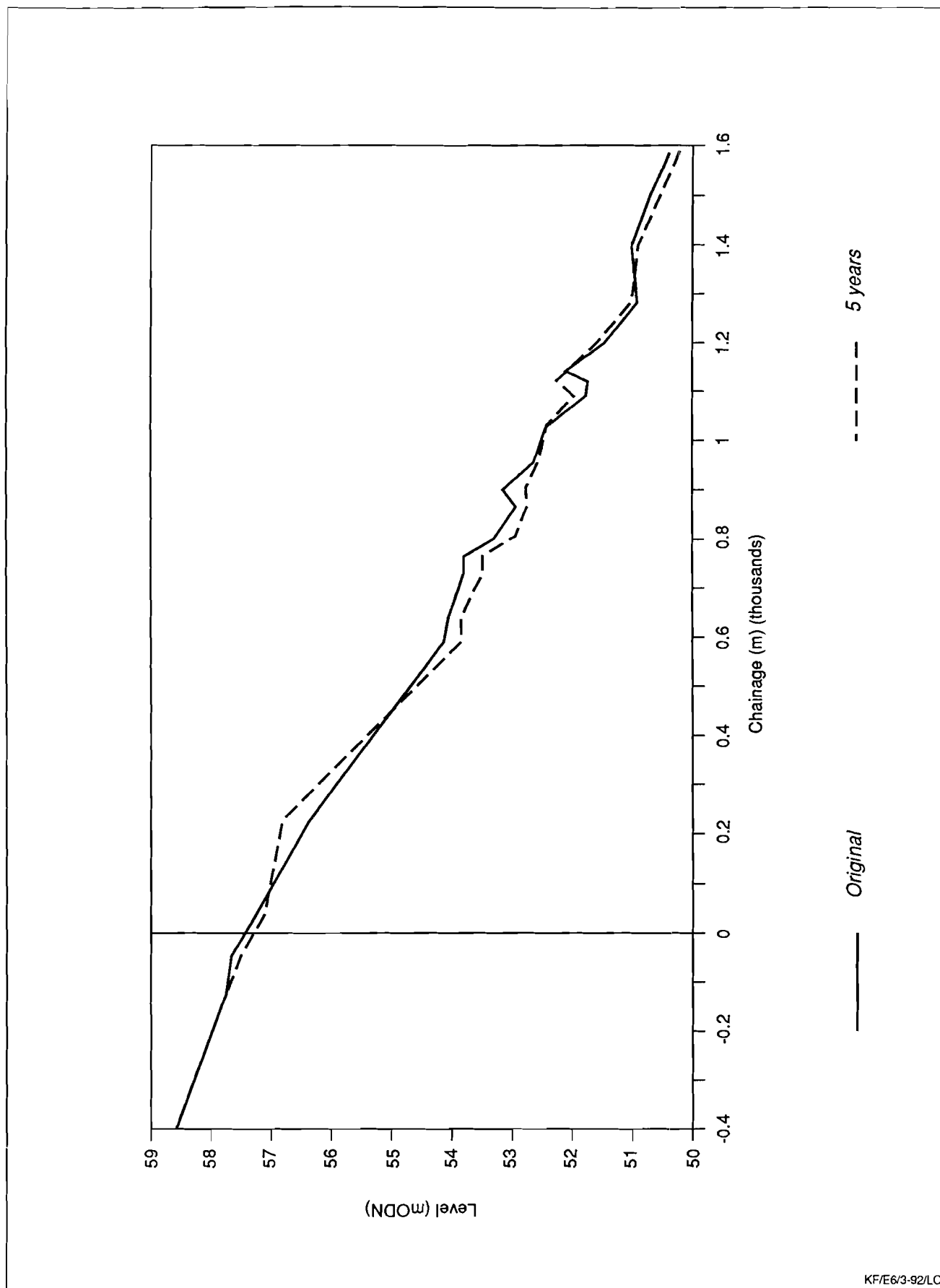
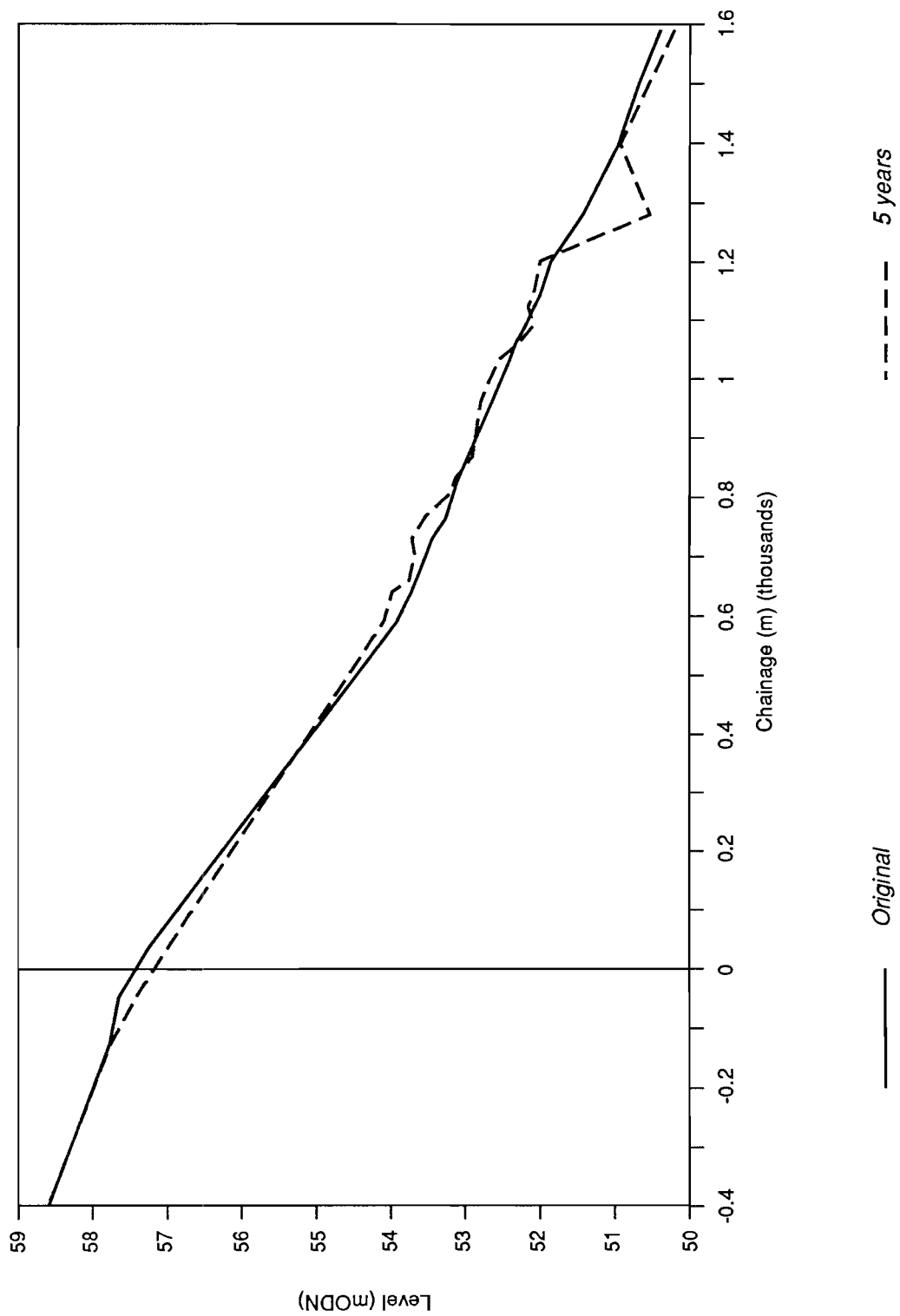
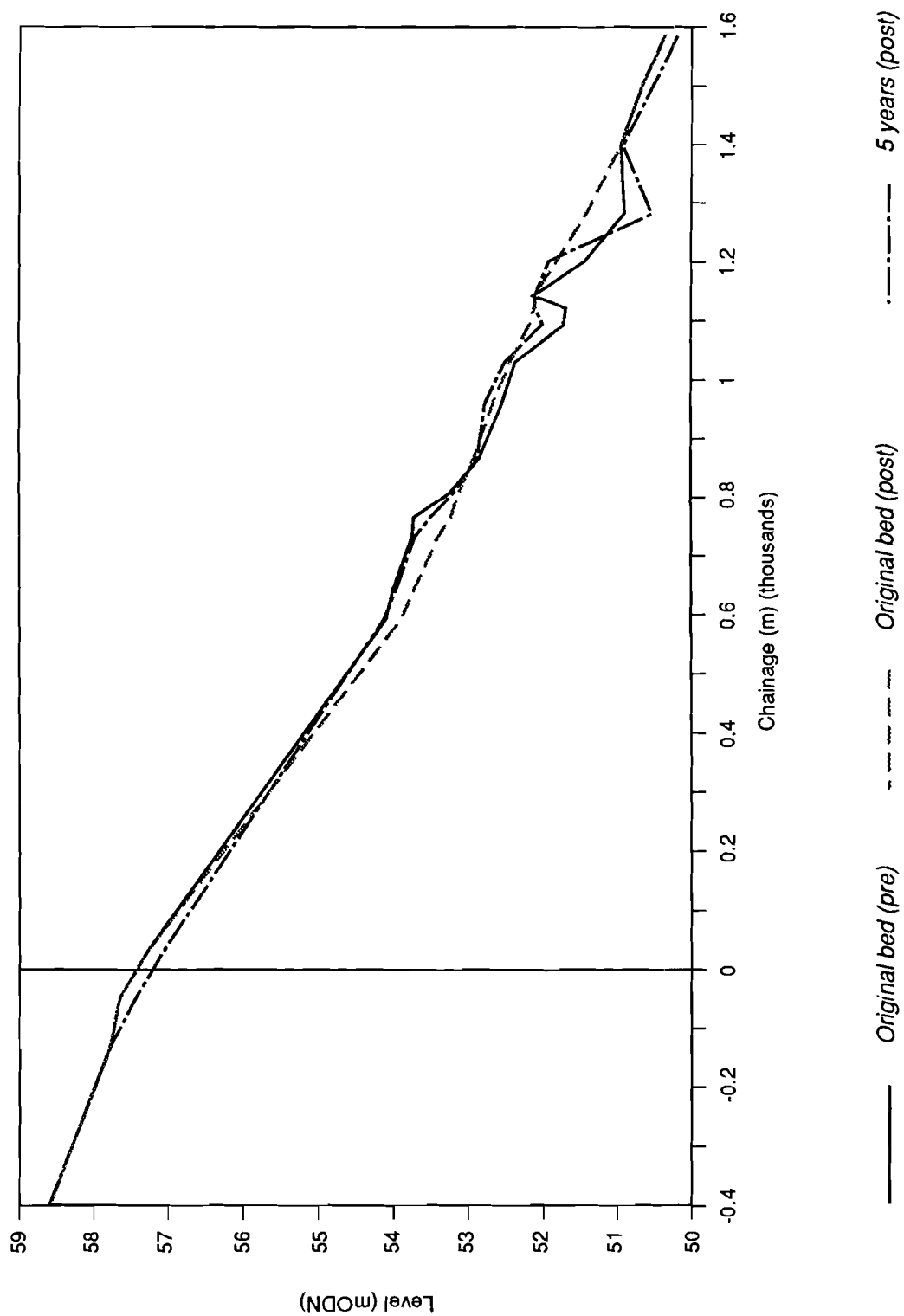


Figure E6 Pre-improvement scheme: Bed levels



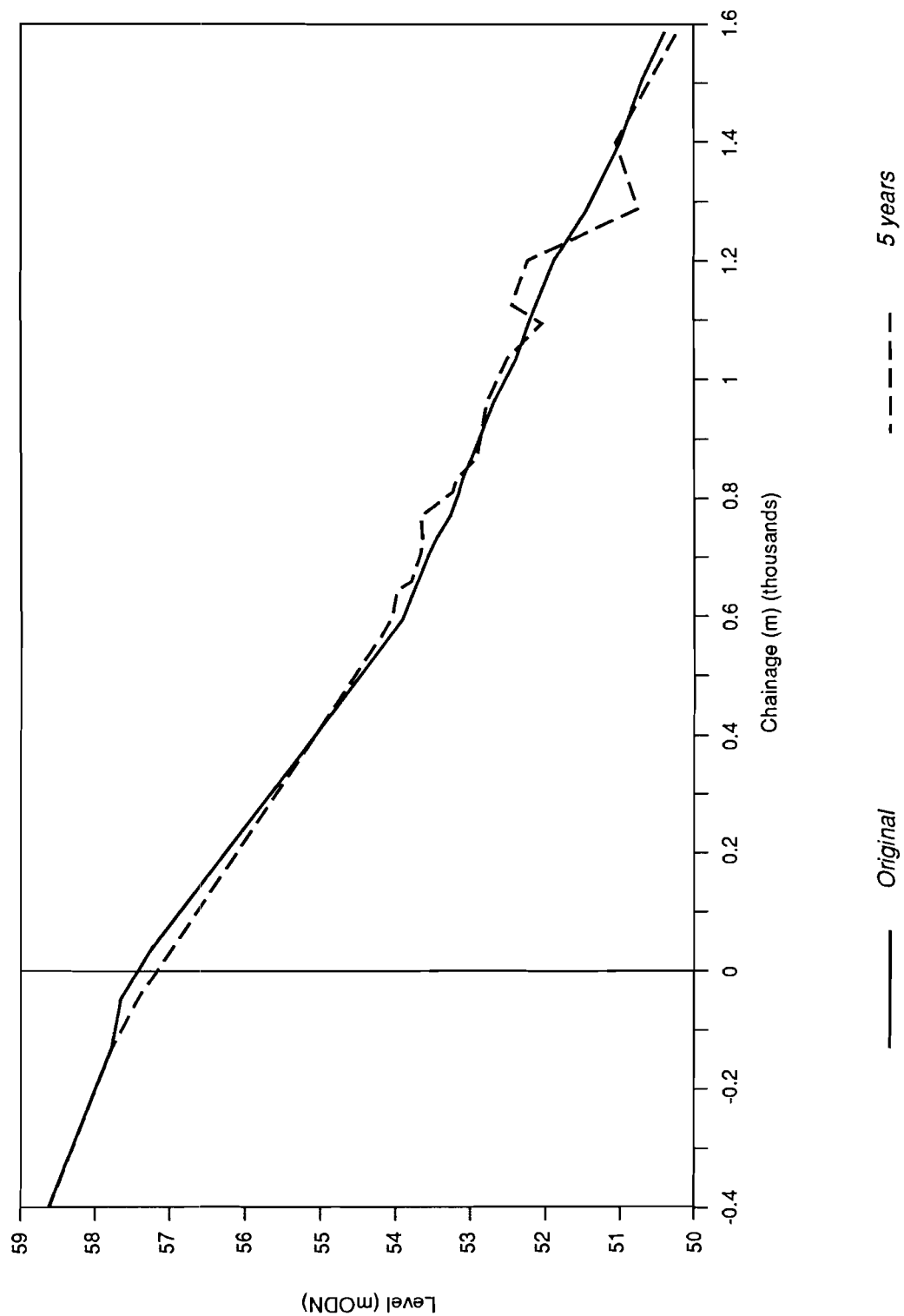
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Figure E7 Post improvement scheme: Bed levels



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Figure E8 Comparison of pre and post scheme bed levels



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Figure E9 Modified scheme bed levels for a narrower channel

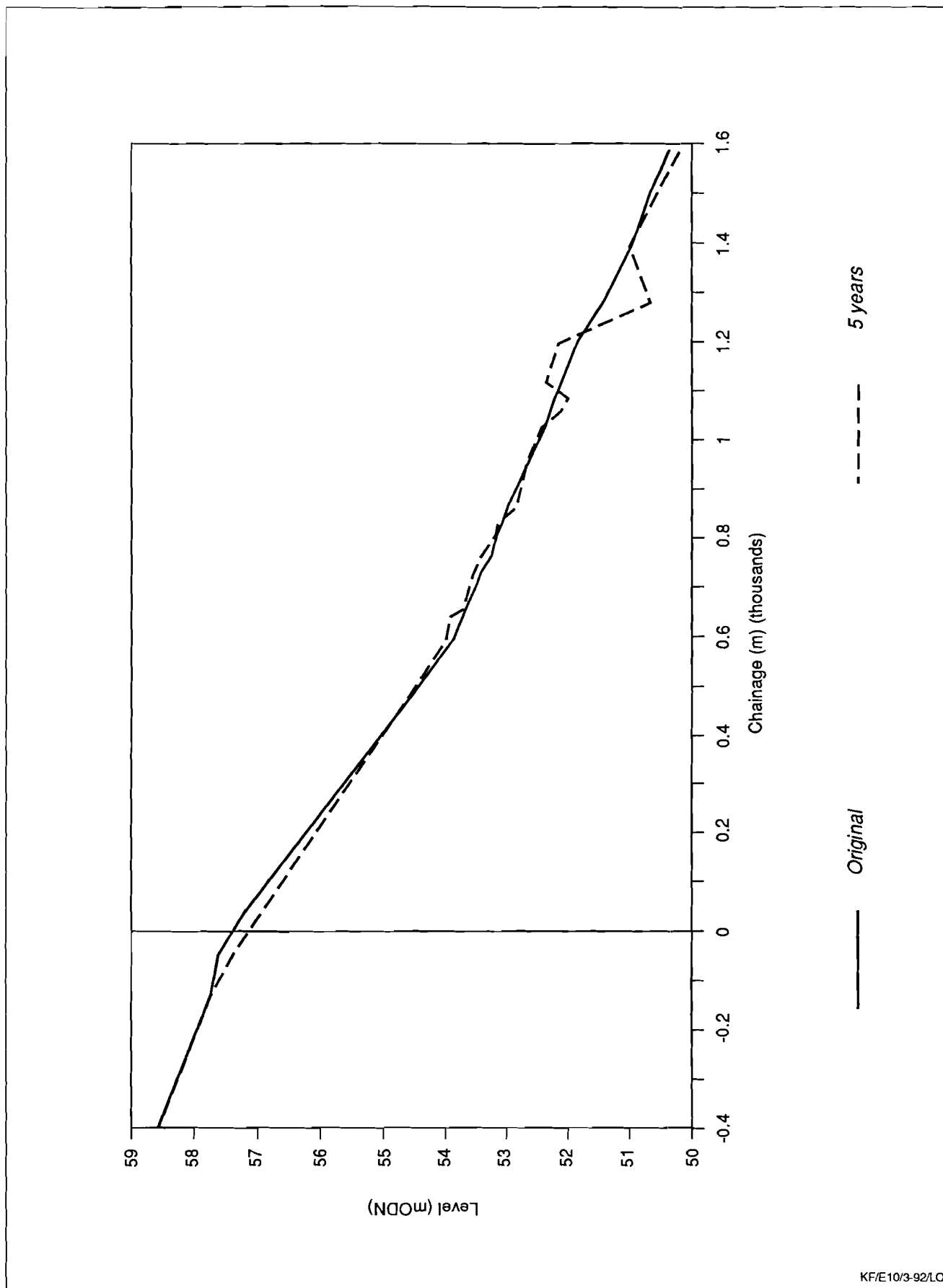
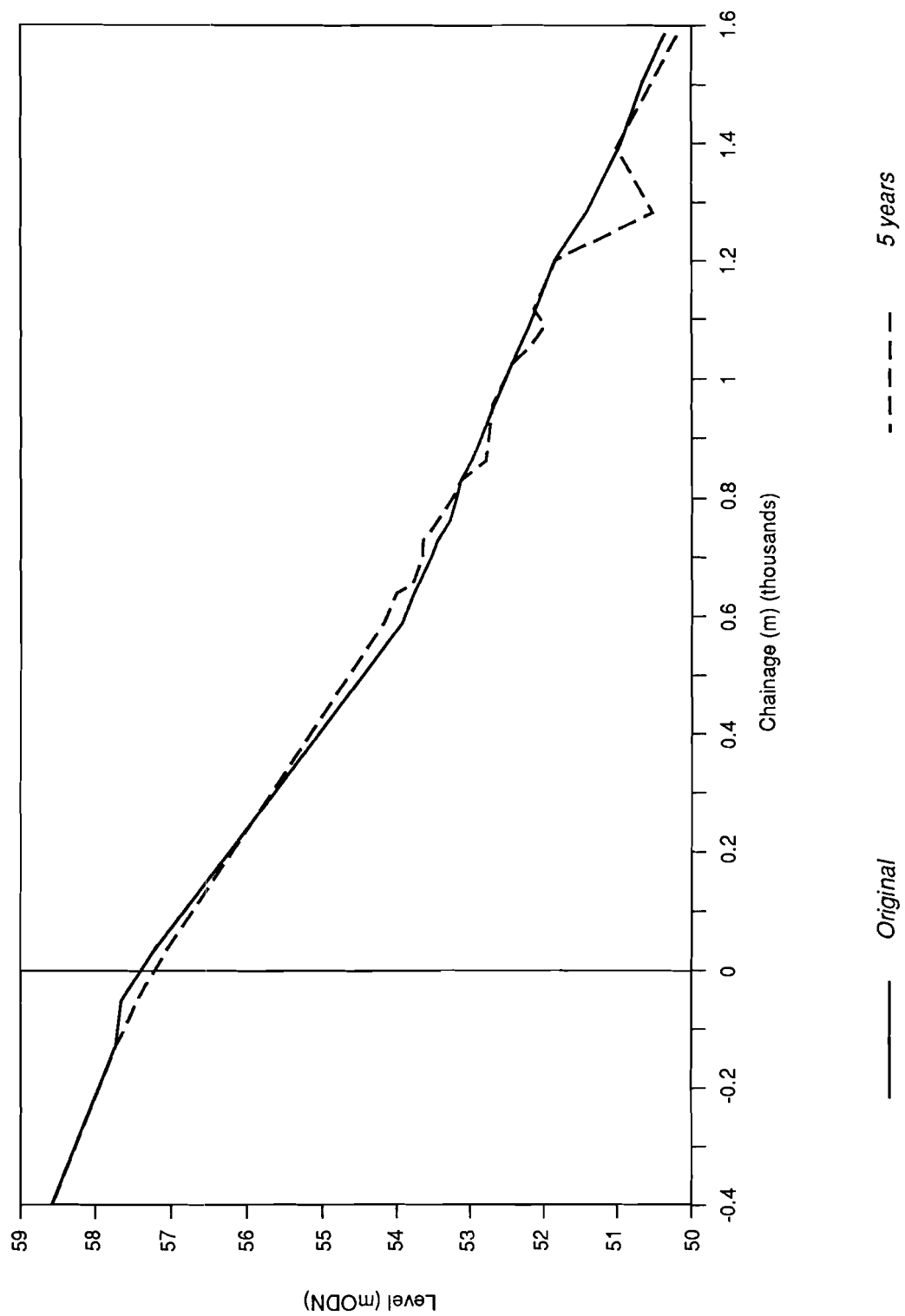
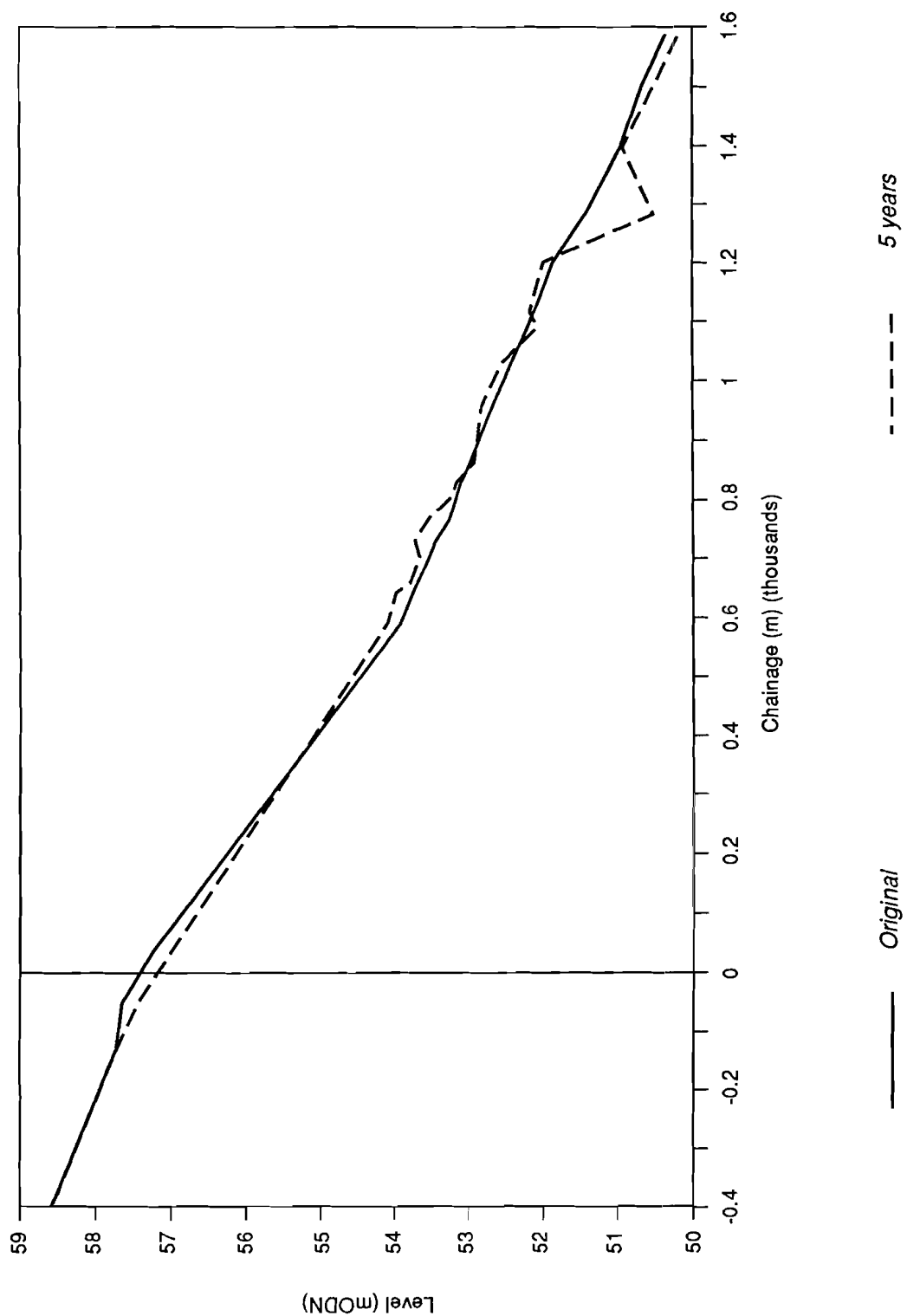


Figure E10 Modified scheme bed levels – two stage channel



KF/E11/3-92/LO

Figure E11 Modified scheme bed levels – wider channel



KF/E12/3-92/LO

Figure E12 Modified scheme bed levels – effect of bridge affluxes

Appendix F

Case Study : Brecon Improvement Scheme. River Usk

F.1 Background

A flood in December 1979 caused extensive flooding in Brecon. This flood had a discharge of 610 cumecs with a return period of 45 years. The existing flood defences would only provide protection against a 15 year flood with a discharge of 490 cumecs. An improved scheme was designed to reduce the frequency of flooding to once every 100 years with a peak discharge of 685 cumecs. The improved scheme design made considerations for erosion and deposition.

F.2 Pre-improvement scheme

The River Usk flows through the centre of the town of Brecon. Two major tributaries enter the River Usk at Brecon; the River Honddu from the north which joins the main river channel immediately upstream of the seven arch bridge, Llanfaes or Usk bridge, in the centre of town; and the Afon Tarell from the south which joins the river about 400m upstream of the bridge. Immediately upstream of the Afon Tarell confluence is a weir across the river which was constructed to divert water for the Monmouth and Brecon canal. The stretch of river between the weir and the bridge is the key to the flood problem in Brecon. On the left bank of the river the ground is relatively high although the roads adjacent to the river are troubled by flooding. The major problem area during floods is the suburb of Llanfaes, on the left bank of the River Usk and the Afon Tarell.

F.3 Post-improvement scheme

The scheme construction in Brecon included regrading of the river bed upstream of the Llanfaes bridge to produce uniform bed slopes. This involved widening of the channel just upstream of the Llanfaes bridge and deepening the channel in some places. The invert of the Llanfaes bridge was lowered by 0.75m in order to achieve the required hydraulic gradients upstream and downstream of the bridge. The bridge was strengthened by providing sheet piling around the perimeter of the piers and abutments and infilling with concrete. Some of the existing flood walls were heightened by 0.8m in areas which were particularly susceptible to flooding both upstream and downstream of the Llanfaes bridge. Flood embankments were constructed on the right bank of the River Usk downstream of the Llanfaes bridge. The post-improvement scheme suggested in the feasibility study included widening of the channel downstream of Llanfaes bridge although this was not carried out.

F.4 Morphological problems and maintenance

The changes in cross-sections upstream of the Llanfaes bridge cause little change in water level but deposition of sediment in the reach from the weir to Llanfaes bridge is a problem. The changes to Llanfaes bridge narrowed the bridge opening giving a greater afflux than pre-improvement scheme conditions and causing deposition upstream of the bridge. No problems are identified downstream of the bridge. The sediment deposited upstream of Llanfaes bridge is mainly gravel with a D_{35} size of approximately 10mm upstream of the bridge (Fig F1) and 40mm downstream of the bridge. The source of this gravel is unknown but it is thought that during high flows gravel is transported across the weir and carried downstream during low flows. Gravel input from the two tributaries, River Honddu and Afon Tarrell, could be another possible source.

The gravel shoals formed in the channel upstream of Llanfaes bridge cause a significant reduction in the design capacity of the channel. Gravel has been removed from the channel both upstream and downstream of the Llanfaes bridge twice since 1979. On each occasion, 5-8,000 tonnes of gravel were removed at a cost of £40,000. Small groynes were constructed in an attempt to prevent shoal formation however these shoals cause the flow to be deflected against unprotected banks and revetment work had to be carried out to counter the resulting bank erosion.

F.5 Model details

F.5.1 Model data

The cross-section data used for the model were taken from the report on the feasibility study undertaken by Sir M Macdonald and Partners. The flow exceedence data for the Usk, Tarell and Honddu were supplied by Welsh Water (Fig F2) and data for high return period floods were taken from the Flood Studies Report (FSR). The roughness of the river channel was represented by a Ks value of 0.15m.

The afflux of the Llanfaes bridge was calculated using a theoretical method. Two methods are available: US method (in FLUCOMP); and HR method (SR 60). Both afflux methods gave the similar results: US method afflux = 0.47m; HR method afflux = 0.57m. In order to gain a more accurate representation of the bridge, the bridge was modelled as a broad crested weir. A range of Cd values tried. The best fit for the available data was a Cd value of 0.83.

The rating curve at the downstream limit of the simulated reach is shown in Figure F4.

F.5.2 Model calibration

There is no sediment data available for the river before the improvement scheme was implemented and the model calibration was carried out using the cross-section and discharge data. The upstream sediment input was calculated using a bed sample D_{35} size of 10mm upstream of Llanfaes bridge and a D_{35} size of 40mm downstream of Llanfaes bridge. 100% of the bed material is assumed to be mobile. The input concentration of sediment can be calculated in several different ways: from the water slope; as a constant concentration over the range of discharges; or from a site specific equation. Calculating the concentration of sediment input from the water slope gave a good fit with the existing data for the pre-improvement scheme condition. The concentration of sediment input at the upstream limit was determined by using an equation so as to simulate the correct sediment input over a range of flows (Fig F5). The equation used is given by:

$X = 0$ when $Q < 39$ cumecs

$$X = \frac{(129 \times \log(Q) - 204)}{(1 \times 10^6)} \quad \text{when } Q > 39 \text{ cumecs}$$

The flow model was calibrated against the flood which occurred in 1979 at Brecon. For a 100 year flood the discharges were obtained by multiplying the discharges for the 1979 flood by a factor, thus raising the discharge at Llanfaes

bridge from 610 cumecs to 685 cumecs and at the upstream limit of the Usk to 526 cumecs, the Tarell to 90 cumecs and the Honddu to 69 cumecs.

F.6 Model results

F.6.1 Pre-improvement scheme

The model was run for the pre-improvement scheme over a simulation period of 5 years. The concentration sediment input was determined by the equation given above but tests were also carried out using a constant sediment input and a sediment input determined by the water slope. The results for pre-improvement scheme tests are shown in Figure F6. In both these cases after one year the results were very similar to results when the sediment input was determined by the water slope.

For the 100 year flood event the results as seen in Figure F7, show water levels which are greater than the 1979 flood event both upstream and downstream of the bridge.

F.6.2 Post-improvement scheme

The channel upstream of Llanfaes bridge was widened, deepened and regraded in the post-improvement scheme. These upstream changes would cause only a minimal change in the water levels upstream of Llanfaes bridge for a flood of the same magnitude as the 1979 flood.

The sediment input at the upstream limit was determined by the water slope. The model shows that deposition occurs upstream of the Llanfaes bridge for both pre and post-improvement scheme conditions. After a simulation time of five years, the deposition for the post-improvement scheme is still increasing from year to year whereas for the pre-improvement scheme situation a state of equilibrium has been reached after 5 years, with no more significant sediment deposition. Deposition of sediment occurs upstream of the bridge for the post-improvement scheme with a total of 4,500 tonnes of sand deposited after 10 years (approximately half of this is deposited in the first year). Figure F8 shows the old and new bed levels both originally and after a simulation period of 10 years.

Llanfaes bridge was modified for the post-improvement scheme and has a much lower invert than the original bridge. For the post-improvement scheme situation, the bridge was no longer represented as a weir and the HR method was used to calculate the afflux. The bridge changes cause deposition at and upstream of the Llanfaes bridge site.

F.7 Modified schemes

Work was carried out to determine if the post-improvement scheme problems of deposition in the reach upstream of Llanfaes bridge could be improved by altering some sections either upstream or downstream of the bridge or both. At this stage no changes to the bridge itself were considered.

F.7.1 Modified scheme 1: Post-improvement scheme plus changes to sections downstream of Llanfaes bridge

The original feasibility study suggested changes to sections upstream and downstream of Llanfaes bridge in addition to bridge alterations. The downstream

modifications were added to the model and some of the morphological results are shown in Figure F9. There is deposition immediately upstream and downstream of Llanfaes bridge and some upstream erosion. For upstream conditions these modifications are an improvement on the post-improvement scheme without the downstream changes. Figure F10 shows the water levels for a flood similar to the 1979 flood experienced at Brecon. The water levels are significantly lower than those observed during the 1979 flood with the exception of the downstream reach of the model where the water levels show a slight increase.

F.7.2 Modified scheme 2: Post-improvement scheme plus changes to sections downstream minus changes made upstream of Llanfaes bridge

The changes made in the post-improvement scheme to the upstream sections were removed for this modification and changes made to the downstream sections as described in the feasibility report. The effect of these changes after 5 years is deposition immediately upstream and downstream of the bridge and some erosion at the upstream limit of the model. However this deposition is less than in the post-improvement scheme condition and less upstream of Llanfaes bridge than for modified scheme 1. Water levels which would occur with these modifications under flood conditions are similar to those experienced in 1979 at Brecon.

F.7.3 Modified scheme 3: Replacing section immediately downstream of the Llanfaes bridge

For this modification, the section immediately downstream of the bridge was replaced by the section recommended in the feasibility report. Some of the sections upstream of the bridge were narrowed by 5m. This modification resulted in less deposition at the narrower sections but worse deposition immediately upstream of the bridge. Water levels upstream of the bridge for a flood event similar to 1979 flood event at Brecon would be reduced but are higher in comparison to water levels achieved if all the downstream bed changes were carried out.

F.7.4 Modified scheme 5: Replacing Llanfaes bridge

One option which was considered for the purpose of these tests was to replace Llanfaes bridge by a single span bridge. For the purposes of modelling a single span bridge with invert well clear of flood waters can be ignored as it has no effect on water levels. For the pre-improvement scheme replacing the bridge causes the simulated flood water levels to be reduced upstream of the Llanfaes bridge for a flood similar to the 1979 flood. Immediately upstream of the bridge there is deposition and then erosion at the bridge section. For the post-improvement scheme situation replacing the existing bridge would give simulated water levels for a flood similar to that in 1979 which are lower than the actual 1979 flood levels. After a simulation over five years the bed levels with a new bridge were very similar to the original post-improvement scheme situation but there is less deposition of sediment upstream of Llanfaes bridge.

F.8 Conclusions and recommendations

Four schemes were suggested to modify the post-improvement scheme that was constructed. These were:

- Post-improvement scheme plus changes to sections downstream of Llanfaes bridge.
- Post-improvement scheme plus changes to sections downstream minus changes made upstream of Llanfaes bridge.
- Narrowing sections upstream of Llanfaes bridge and replacing section immediately downstream of bridge.
- Replacing Llanfaes bridge.

The advantages and disadvantages of these schemes are given in comparison to the improvement scheme constructed. In summary we can say that changes could be made to the present situation to prevent the same volume of sediment being deposited. Bearing in mind that the improvement scheme was originally designed to lower flood water levels any modified scheme which did not satisfy that criterion would be unacceptable. On that basis the modified scheme 3 would be unacceptable. Modified schemes 1 and 2 would both retain flood water to the same degree as the present improvement scheme and there would be less deposition at the Llanfaes bridge site and upstream than in the present situation. Modified scheme 4 would reduce the amount of deposition upstream of the Llanfaes bridge site but is probably unacceptable due to cost and objections from local residents. From the schemes tested, modified scheme 1: changes to the downstream sections, seems to be the most effective solution to reducing deposition of sediment at and upstream of Llanfaes bridge.

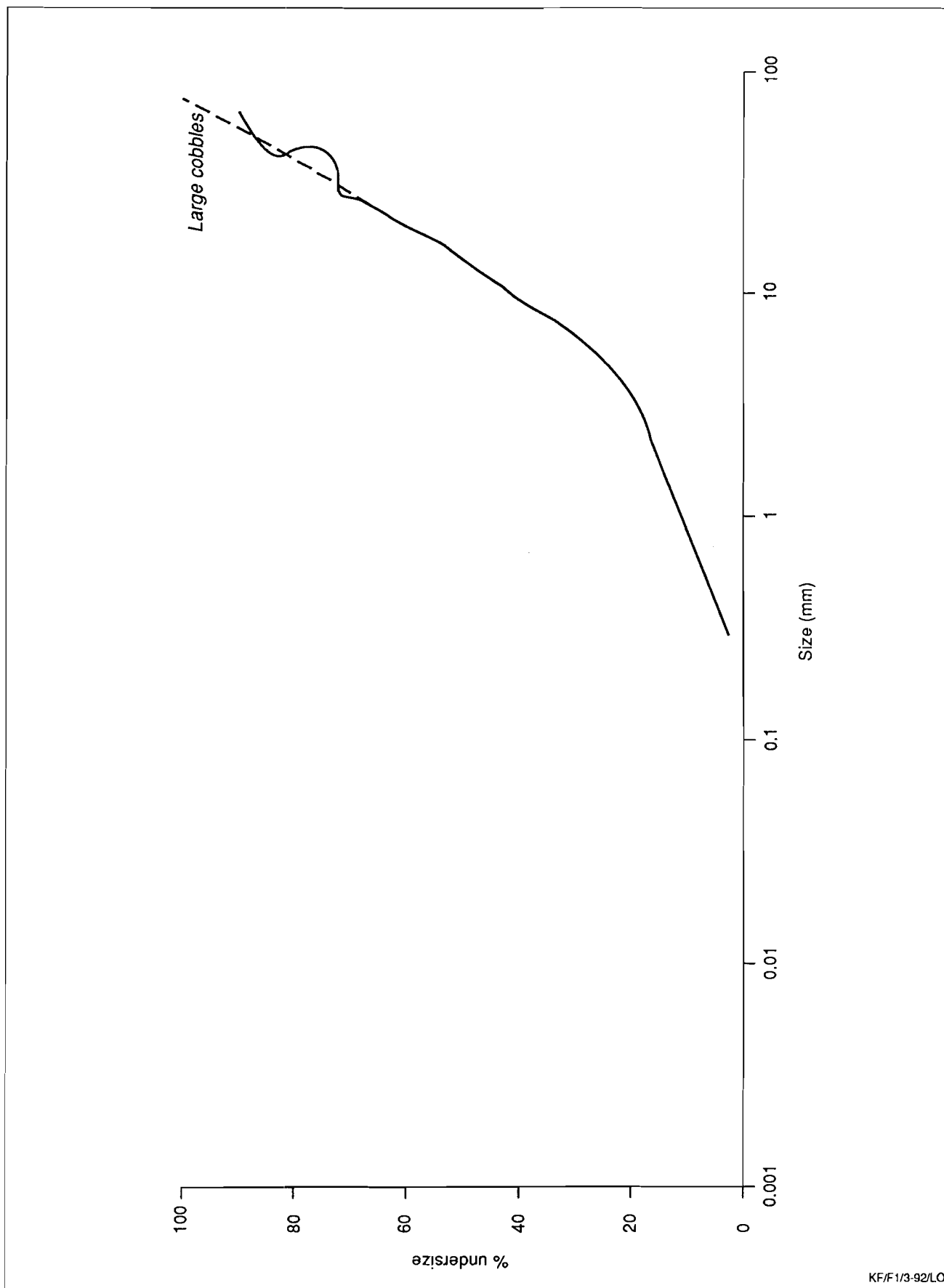


Figure F1 Sample of sediment upstream of Llanfaes bridge

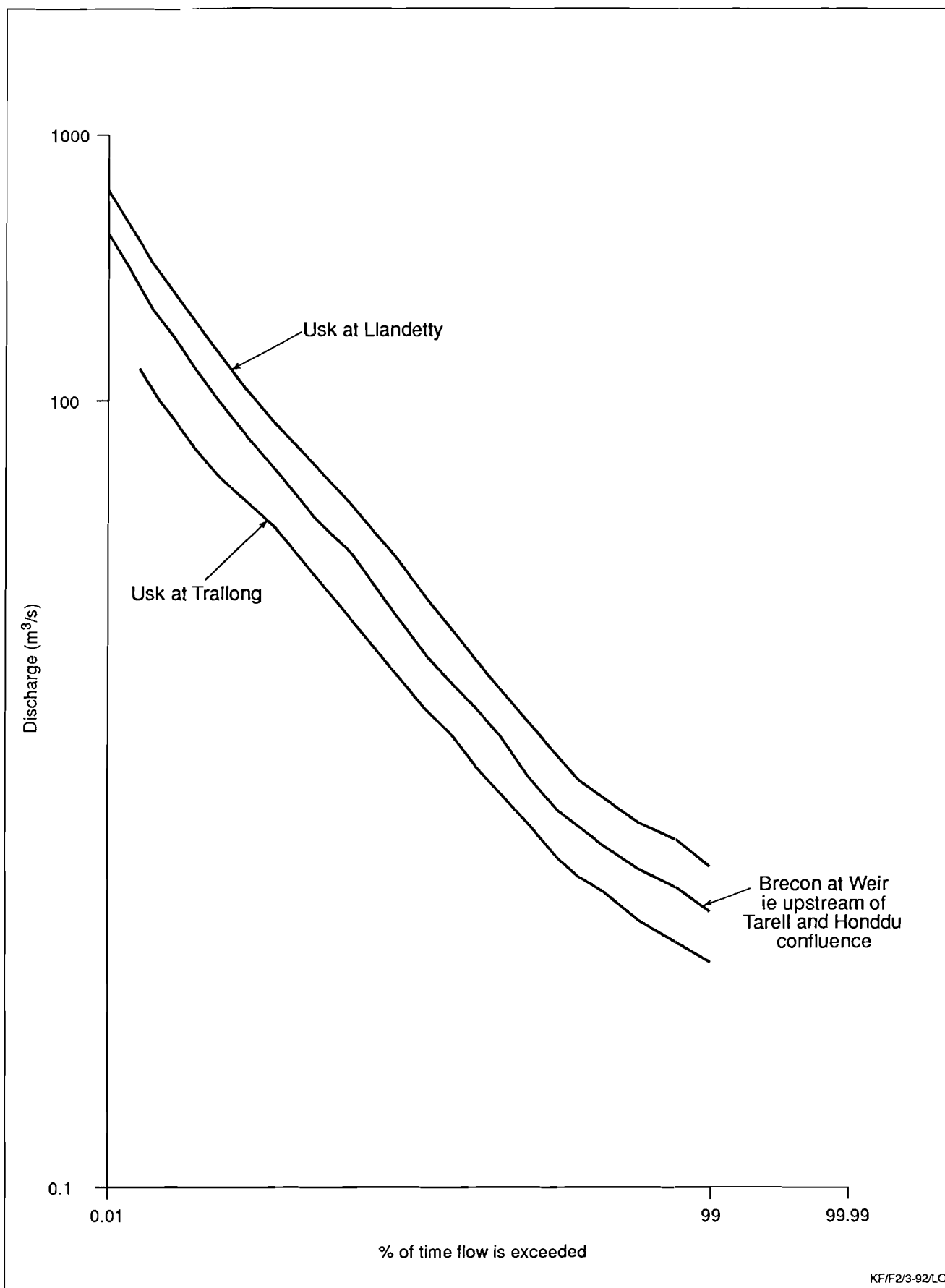


Figure F2 Flow exceedence curves

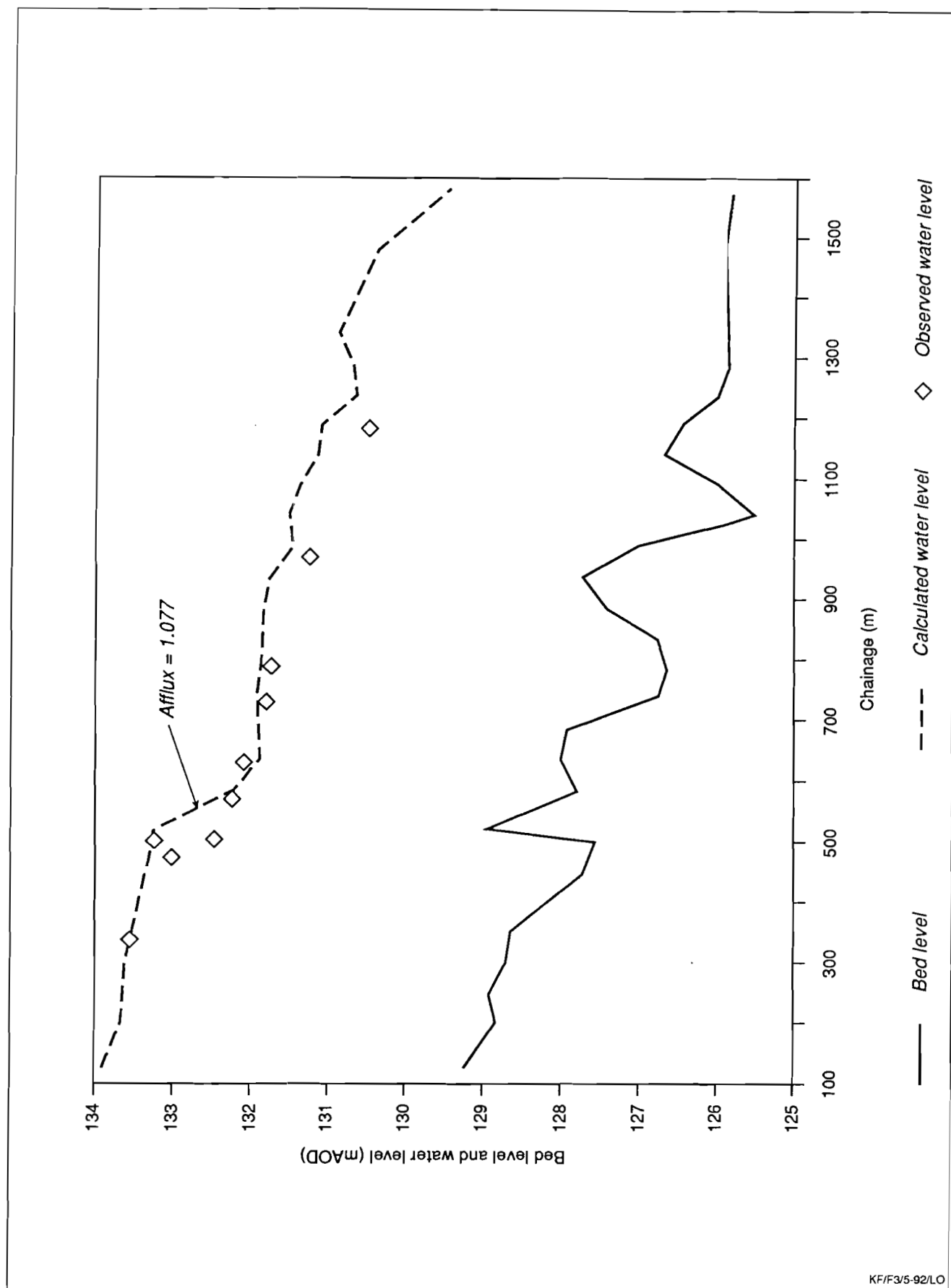


Figure F3 Water levels and bed levels for 1979 flood

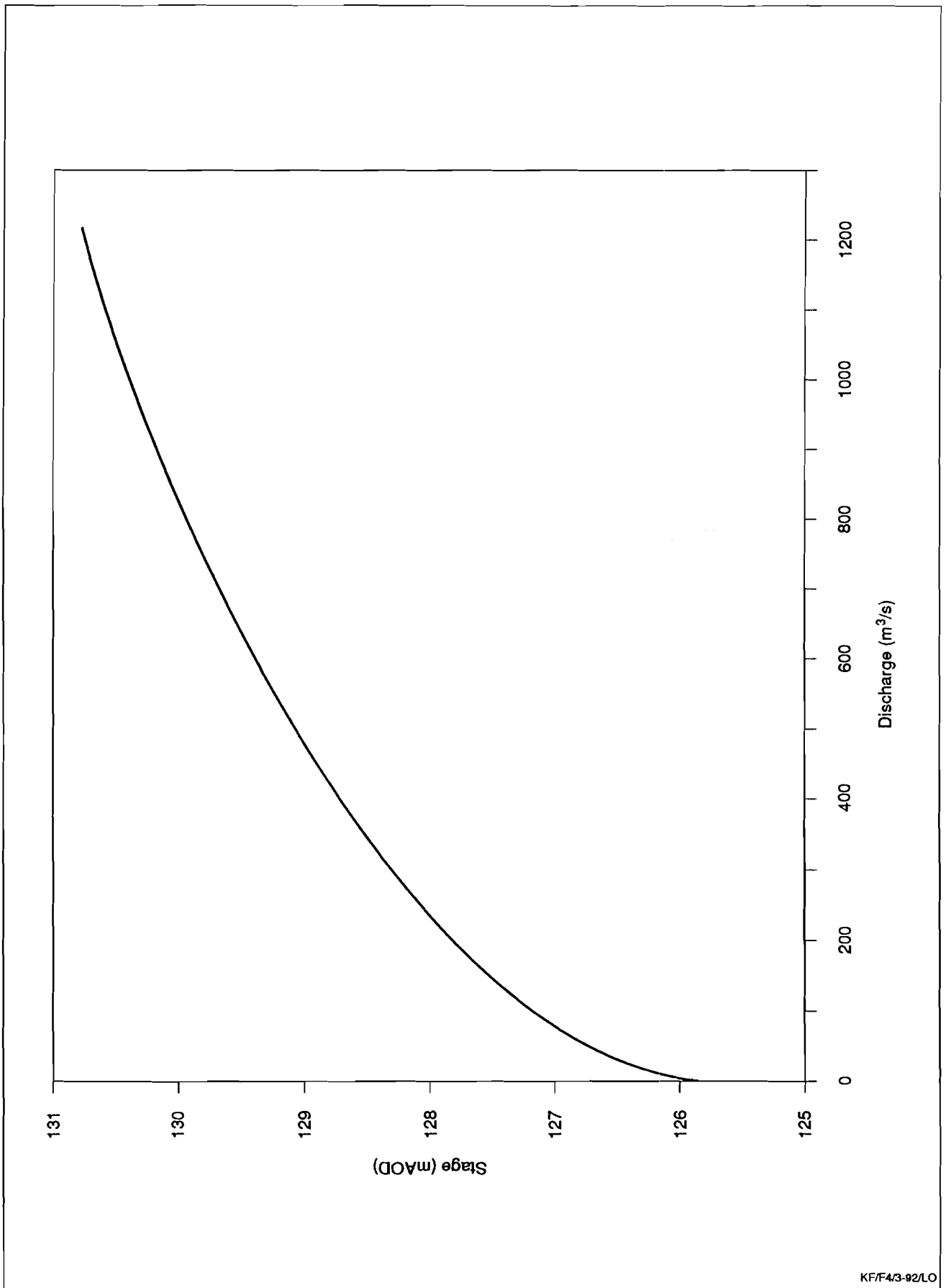


Figure F4 Stage discharge for downstream section

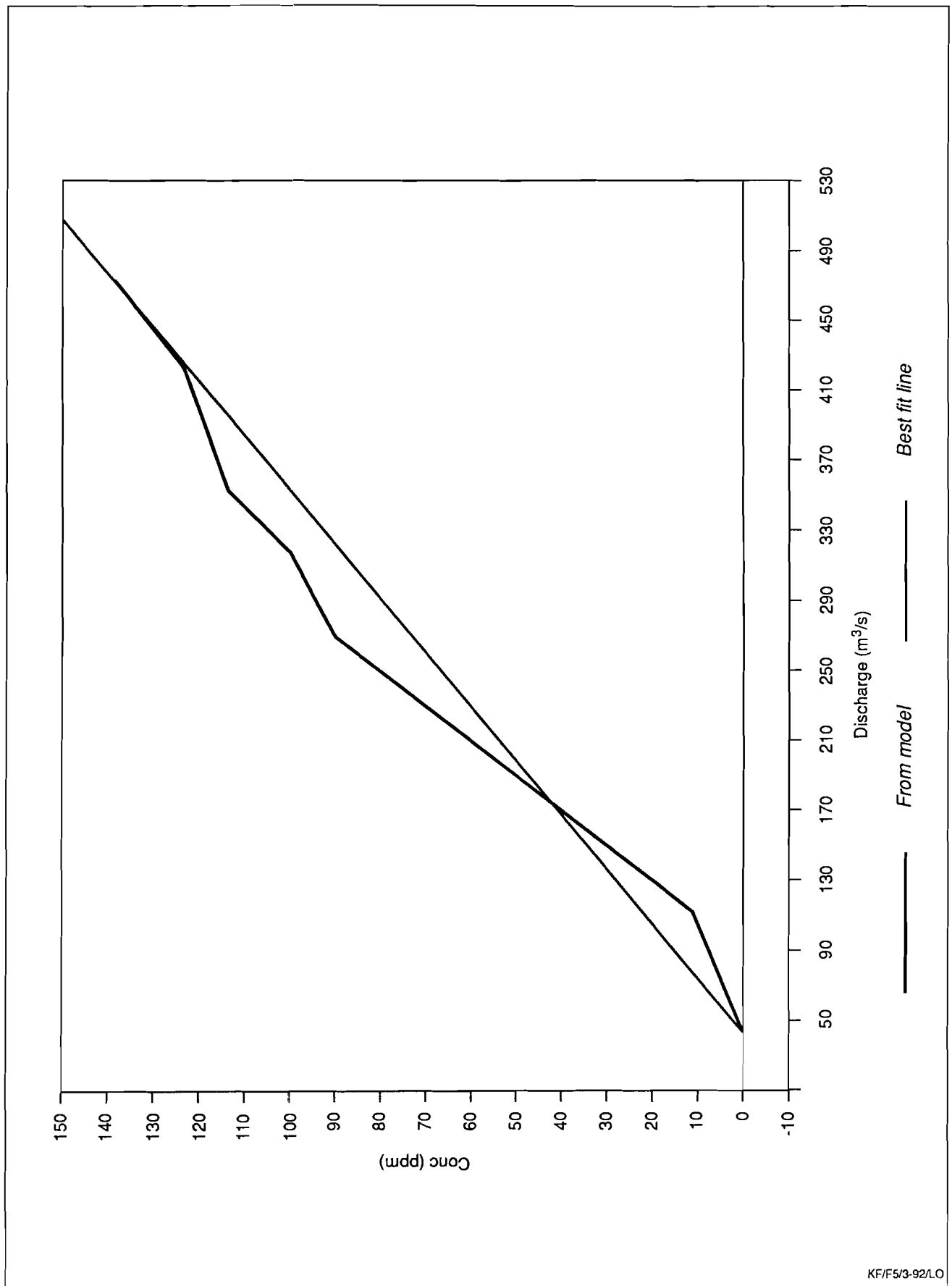
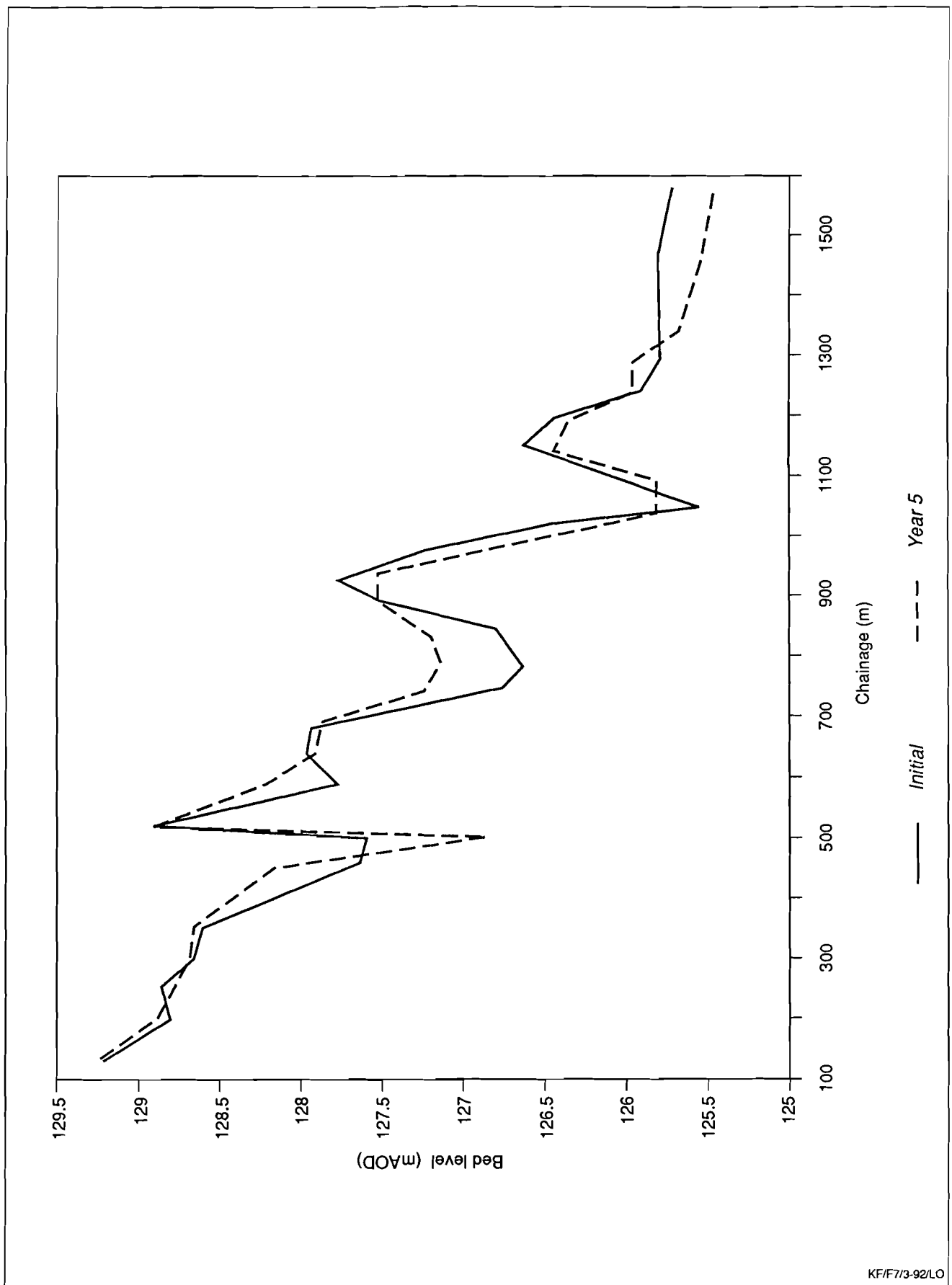


Figure F5 Sediment rating



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Figure F6 Pre-improvement scheme: Bed levels initial and after 5 years

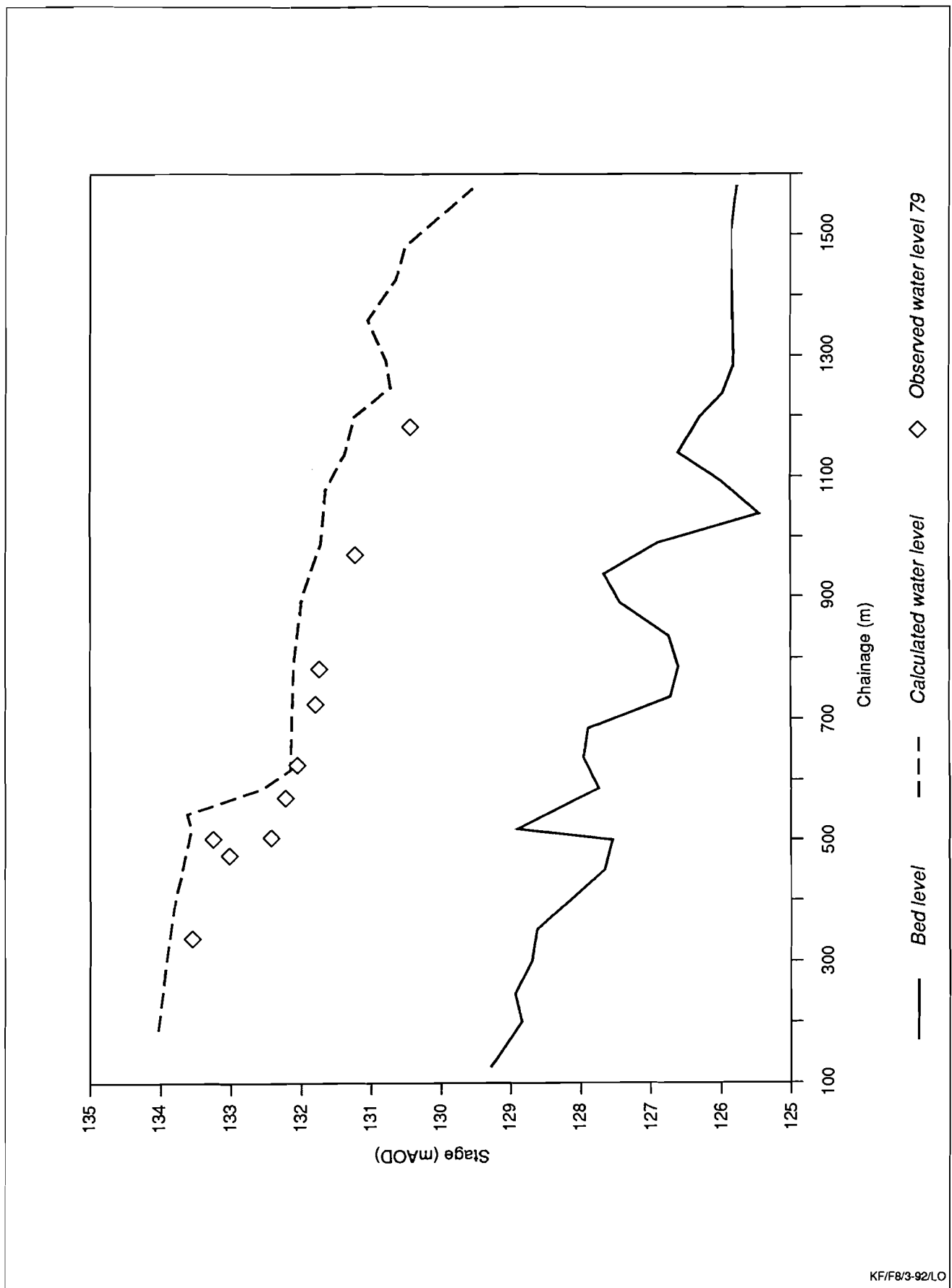
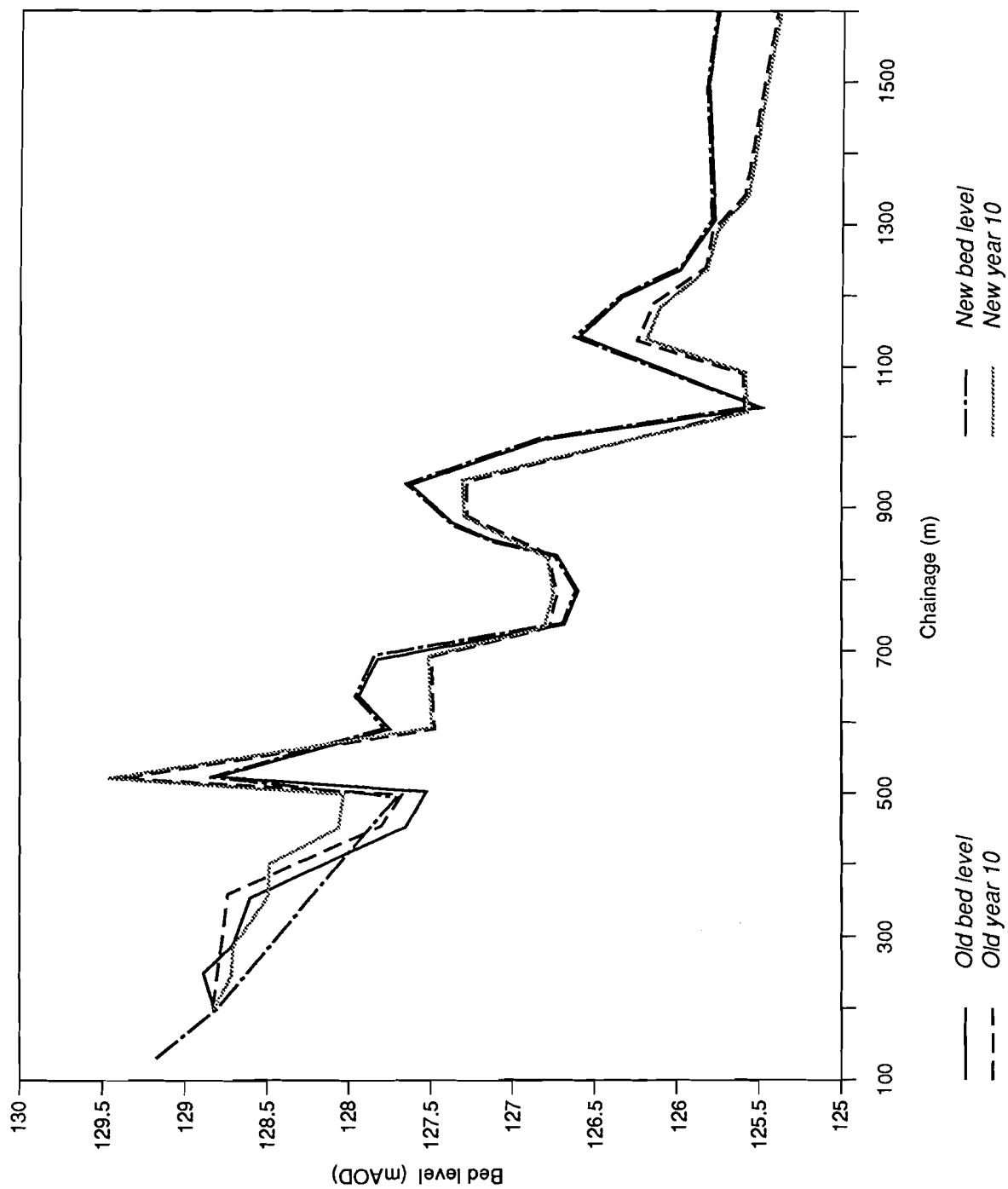
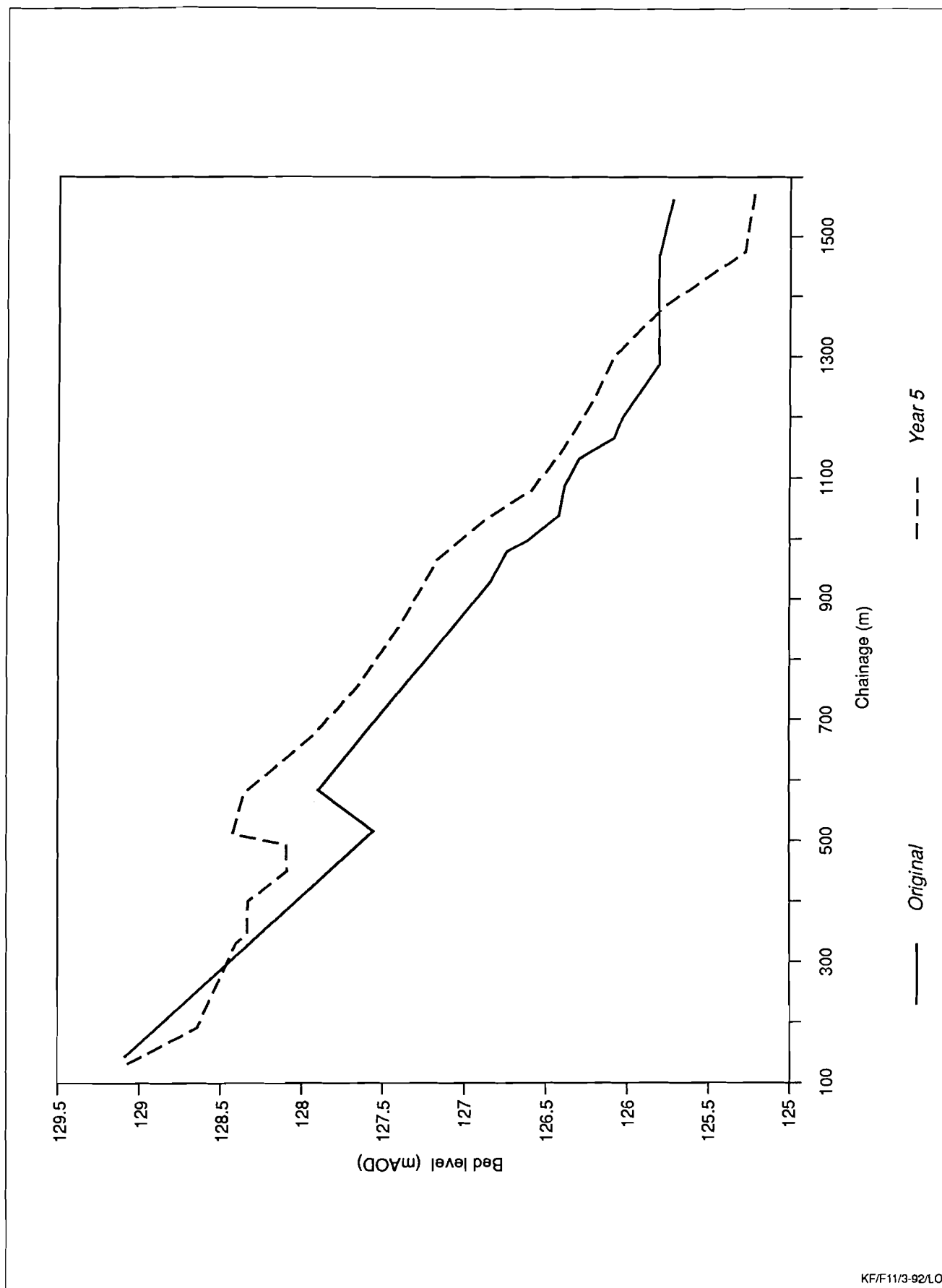


Figure F7 Pre-improvement scheme: Bed levels and water levels (calculated and observed) for a 1 in 100 year flood



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Figure F8 Post-improvement scheme: Bed levels initially and after 10 years



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Figure F9 Modified scheme 1: Bed levels initially and after 5 years

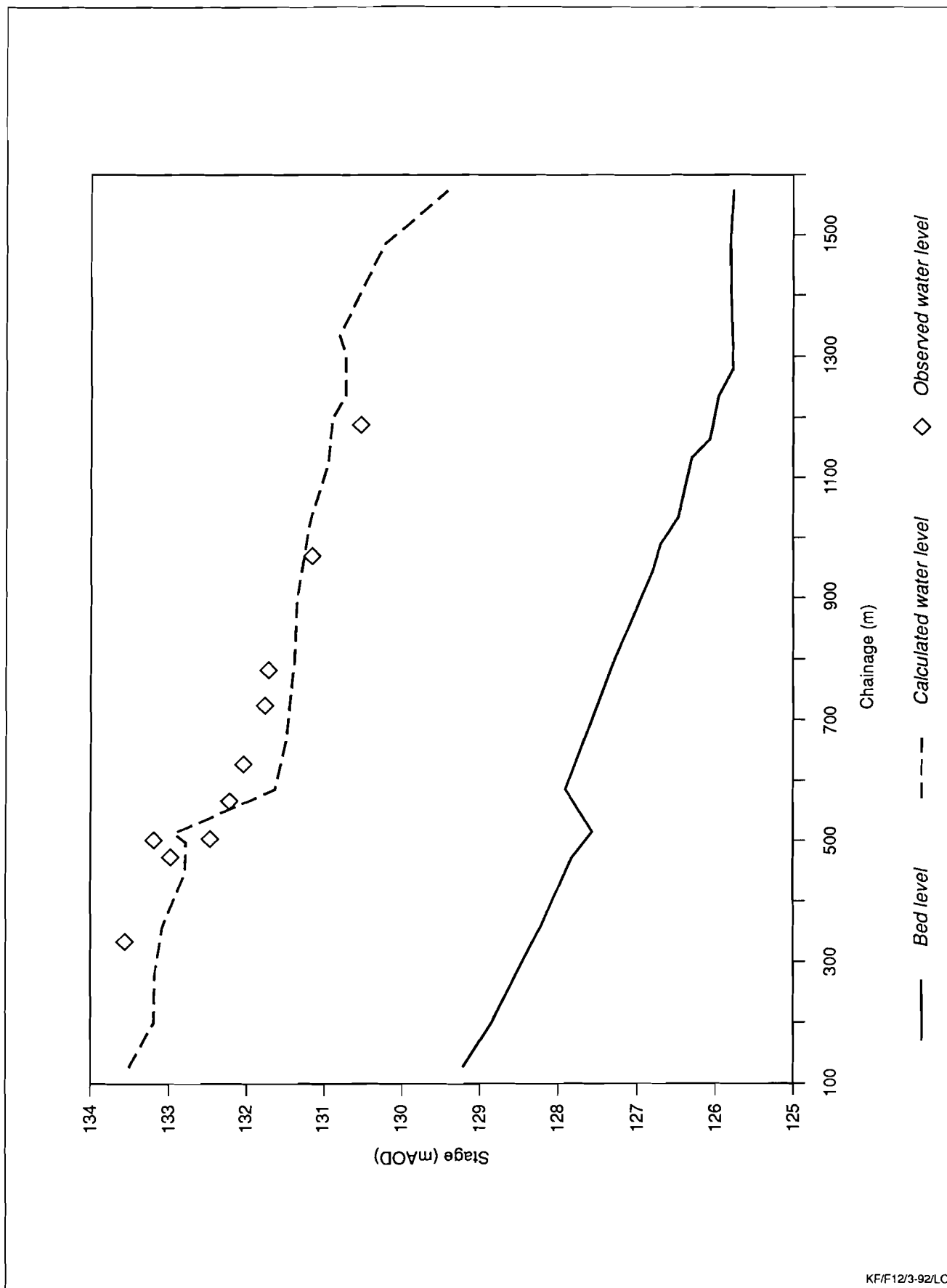


Figure F10 Modified scheme 1: Bed levels and water levels (calculated and observed) for a flood similar to 1979 flood

Appendix G

Case Study : Aylesford Stream Improvement Scheme

G.1 Introduction

The Aylesford Stream in Kent drains a catchment of approximately 20km² which lies to the East of Ashford. The River joins with the East Stour at Ashford. The catchment is mainly agricultural land and woodland, although development of Ashford since the mid-70's has increased the urban area. The river flooded in 1972, and this prompted the design of a flood prevention scheme. The scheme, consisting mainly of channel widening and removal or modification of bridges, was constructed in 1973/4. Catchment development continued in the late 70's and 80's, including construction of housing estates and roads. The river flooded again in November 1986, and after this event, remedial works were carried out to remove significant quantities of sediment from the channel. It is thought that increased bed levels resulting from deposition of sediment in the post scheme period (1974-1986) may have been partly responsible for the flooding in 1986.

For this study, a one-dimensional morphological model has been used to investigate the morphological changes which have taken place in the river both before and after the 1973/4 flood prevention scheme. In particular, it has been possible to draw conclusions about the likely reasons for the deposition which occurred between 1974 and 1986.

G.2 Pre-improvement scheme

Flooding took place in 1972, and the Kent River Authority designed a channel improvement scheme to alleviate flooding and protect nearby land and buildings. A short time before work on the scheme was due to commence, in late September 1973, another major flood occurred, of similar magnitude. Records show that several dozen properties were affected. The most serious flooding was just downstream of the Crowbridge Road Bridge. Figure G1 shows the pre-improvement scheme conditions.

G.3 Post-improvement scheme

The flood prevention scheme was designed to increase channel capacity in the downstream reaches of the river. The upstream limit of the improvements was some 0.8km downstream of Swatfield Bridge on the A20 road, and the downstream limit was the confluence with the East Stour. The length of channel which was improved was 3km. The main works which were carried out are summarised below

- a) Channel cross section was enlarged throughout the improved reach, by deepening and widening.
- b) Several bridges were removed or reconstructed to reduce flow resistance due to bridges.
- c) Two short sections of channel were re-aligned to reduce channel curvature.
- d) A 30m long concrete flume section was constructed immediately downstream of the Crowbridge Road bridge.

- e) The left bank was raised by an 80m long concrete wall immediately downstream of the concrete flume section.

The flood prevention scheme was established in later 1973 to 1974 and the post-improvement scheme can be seen in Figure G2.

The improved channel was designed to offer protection against a flood with a return period of 100 years.

A number of developments took place in the catchment during the period 1984-1986. New housing estates were built adjacent to the river in 1974/5 and from 1985 onwards. The housing development which took place from 1974 to 1985 covered an area of approximately 0.4km², the majority of which was on green field sites. Drainage from the estates is attenuated by the use of storage tanks. Major road construction has also taken place. A section of the M20 motorway was built within the catchment, north of the river, in 1978/9. Part of a new Ashford bypass, called the Southern Orbital Route (SOR) was built on a sandy embankment crossing the Aylesford Stream, in 1983. Both of these road schemes involved major earthworks. All of these developments could have caused changes in the flows and sediment loads in the Aylesford Stream.

G.4 Maintenance

In the period from 1974 to 1986, the only regular maintenance carried out on the river channel was a yearly program of aquatic weed and bank vegetation clearance. No major dredging took place during this period.

The Water Authority carried out dredging to the river in the summer of 1987, in the reach between the Crowbridge Road Bridge and the confluence of the river with the East Stour. Maintenance staff estimate that the quantity of sediment removed was between 1m³ and 2m³ (bulk) per metre length of river, and the dredged material was silt and sand. During the dredging operation, overnight deposition of sand was observed in the reach which was being dredged.

After the dredging had been completed, the channel was re-surveyed, in November 1987. This November 1987 stream bed profile can be compared with the bed profile of the 1972 improved channel design. The 1987 bed levels are similar to or slightly above the 1972 designed bed levels. This comparison, together with the estimate of dredged quantities, enables us to estimate the quantity of net deposition which took place between 1973 and 1987. It is thought that this deposition, and consequent reduced channel capacity, may have been at least partly responsible for the flooding in 1986.

G.5 Representation of bridges

The modelled reach has up to 10 arch bridges. These are expected to increase water levels upstream due to an effect called afflux. Afflux was calculated using the method described in Hydraulics Research Report SR 182. The method was derived from physical model tests on arch bridges, and is designed to be especially suitable for many older British bridges which have arches rather than having vertical piers.

If the downstream depth was below a critical threshold level, it was assumed that the bridge ceased to cause afflux, but instead the invert acted as a control. In this case, the upstream water level relative to the invert level was assumed to be proportional to $q^{3/2}$, where q is the unit discharge at the bridge. The

constant of proportionality was calculated by assuming that, in the critical state, the upstream water level given by the power law relationship equalled the upstream water level given by the afflux calculations.

G.6 Model calibration

The model was calibrated by assuming that the river was in regime before the flood alleviation scheme was constructed. The model was run for a ten year period, and the main criterion for calibration was that there should be no predicted change in bed level during this simulation period.

Flow data was obtained by two methods. The river itself is not gauged, but there is a gauging station on the East Stour river, which drains a neighbouring catchment. Flows measured at the gauging station were scaled to account for the smaller catchment of the Aylesford Stream, and a flow exceedence curve for the Aylesford Stream was constructed (Fig G3). A flow exceedence curve extending to more rare events was also constructed using mean daily flows with return periods of up to 100 years which were calculated using methods contained in the Flood Studies Report. The flow exceedence curves obtained by the two methods were in close agreement.

The downstream flow boundary condition was defined as the normal flow depth in the channel at any particular discharge. In reality the tailwater level is affected by the water level in the East Stour river, but this effect was ignored in the model.

Sediment input to the reach was calculated from the bed slope and flow conditions at the upstream end of the reach. Bed material grain size in the reach was based on grab samples taken in 1989 (Fig G4), but these may not be representative of the bed material before 1973, and no information was available concerning bed material properties before the scheme was constructed.

Channel geometry was based on Kent River Authority drawings of 1972. These show both existing (1972) cross-sections, and design scheme cross-sections.

The morphology model was used to carry out a ten year simulation based on pre-scheme conditions. The result of this simulation is shown in Figure G5. It can be seen that the reach is approaching equilibrium conditions after ten years - the change in bed levels during the final years is considerably less than the change during the first five years. Overall, the model predicts some erosion from the bed, compared with the original bed levels. This predicted tendency to erode may in fact be prevented by a tendency of the stream to armour with coarser grains as finer grains are removed.

G.7 Model results

The two major elements of the scheme which are most likely to have effected channel morphology are channel enlargement and removal/modification of bridges. This study has examined the effects of each of these elements, both separately and together. The effect of increasing the inflow of sediment into the modelled reach has also been assessed. The effect of the alternative schemes can be seen from Table G1. For each scheme, this gives the quantity of material deposited in 10 years downstream of Crowbridge Road Bridge, relative to the predicted deposition resulting from the existing, pre scheme channel.

G.7.1 Effect of channel widening and deepening

The long term effect on bed and water levels of the channel enlargement works is shown in Figure G6. It was assumed for this test that the new channel cross-sections were constructed, but the original bridges remained in place. Similar erosion to that from the calibration run is predicted upstream of Crowbridge Road Bridge, but downstream of this bridge, more deposition is predicted.

G.7.2 Effect of bridge removals

The effect of the changes to bridges carried out as part of the scheme can be seen in Figure G7.

There is no deposition downstream of Crowbridge Road bridge due to the effect of the bridge removals and there is similar erosion as in the pre-scheme conditions. We conclude therefore that just the effect of removing bridges has had little effect on the morphology of the river.

G.7.3 Effect of overall scheme

The combined results of channel widening and deepening and the bridge modifications are shown in Figure G8. The major contribution to the deposition of sediment (1180m^3) comes from channel widening and deepening and a smaller proportion from bridge modifications. There is some erosion upstream of Crowbridge Road bridge.

G.8 Modified schemes

G.8.1 Post-improvement scheme sections and modified bridges

For this alternative scheme the model was run with three of the bridges downstream of Crowbridge Road Bridge removed. The results of this modified scheme are shown in Figure G9. The deposition downstream of Crowbridge Road Bridge has been reduced from the post-improvement scheme condition and although there is some deposition this appears to be stable after a simulation time of 10 years.

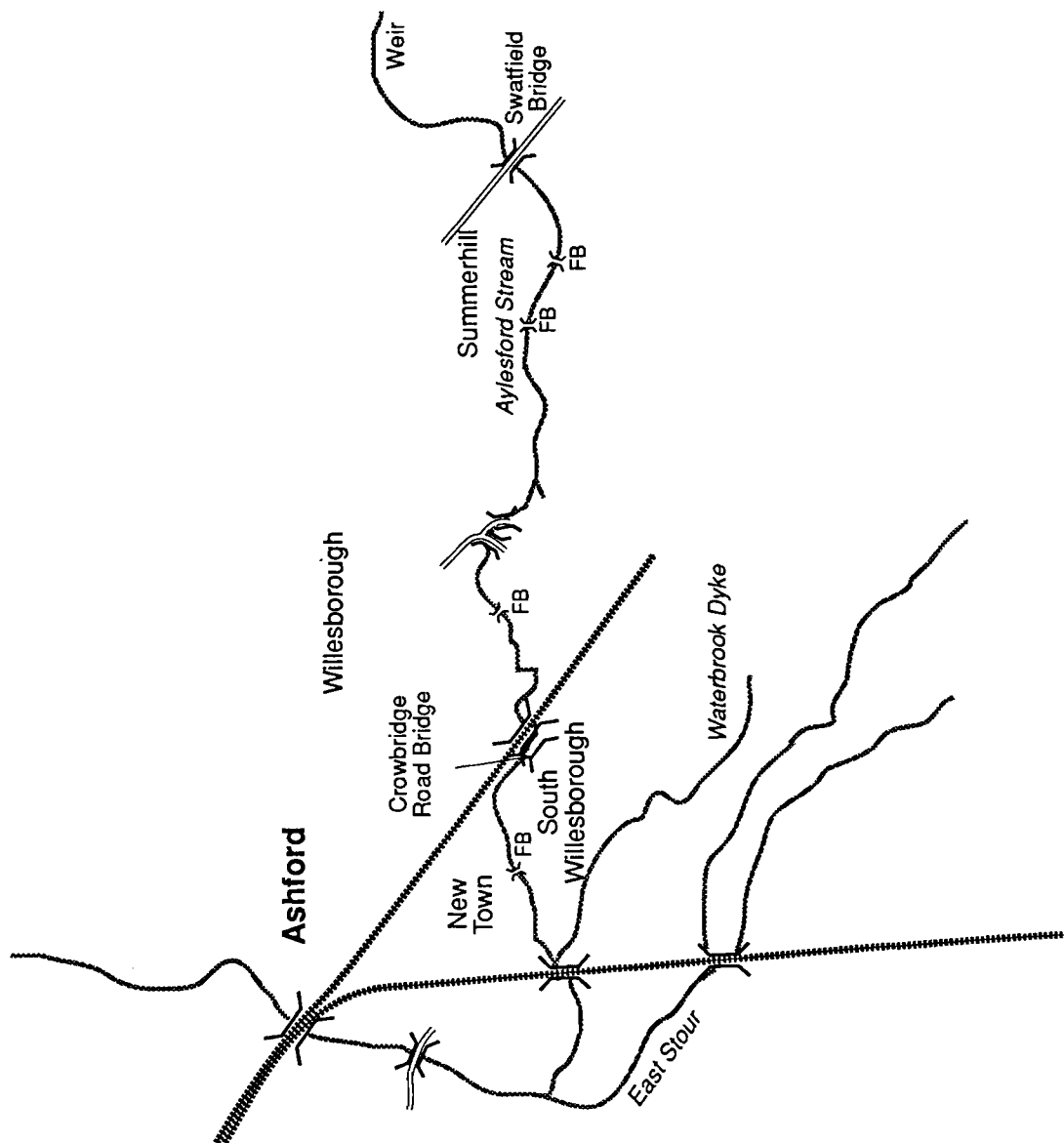
G.9 Conclusions and recommendations

The improvement scheme of Aylesford stream involved widening and deepening of the channel and removing or making modifications to some of the bridges. The combination of these two effects caused deposition in the reaches downstream of the Crowbridge Road Bridge. The major part of this deposition was, it appears, due to widening and deepening the channel.

By removing the effects of three of the bridges downstream of Crowbridge Road Bridge and retaining the new sections from the improvement scheme a modified scheme shows less sediment deposition downstream of Crowbridge Road Bridge.

Table G1 Effect of different schemes on sediment deposition

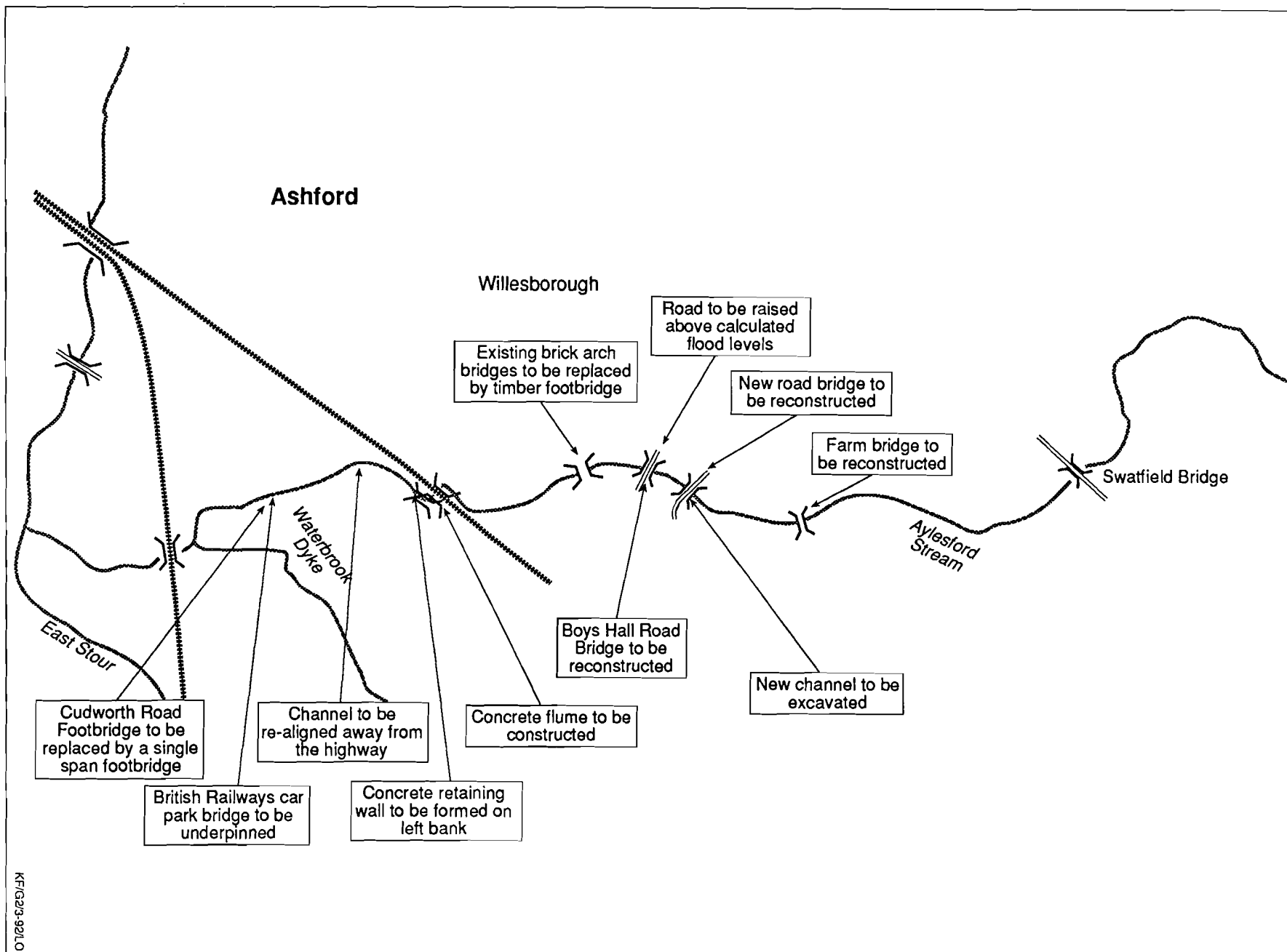
Scheme	Sediment deposition downstream of Crowbridge Road Bridge, relative to pre-scheme
	(m ³)
pre-scheme cross sections and pre-scheme bridges	0
pre-scheme cross sections and post-scheme bridges	260
post-scheme cross sections and pre-scheme bridges	900
post-scheme cross sections and post-scheme bridges	1180



KF/G1/3-92/LO

Figure G1 Pre-improvement scheme plan

Figure G2 Post-improvement scheme plan



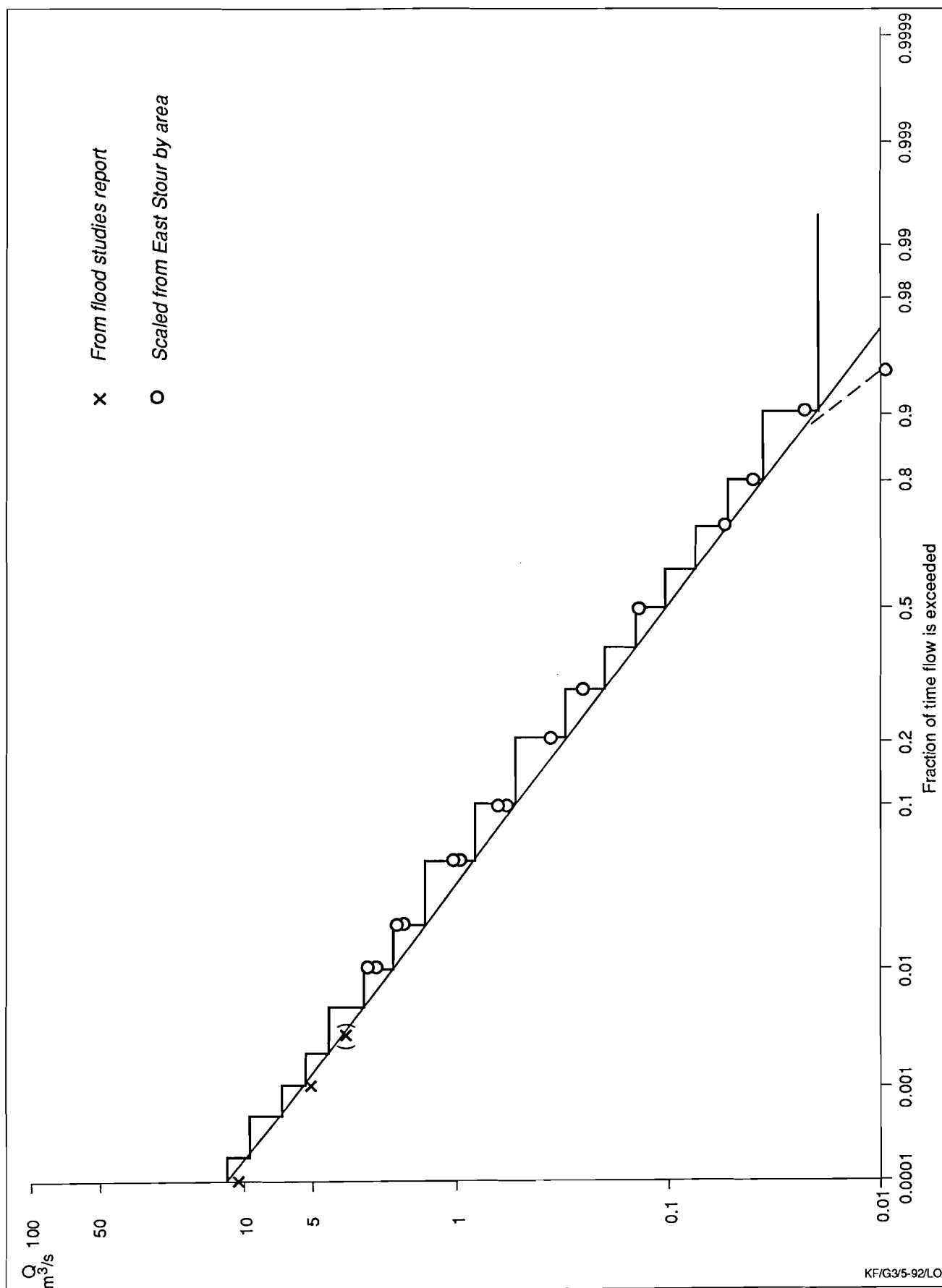


Figure G3 Flow exceedence curve

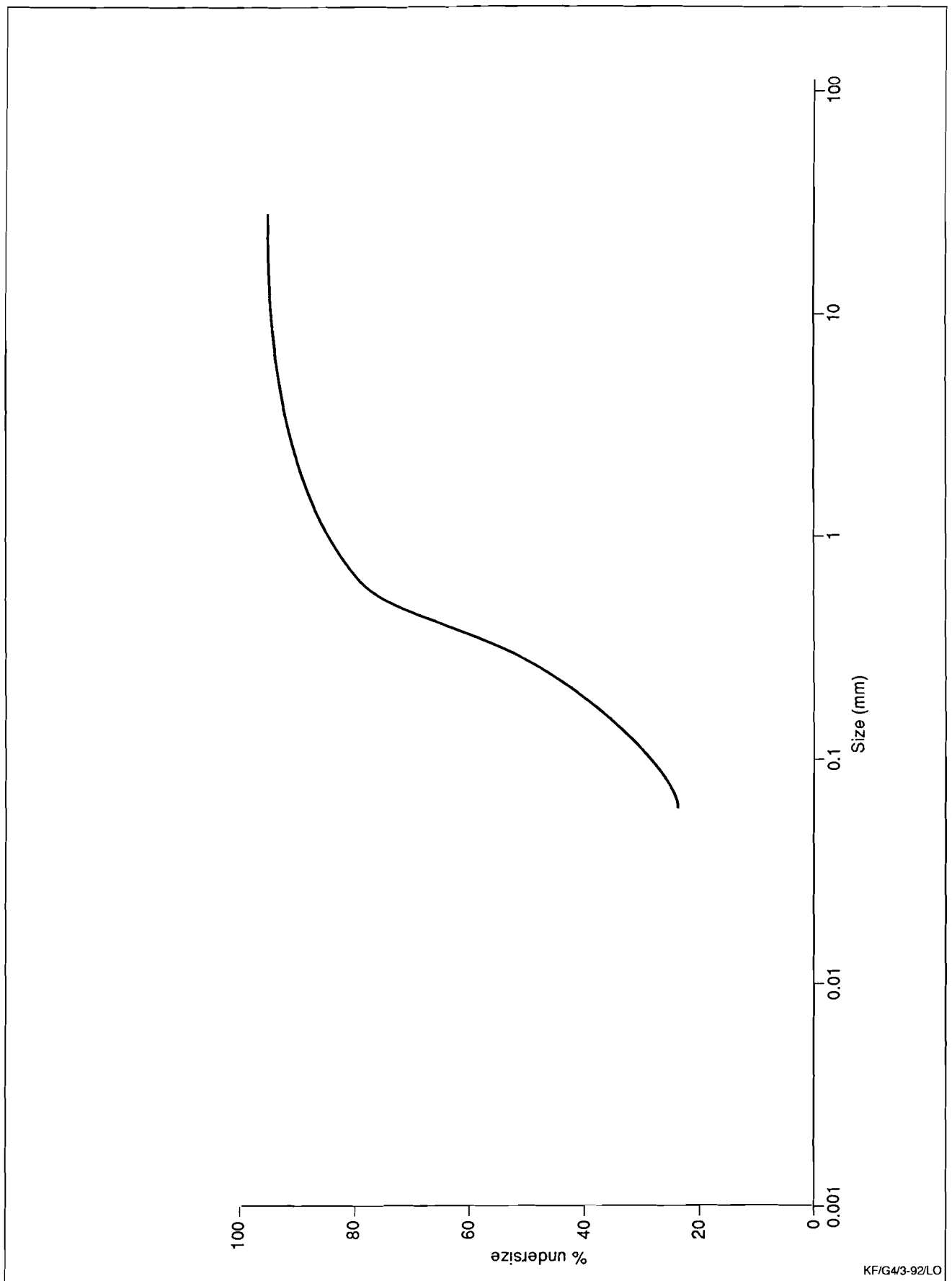
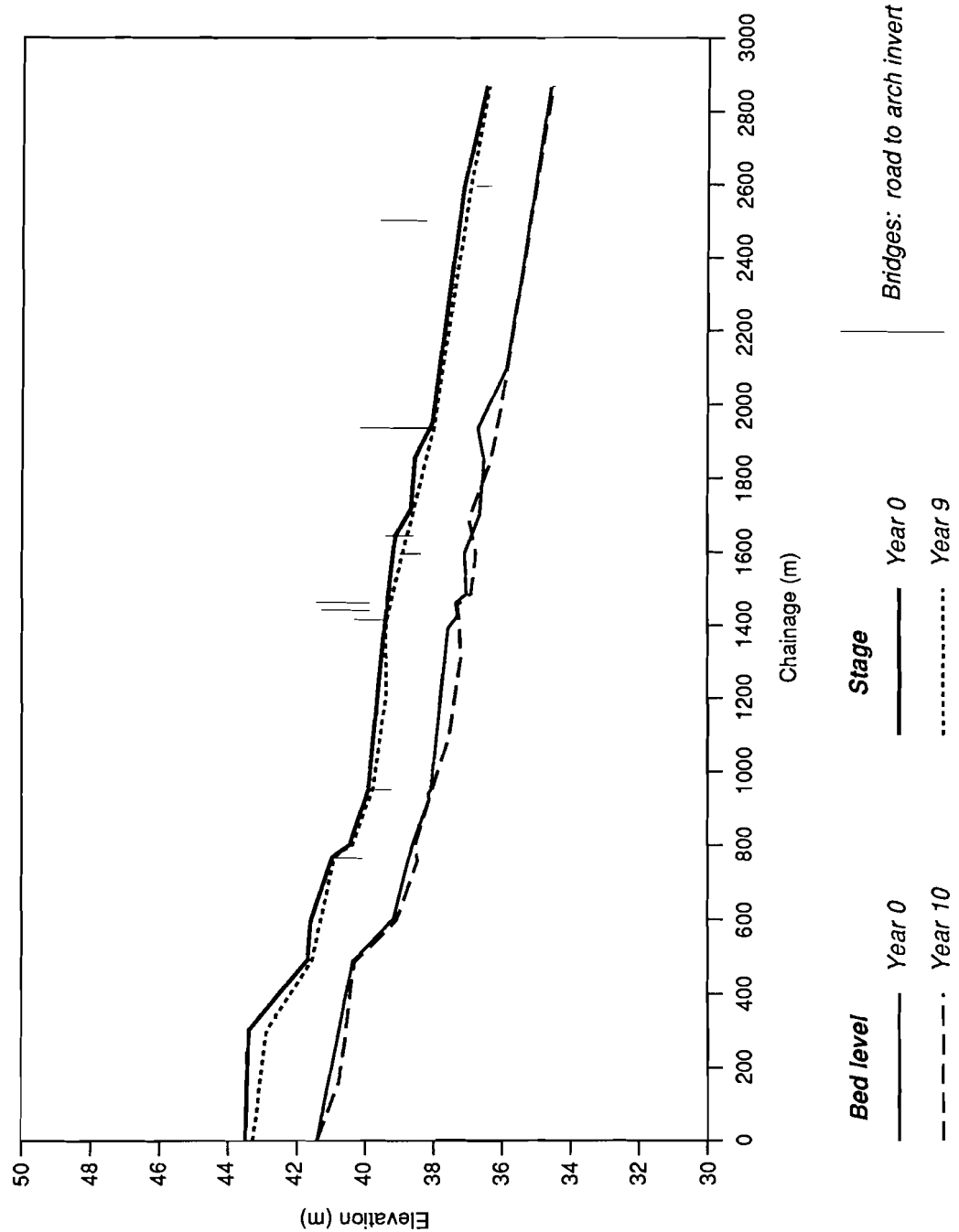
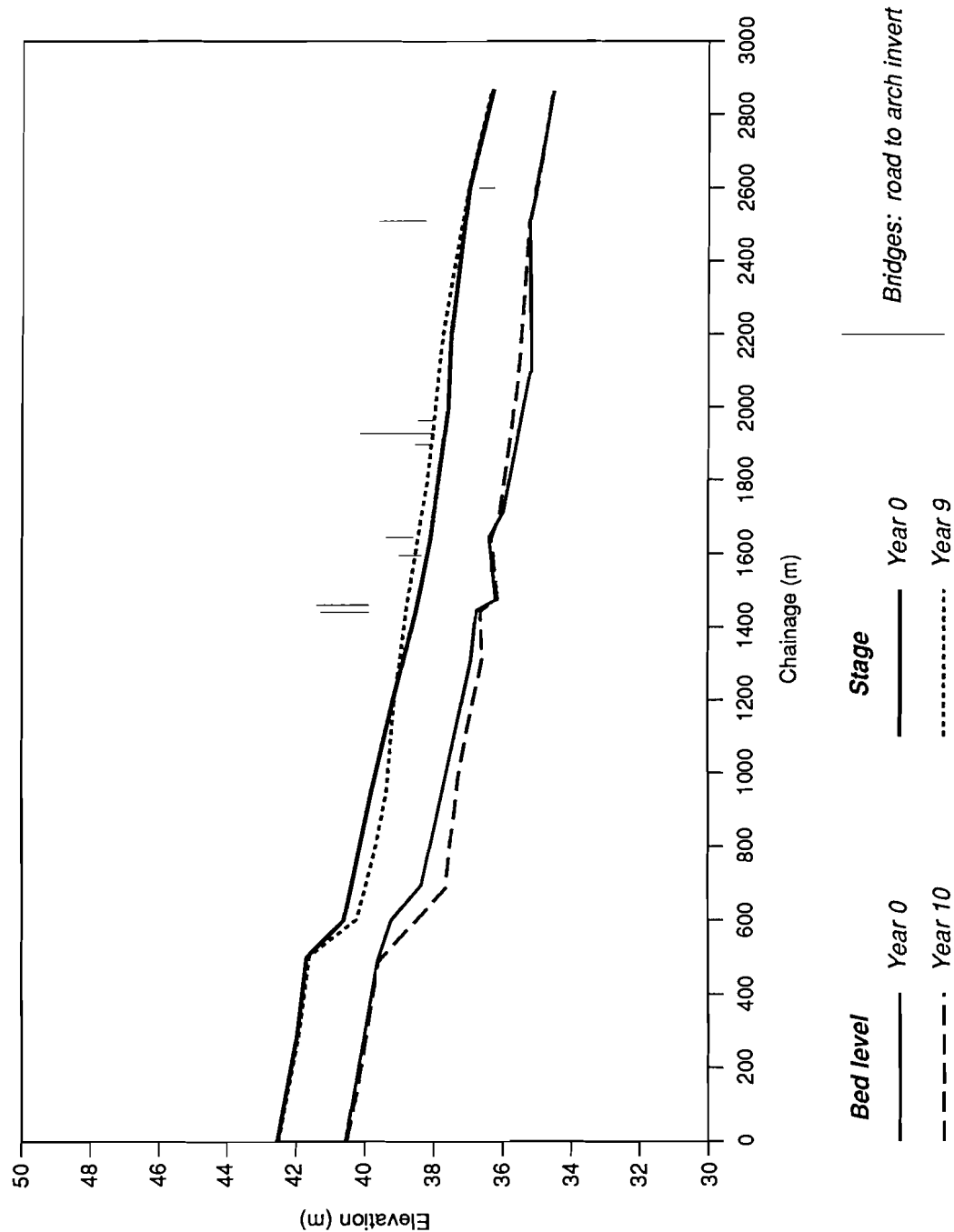


Figure G4 Sediment rating



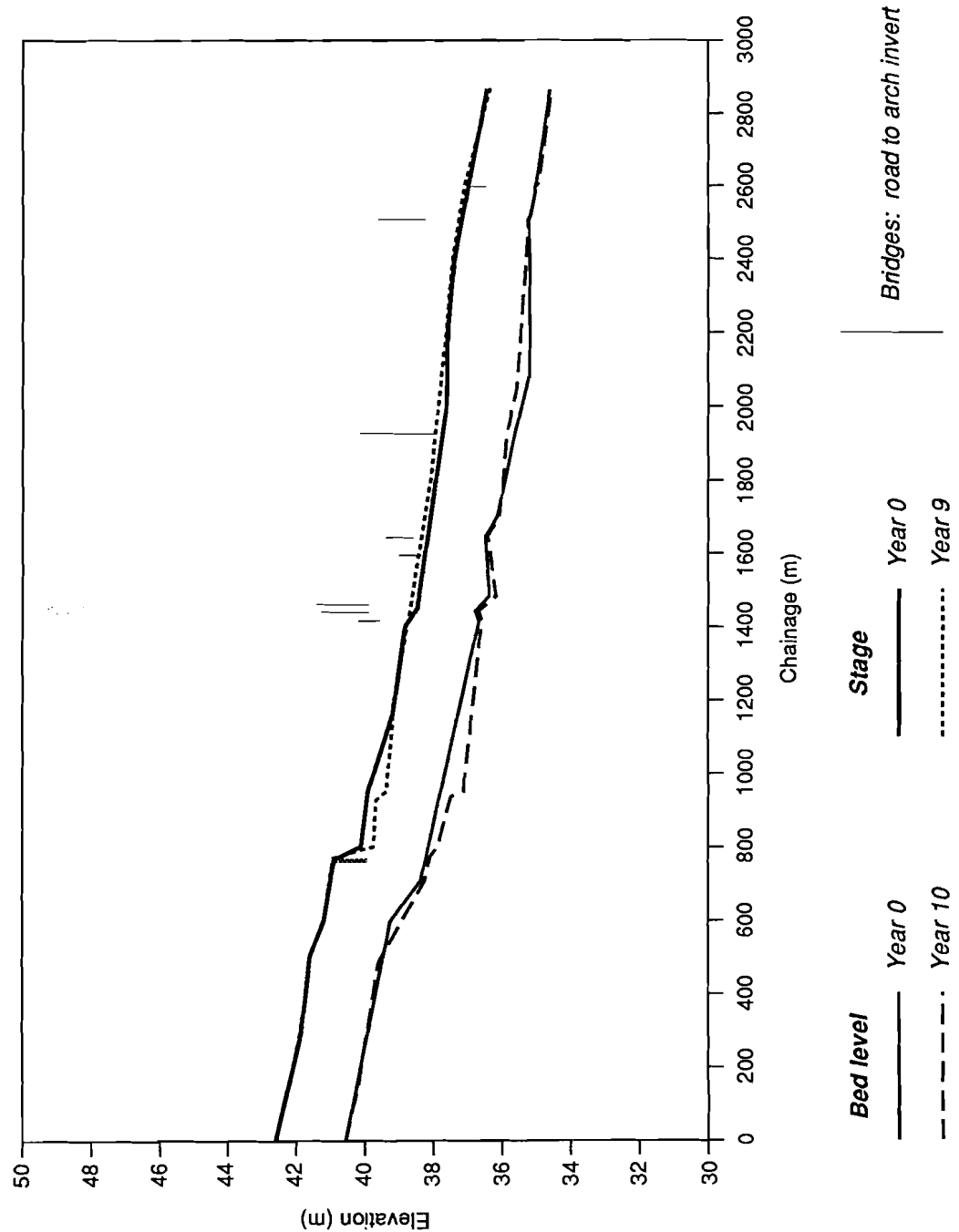
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Figure G5 Pre-improvement scheme: Bed levels and water levels initially and after 10 years



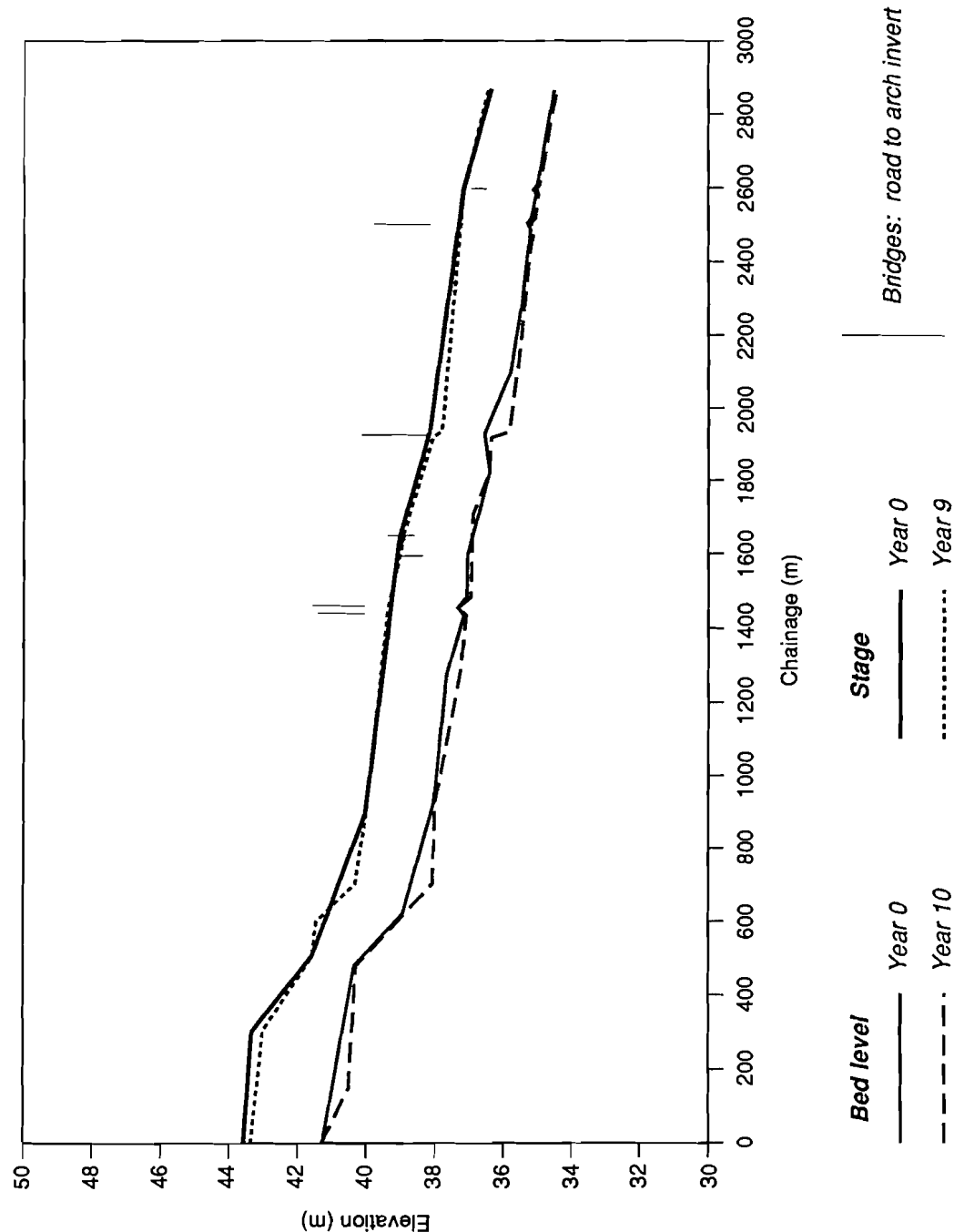
KF/G6/3-92/LO

Figure G6 Post improvement scheme: Effect of channel widening and deepening: Bed levels and water levels initially and after 10 years



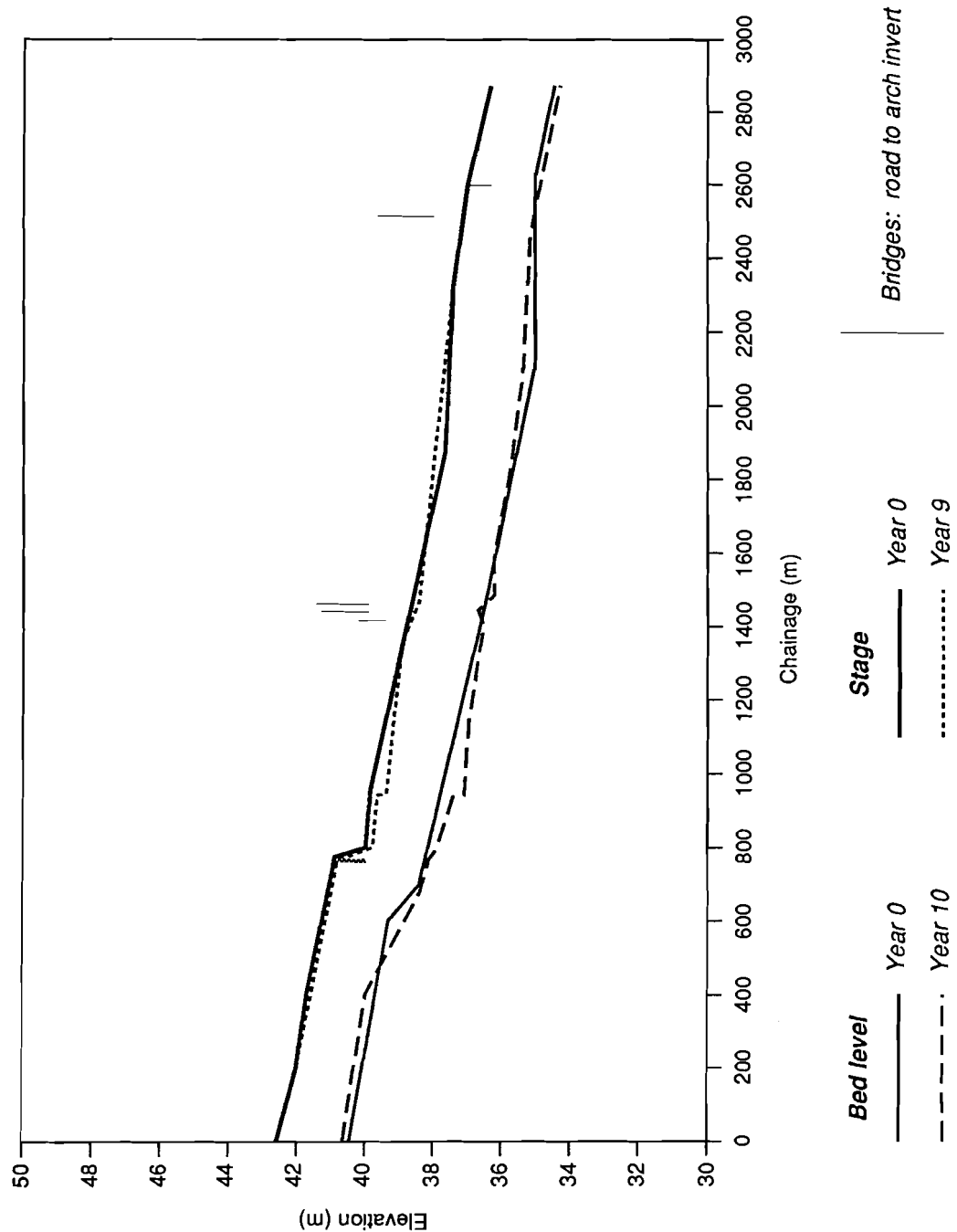
KF/G7/3-92/LO

Figure G7 Post improvement scheme: Effect of bridge removals: Bed levels and water levels initially and after 10 years



KF/G8/3-92/LO

Figure G8 Post improvement scheme: Effect of overall scheme: Bed levels and water levels initially and after 10 years



KF/G9/3-92/LO

**Figure G9 Post improvement scheme sections and modified bridges:
Bed levels and water levels initially and after 10 years**

Appendix H

River Sence Improvement Scheme

H.1 Introduction

The River Sence flows through the county of Leicestershire. The source of the river is to the south-east of Leicester and it flows in a westerly direction until it joins with the River Soar to the west of the town of Wigston.

The reach of river being studied in this project is between Kilby Bridge and the bridge adjacent to the mill downstream. This reach is approximately 3km long and here the river flows through mainly agricultural land to the south of Wigston. There are numerous ditches draining into the river south of Wigston. There is one significant tributary 500m upstream of the mill. This reach of river follows a very sinuous path with tight bends and the cross-sections show that the river is generally deep relative to the width in this area.

The general layout of the river from Kilby bridge to the confluence with the River Soar is shown in Figure H1.

H.2 Background

There have been some flooding problems on the agricultural land adjacent to the River Sence in this area and there is an extensive drainage network draining water from the land into the river. In 1973 a river improvement programme was undertaken for the reach of river between Kilby Bridge and the bridge adjacent to the mill downstream. This improvement programme involved widening, deepening and straightening of the channel.

In the period up to 1973 there had been few problems with deposition of sediment although cross-sections from that reach between 1965 and 1971 show that there was a small amount of deposition which was cleared or controlled by routine maintenance.

After the river improvement works in 1973 more sediment was deposited until 1990 when maintenance work was carried out to remove approximately 0.5m depth of sediment. During the period 1973 to 1990 there was considerable bank instability with bank slumping and cracking. Vegetation has grown on the 'terraces' which formed as a result of the bank collapse indicating that they are well-established. The contractors were instructed to leave the 'terraces' intact during the recent maintenance in 1990.

The sediment which was removed during the de-silting was placed on the banks of the river and is widely graded from gravel to fine sand/silt. The exposed bed can be seen as a gravel layer with a covering of silt which remains from the de-silting programme. Below the gravel layer there is finer sand and silt material. The details of the bed material are given in more detail in Section H3.

Some recent reed and vegetation growth can be seen in the river channel apart from the established vegetation on the river terraces.

The present morphological model study was undertaken to model this reach of river, using existing data to calibrate the model and to give an estimate of the sedimentation problems in the next few years and the effect on water levels.

H.3 Model details

H.3.1 Model data

The data required for the morphological modelling were provided from National Rivers Authority (NRA) and Silsoe College.

The cross-section data provided includes:

- cross-sections pre-improvement scheme: 1965
- cross-sections pre-improvement scheme: 1971
- design cross-sections for improvement scheme: 1973
- cross-sections post-improvement scheme: 1989
- cross-sections post-improvement scheme & post maintenance: 1990.

A flow exceedence curve for the River Sence measured at South Wigston is taken from the period January 1984 to December 1987. Although the flow frequency is only a short record over a three year period the location is within the simulated reach. The flow frequency curve can be seen in Figure H2.

The boundaries were taken at the upstream end at Kilby Bridge and at the downstream end at the overflow weir, upstream of the mill. The layout of the simulated reach can be seen in Figure H1. The position of the downstream boundary was selected to be upstream of the tributary which joins the River Sence, 500m upstream of the mill. The only data known for the overflow weir are the crest level and details of the cross-sections upstream and downstream of the weir. Due to this shortage of data, the downstream boundary condition is based on a normal depth calculation for the channel downstream of the overflow weir. The boundary condition therefore is a stage-discharge relationship at the downstream end of the simulated reach.

Some general background to the deposition problems were given in Section H2. Observations made on a site visit indicate that the exposed bed is mainly gravel, widely graded up to a size of 20 or 30mm. Below the gravel layer the lower bed material is much finer sand/silt. Two samples of sediment were taken during the site visit which are assumed to be typical of the dredged material removed. These samples were graded and the results seen in Figures H3 and H4. It can be seen that the samples are very widely graded, especially sample 2, with D35 sizes of:

Sample 1: $D_{35} = 0.32\text{mm}$

Sample 2: $D_{35} = 5.5\text{mm}$

From these size gradings it can be seen that there is actually a very small amount of silt (< 5%) in both samples if we consider the sand/silt boundary size to be 0.07mm. As a result of the size gradings, 3 sizes of sand are used in the model (0.7mm, 1mm, 3mm) with no silt.

The total amount of sediment introduced into the model at the upstream boundary can be calculated from the surface water gradient using a sediment transport formula, eg Ackers-White sediment transport equation, or can be a constant input. The amounts of sediment being transported into the model can be checked if the sediment yield for the catchment is estimated and the size of the catchment is known. The size of the River Sence catchment is approximately 203km^2 and a typical sediment yield for this type of catchment is 20

tonnes/km²/yr. For sand the specific gravity is 2.65 so we can calculate the total volume being transported every year:

$$\text{Volume} = (20 \times 203) / 2.65 = 1532 \text{m}^3/\text{yr}$$

H.3.2 Model Calibration 1: pre-improvement scheme

With the relatively large amount of cross-section data available two calibrations were carried out: one pre-improvement scheme; and one post-improvement scheme. The initial cross-section data available are for 1965 and 1971. Over that six year period there is some sediment deposition and the first calibration simulates this deposition. The first calibration tests are performed with a time-step in the model of 12 hours.

The channel is very sinuous and has weed growth in several places. These two factors cause the channel roughness to be higher than would be expected for a channel of this type. The roughness is represented by:

$n = 0.045$ for chainage 1786m to downstream section

$n = 0.04$ for upstream section to chainage 1786m

$n = 0.08$ for floodplains

The total sediment transported past the upstream section over a period of 6 years, as predicted by the model, is 9950m³, which represents a volume of 1658m³ per year. This volume is comparable with the volume 1532m³/yr predicted for a catchment of this size, as described above and gives a catchment yield of 21.65 t/km²/yr. The total net deposition for this calibration is predicted by the model to be 2191m³ over a period of six years. From the cross-section data an estimate of sediment deposition gives a value of 1700m³ over a period of six years.

H.3.3 Model Calibration: post-improvement scheme

The data available for the second calibration are the design cross-section data for 1973 and for 1989. After 1989, contractors removed sediment from the reach and the cross-section were restored to the same shape and bed level as in 1973. The contractors estimated that 0.5m of sediment was removed from along the total length of reach. This would involve the removal of 9300m³ of sediment which is probably the upper limit of actual sediment removed. Comparing the actual minimum bed levels in 1973 and 1989, 3740m³ of deposition is estimated. This value is the lower bound limit of deposition and the likely actual deposition is somewhere between 9300 and 3740m³.

The sediment input is calculated from a rating equation at the upstream end of the reach where the sediment concentration is related to discharge. This approach was used to ensure a sediment yield for the catchment which is as close as possible to the sediment yield for the model calibration: pre-improvement scheme of 21.65 t/km²/yr. The three sand sizes are the same as in the pre-improvement scheme model calibration: 0.7mm; 1mm; 3mm. The roughness remained the same as the pre-improvement scheme model calibration.

H.4 Model results

H.4.1 Calibration 1: pre-improvement scheme

The long profile of the bed and water levels shown in Figure H5 and the actual and model deposition between each cross-section is given in Table H1. It can be seen from Table H1 that the distribution of the sediment measured is different from the deposition predicted in the model. As the model is one-dimensional, two dimensional deposition effects found in a sinuous river are not simulated and the actual and model predicted deposition could be different. Figure H6 shows the different pattern of model predicted and actual deposition, for a sample cross-section, although it can be seen that overall the deposition is approximately correct for this section. The total amount of sediment entering the river at the upstream section is reasonable for this type of catchment ($21.65 \text{ t/km}^2/\text{yr}$).

H.4.2 Calibration 2: post-improvement scheme

The long profile of the post-improvement scheme bed and water levels shown in Figure H7 and the actual and model deposition between each cross-section is given in Table H2. The bed levels in the mid-section of the reach are predicted by the model to have increased by approximately 30cm in the worst case over 15 years with a corresponding rise in water level of approximately 15cm for a high discharge of $9.26 \text{ m}^3/\text{s}$. Figure H8 compares the actual and predicted bed levels after 15 years and demonstrates that the model predicts that the sediment deposits at different sections to the actual case. The total amount of sediment, which the model predicts, being deposited in the reach is 3750 m^3 (Table H2) which is close to the lower bound of the actual sediment deposition mentioned in Section H3.3. From Figure H9 it can be seen that the major amount of sediment is deposited in the first six years 1973 to 1980 suggesting that the river quite quickly reaches an equilibrium state.

In the middle part of the reach there is deposition to a depth of 20 to 25cm. This deposition will cause a rise in water level. Figure H10 shows that the rise in water levels is up to 18cm in the mid-section of the reach where the deposition is greatest which is less than the corresponding rise in bed levels.

At each cross-section a stage/discharge relationship for 1980 and 1989 is useful to assess the effect of the change in water levels on flooding on left and right banks. For a selected cross-section, Figure H11 shows that on the right bank the bank full capacity is reduced from $9.3 \text{ m}^3/\text{s}$ to $7.5 \text{ m}^3/\text{s}$ and the left bank full capacity is reduced from $4.8 \text{ m}^3/\text{s}$ to $3.5 \text{ m}^3/\text{s}$. This is an average reduction in capacity of 23% for this section. For the whole reach the average reduction in capacity between 1980 and 1989 is 12%.

H.4.3 Simulation run 1: 1991-2006

In 1991 maintenance work was carried out to remove sediment, cut weeds and return the reach to the post-improvement condition of 1973. Comparing the initial bed levels of Figure H7 and Figure H12 it is shown that in 1991 the minimum bed levels were returned to 1973 bed levels except in a few isolated places: at the upstream section where the bed in 1991 is approximately 0.5m higher than in 1973; at chainage 2234m where the 1991 bed level is approximately 20cm lower than the corresponding 1973 level; at the downstream end of the reach where the bed level in 1991 is over 0.5m lower than the bed level in 1973. The model simulation extended over a 15 year period and used the same sediment input characteristics as Calibration 2.

A model simulation was run for 15 years and the effect on water and bed levels is shown in Figure H12. The deposition is represented in graphical form in Figure H13 which shows a large amount of deposition at the downstream end of the reach where the bed was excavated below the 1973 bed level. The deposition in the middle sections of the reach is slightly greater than the deposition between 1973 and 1989. The total overall deposition is 3758m^3 for 1973 to 1989 (Table H2) and 3991m^3 for 1991 to 2006 (Table H3).

In between chainage 1207m and 1509m, the model predicts significant erosion, Figure H13. This erosion is occurring during the highest flow condition of $9.26\text{m}^3/\text{s}$ and the main cause of the erosion is bank and floodplain erosion at the high discharge when the water level is high. There is deposition in the channel at lower flows and there is a rise in bed level after 15 years. However due to the bank erosion at these sections there is net erosion.

The average rise in bed levels in the middle section of the reach is predicted by the model to be approximately 30cm with a corresponding water level rise of 20 to 25cm, a little higher than the rise in water levels from 1973 to 1989. The rise in water levels for selected cross-sections is shown in Figure H15 for the 15 year period 1991 to 2006. A comparison of Figure H10 and Figure H15 shows a greater rise in water levels for the period 1991 to 2006 in the mid-sections of the reach.

Figure H14 shows a stage discharge relationship for a sample cross-section Ch 2234m. The right bank full capacity would be reduced from $9.5\text{m}^3/\text{s}$ in 1989 to $7\text{m}^3/\text{s}$ in 2006 and the left bank full capacity would be reduced from $4.63\text{m}^3/\text{s}$ in 1989 to $3.4\text{m}^3/\text{s}$ in 2006. This would be an average reduction in capacity of 26% for this section.

The rise in water level with time for a number of cross-sections is shown in Figure H16. Chainage 2234m shows the greatest rise in water level of 23cm with the greatest part of this rise occurring in the first 6 years. For cross-sections in the upstream part (792m) and downstream part (3005m) of the reach, the rise in water levels is less than in the central part of the reach. Over the whole reach the major rise in water levels occurs in the first 9 years: 1991 to 2000 which is when the major proportion of the deposition occurs.

H.4.4 Simulation run 2: 1991 to 2006

Using the same cross-section and discharge data as for simulation run 1, six more tests were carried out to investigate the sensitivity of the reach to changes in roughness values. Three roughness values were tested:

- Mannings $n = 0.035$;
- Mannings $n = 0.04$;
- Mannings $n = 0.05$

at two conditions:

- no sediment;
- a sediment input rating at the upstream boundary identical to that in calibration run 2 (Section H4.2).

Tests 1 to 3 were performed with no sediment input to the reach to assess the possible rise in water levels caused by a change of roughness caused by weed growth or deposition of sediment. For an increase in roughness from $n = 0.035$

to $n = 0.05$, the maximum rise in water level in the reach is 30cm as shown in Figure H17. Figure H17a shows the fixed right and left bank levels and the initial bed level. When Figure H17 and H17a are compared it is noticeable that at a high discharge of $9.26\text{m}^3/\text{s}$ the bank level is exceeded on both left and right bank for the major part of the reach.

Tests 4 to 6 were performed with a sediment input rating at the upstream boundary and varying roughness values as given above. Figure H18 shows the impact on bed level of various roughness values over a 15 year period. For roughness values of $n = 0.045$ and $n = 0.05$ there is deposition over most of the reach and increasing the roughness from $n = 0.045$ to $n = 0.05$ increases the deposition considerably at the upstream end of the reach. The volumes of deposition are compared for different roughness values in Figure H19 for each cross-section along the reach.

For a roughness value of $n = 0.035$ Figure H18 indicates that there would be erosion at the upstream end of the reach. The bed level at the downstream end of the reach are shown as remaining stable for this roughness value.

H.5 Conclusions and recommendations

From the model study carried out a number of conclusions can be drawn.

1. Over the 6 year period between 1965 and 1971, the volume of deposition over the reach was estimated by the model to be 2191m^3 . After the river improvement works in 1973, the model estimated a deposition of sediment of 1956m^3 between 1973 and 1980. The overall total volume of deposition over the reach was apparently not affected by the improvement works. The major deposition in the pre-improvement scheme condition from 1965 to 1971 is where the low points in the bed are being filled in (Fig H5). However, Figure H7 shows that there was consistent deposition of sediment in the central part of the reach, where the surface gradient is constant, from 1973 to 1980. Some of the deposition from 1973 to 1980 and then to 1989 was caused by the deepening and widening of the cross-sections in the river improvement works carried out in 1973.
2. The major part of the deposition between 1973 and 1989 occurred in the first six years to 1980 (Fig H9). The change in bed level over this period of up to 30cm caused a corresponding rise in water level of 17cm (Fig H10) and reduced the flow capacity of a section by up to 25% (Fig H11). Although the bed level increase from 1980 to 1989 was smaller than the increase from 1973 to 1980, the water level increase was similar (Fig H11). Even the small rise in bed level from 1980 to 1989 causes the capacity of the channel to be reduced. Therefore the discharge at which the water would overtop left and right banks decreases and the flooding frequency increases.
3. In 1989 maintenance work was carried out to remove sediment accumulated since 1973. Most of the cross-sections were returned to the 1973 profiles but a number of sections were made deeper than the 1973 level. Over a 6 year period 1991 to 1997 predicted deposition in the reach was 2521m^3 . The sediment deposition is greater at the sections which were deepened (Fig H12) and the predicted rate of deposition at some sections shows only a slow reduction after 9 or 15 years post 1991 (Fig H13). This indicates that in 2006, deposition will not have reached an equilibrium and will continue causing further rise in bed and water levels.

4. The deposition over the period 1991 to 2006 will cause a greater rise in water level, up to 23cm (Fig H14) at high discharge values ($9.26\text{m}^3/\text{s}$). This increased rise in water level occurs at those cross-sections which were significantly deepened in 1991 and is where the model predicts there will be large deposition of sediment.
5. The sensitivity tests (simulation run 2) indicate that an increased channel roughness caused by the presence of weeds and excess sediment will cause the water levels to rise. Routine maintenance should be carried out to maintain the original channel roughness. The increase in water level due to increase roughness increases the danger of flooding (see Fig H17 and H17a).
6. With a Mannings 'n' roughness in the channel of 0.035 there is erosion along the upper part of the reach over a period of 15 years. It is unlikely that, due to the sinuous nature of the river the roughness factor would be this low. Deposition of sediment is sensitive to increase in roughness. The model predicts that an increase in Mannings 'n' from 0.045 to 0.05 causes an increase in deposition of 4397m^3 over the whole reach for the same sediment input (Fig H19).

Table H.1. Model predicted and actual deposition of sediment.
Calibration 1

U/S	D/S	1968	1971	1965-71
Chainage (m)	Chainage (m)	Volume (m ³)	Volume (m ³)	Actual Volume (m ³)
3774	3481	-850.19	-962.98	36.99125
3481	3194	-37.14	2.67	200.039
3194	3005	524.522	486.786	186.9683
3005	2731	258.95	396.546	114.669
2731	2432	0	45.461	130.8873
2432	2234	697.553	747.091	148.9455
2234	2060	425.015	432.457	54.723
2060	1969	230.956	304.238	0
1969	1786	-66.96	141.646	0
1786	1509	484.535	759.726	0
1509	1207	-101.08	8.198	95.885
1207	1049	-422.18	-444.21	124.3855
1049	853	238.193	197.213	342.314
853	792	66.553	79.894	99.247
792	603	204.292	199.805	66.19725
603	336	426.377	428.505	0
336	49	-492.95	-632.49	98.5845
	Total Volume	1393.066 m ³	2190.554 m ³	1699.837 m ³

Table H.2. Model predicted and actual deposition of sediment.
Calibration 2

D/S	U/S	1979	1989	1973-89
Chainage (m)	Chainage (m)	Volume (m ³)	Volume (m ³)	Actual Volume (m ³)
3774	3481	-65.85	-33.48	-199.41
3481	3194	-31.19	-13.63	-448.11
3194	3005	48.645	149.887	83.27
3005	2731	120.557	205.176	76.79
2731	2432	224.632	308.023	0
2432	2234	313.03	419.559	7.93
2234	2060	251.532	357.906	125.46
2060	1969	188.042	247.098	357.7
1969	1786	246.186	389.043	410.48
1786	1509	227.309	311.215	787.39
1509	1207	108.835	207.926	253.89
1207	1049	74.698	187.344	94.95
1049	853	28.354	83.072	-7.85
853	792	20.01	149.176	56.35
792	603	-48.29	131.176	628.31
603	336	-69.29	178.463	1069
336	49	-61.29	178.463	597.48
	Total Volume	1956.937 m ³	3758.972 m ³	3726.05 m ³

Table H.3. Model predicted and actual deposition of sediment.
Simulation run 1

D/S	U/S	1994	2000	2006
Chainage (m)	Chainage (m)	Volume (m ³)	Volume (m ³)	Volume (m ³)
3774	3481	-134.43	-109.49	-76.32
3481	3194	897.893	982.014	1018.482
3194	3005	-11.56	117.579	176.477
3005	2731	81.696	223.91	266.979
2731	2432	195.582	346.215	399.605
2432	2234	266.624	399.931	458.078
2234	2060	311.673	311.263	300.029
2060	1969	151.853	245.526	312.254
1969	1786	174.317	398.969	554.41
1786	1509	218.036	347.762	434.354
1509	1207	-186.33	-359.48	-499.88
1207	1049	48.771	143.758	218.764
1049	853	13.21	97.141	173.287
853	792	12.583	51.07	87.609
792	603	-111.88	-126.08	-137.40
603	336	-117.48	13.434	129.674
336	49	-127.89	8.309	174.857
	Total Volume	1682.664 m ³	3091.836 m ³	3991.26 m ³

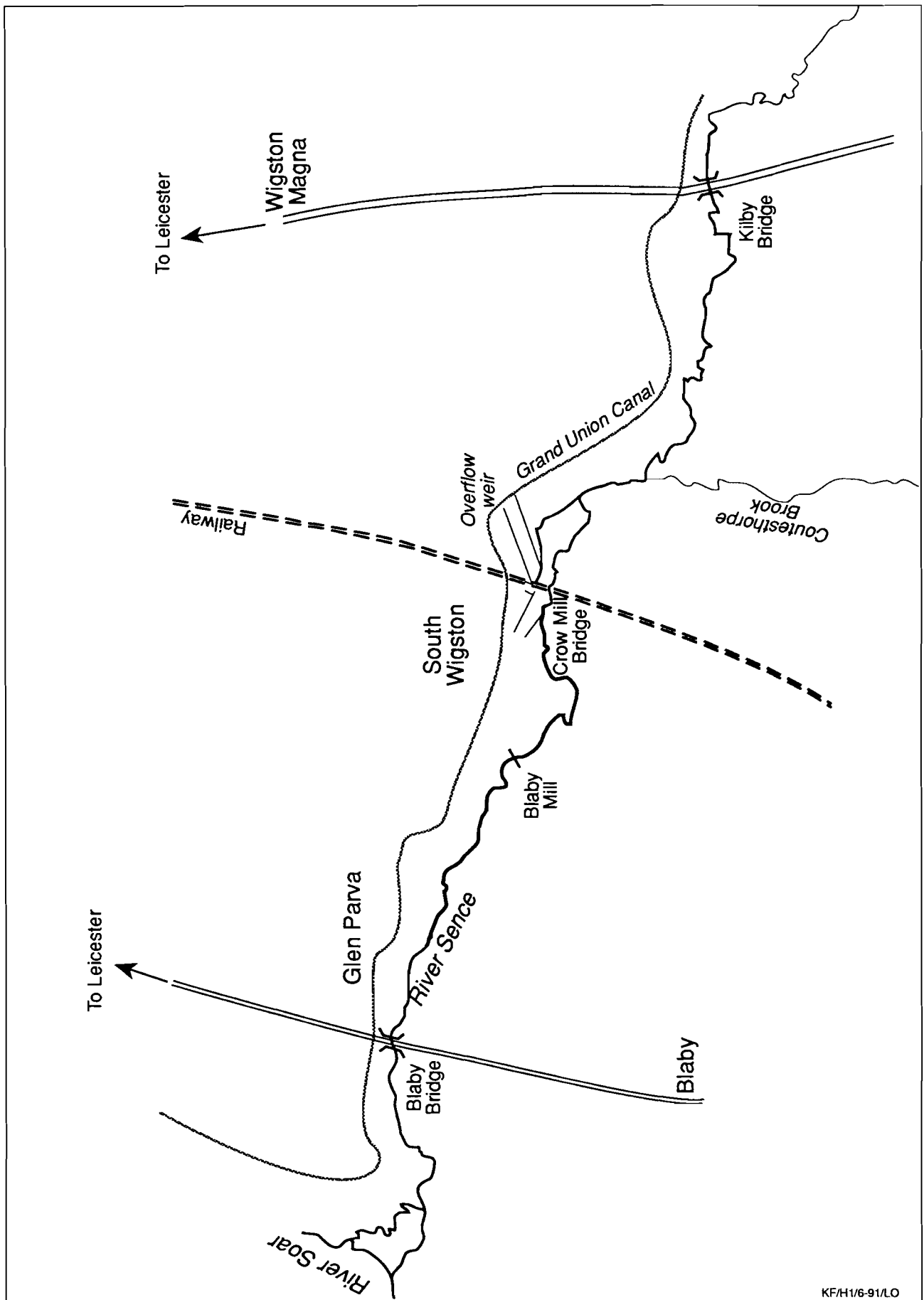


Figure H1 General layout of River Sence: Kilby Bridge to confluence with River Soar

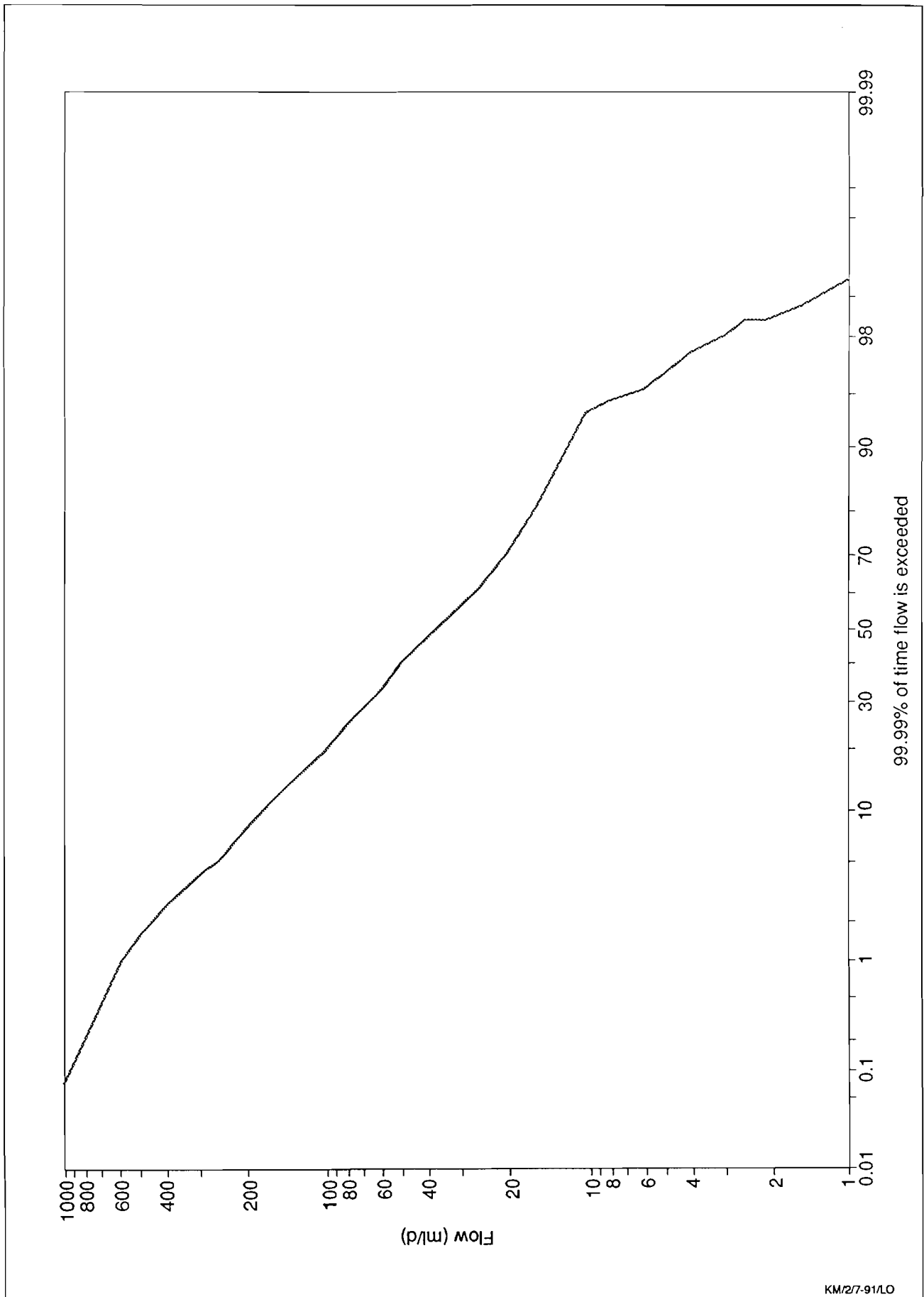
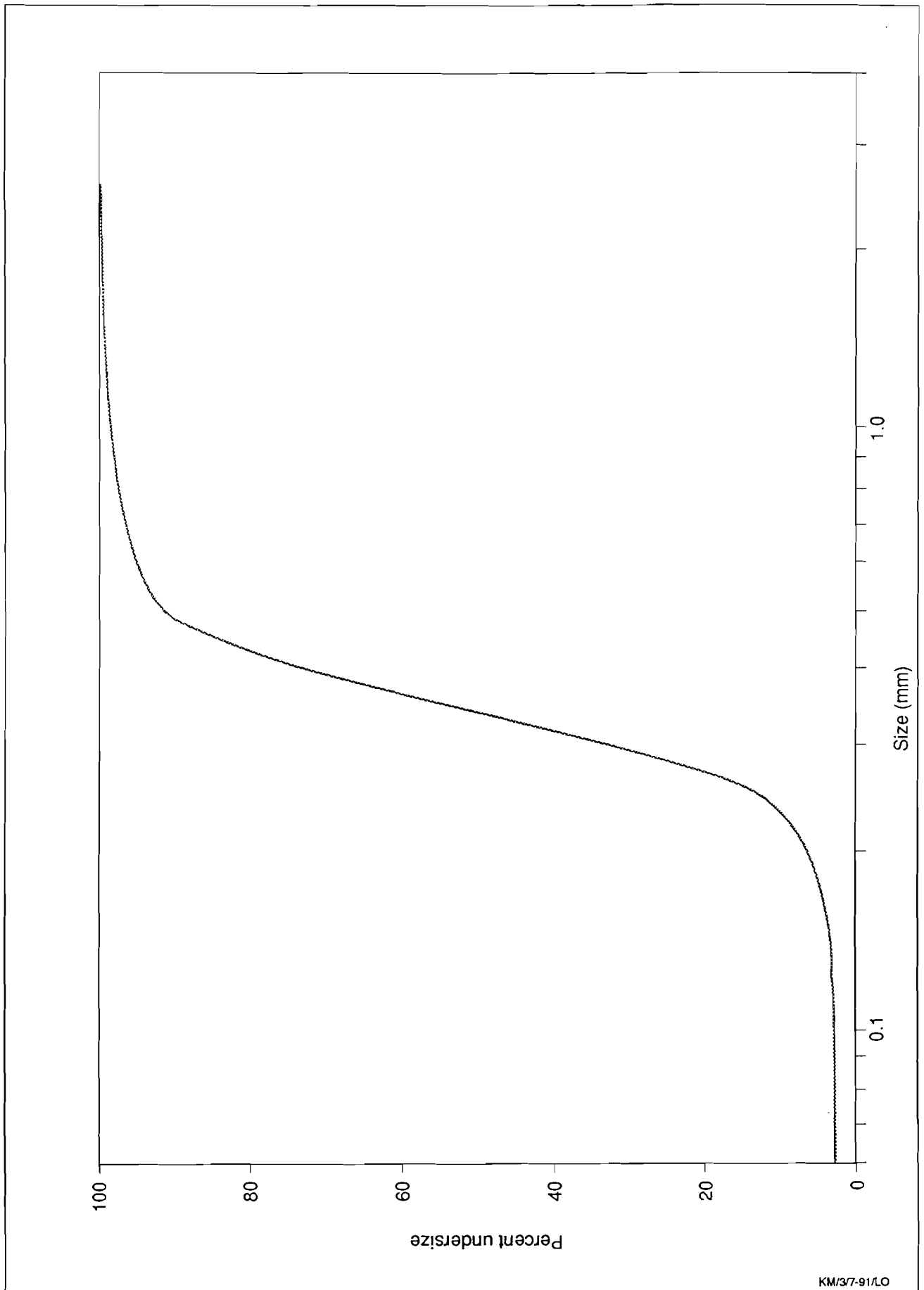


Figure H2 River Sence at South Wigston - flow frequency Jan 84 - Dec 87



KM/3/7-91/L.O

Figure H3 Size grading - sample 1

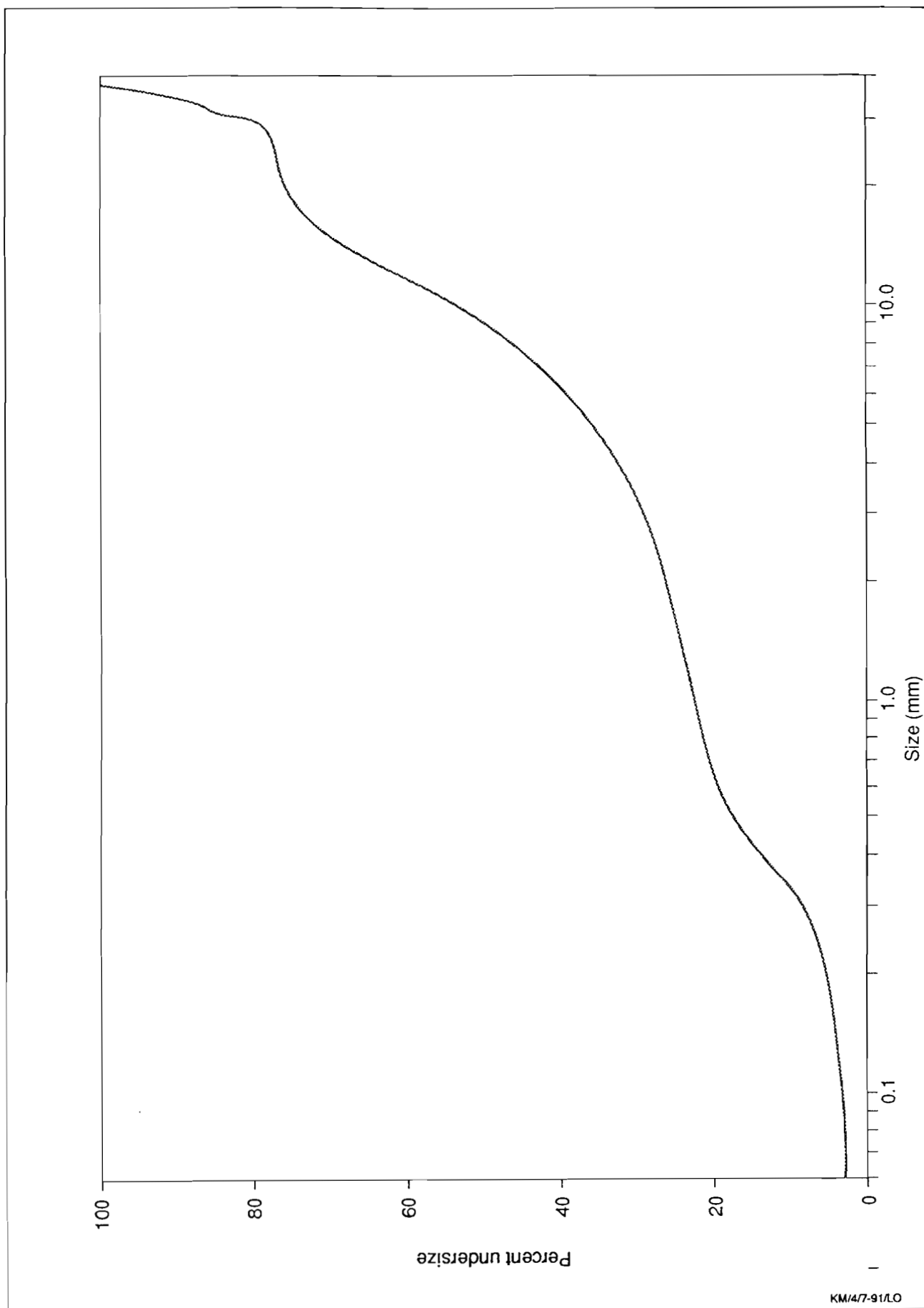


Figure H4 Size grading - sample 2

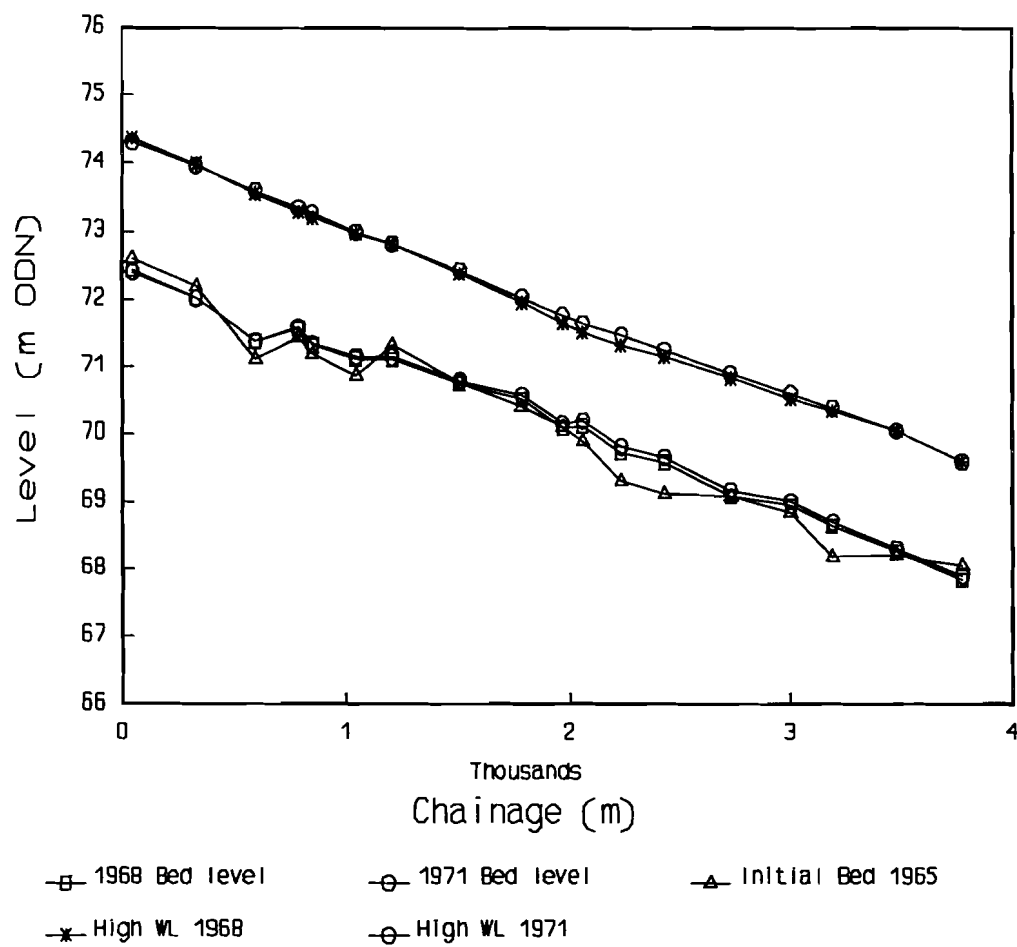


Figure H5 Long profile: bed and water levels. Calibration 1: pre-improvement scheme.

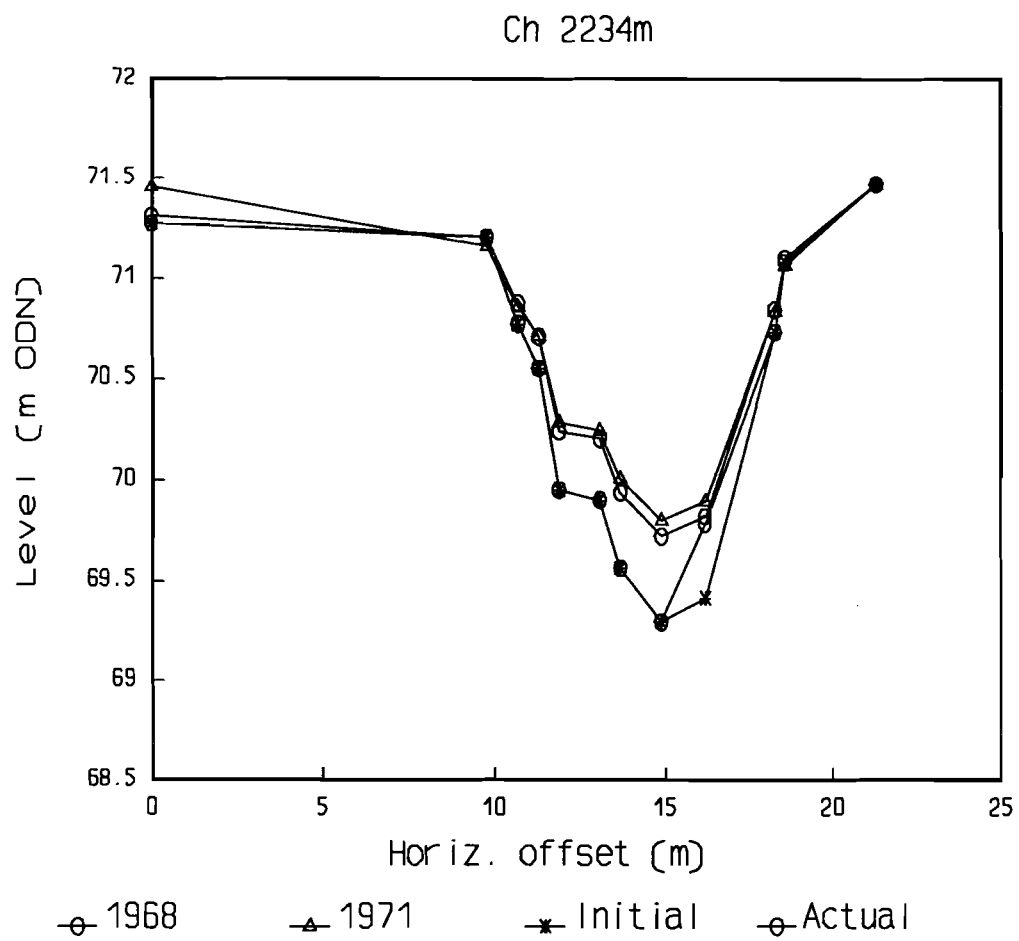


Figure H6 Actual and model deposition for a sample cross-section. Calibration 2.

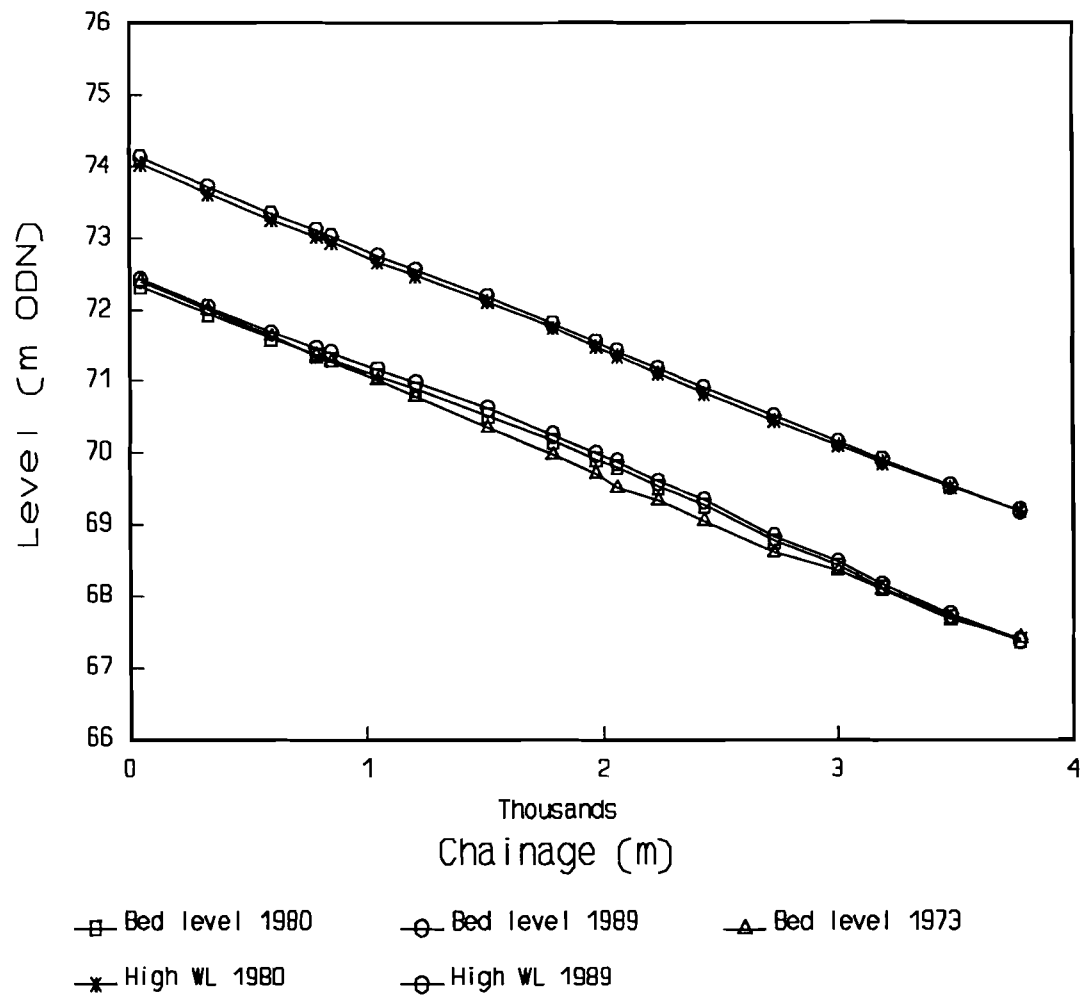


Figure H7 Long profile: bed and water levels. Calibration 2: post improvement scheme.

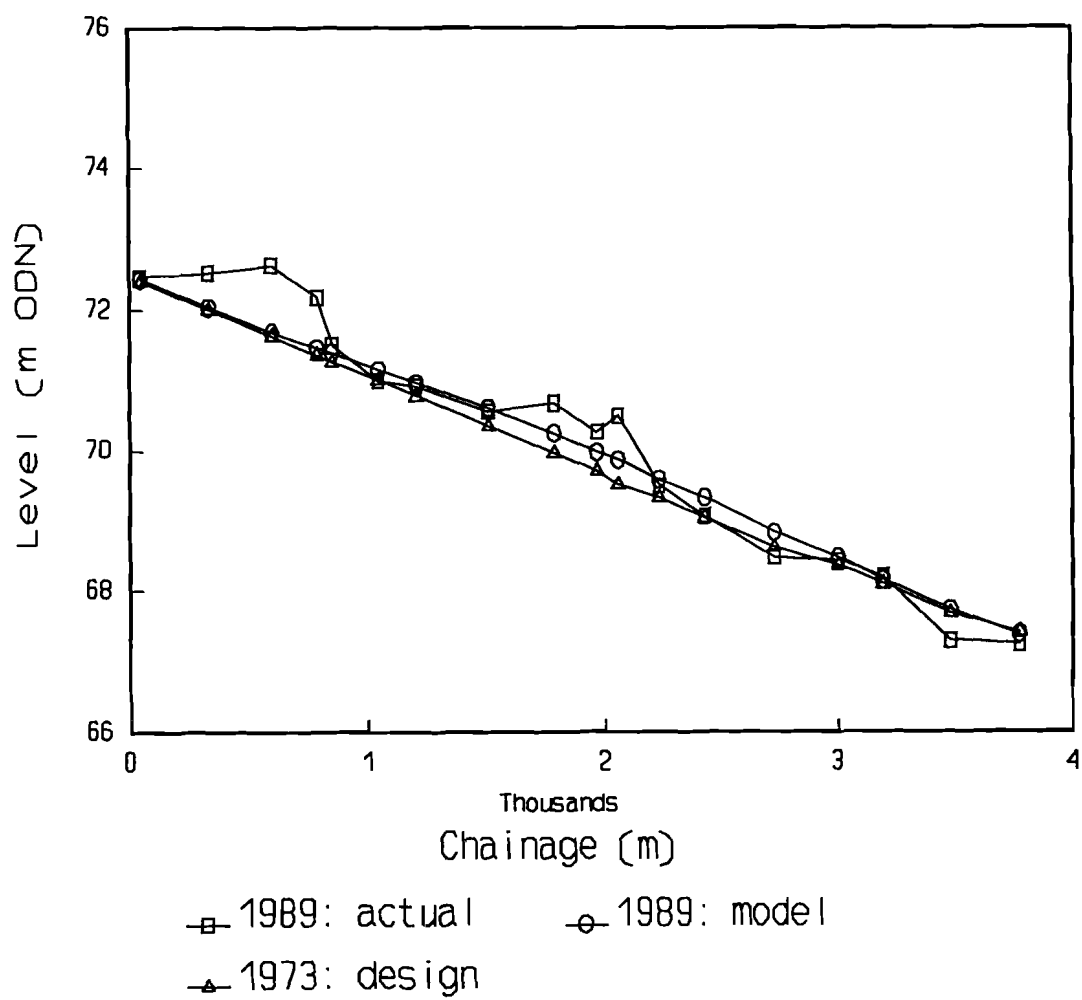


Figure H8 Long profile: actual and predicted bed levels. Calibration 2: post-improvement scheme.

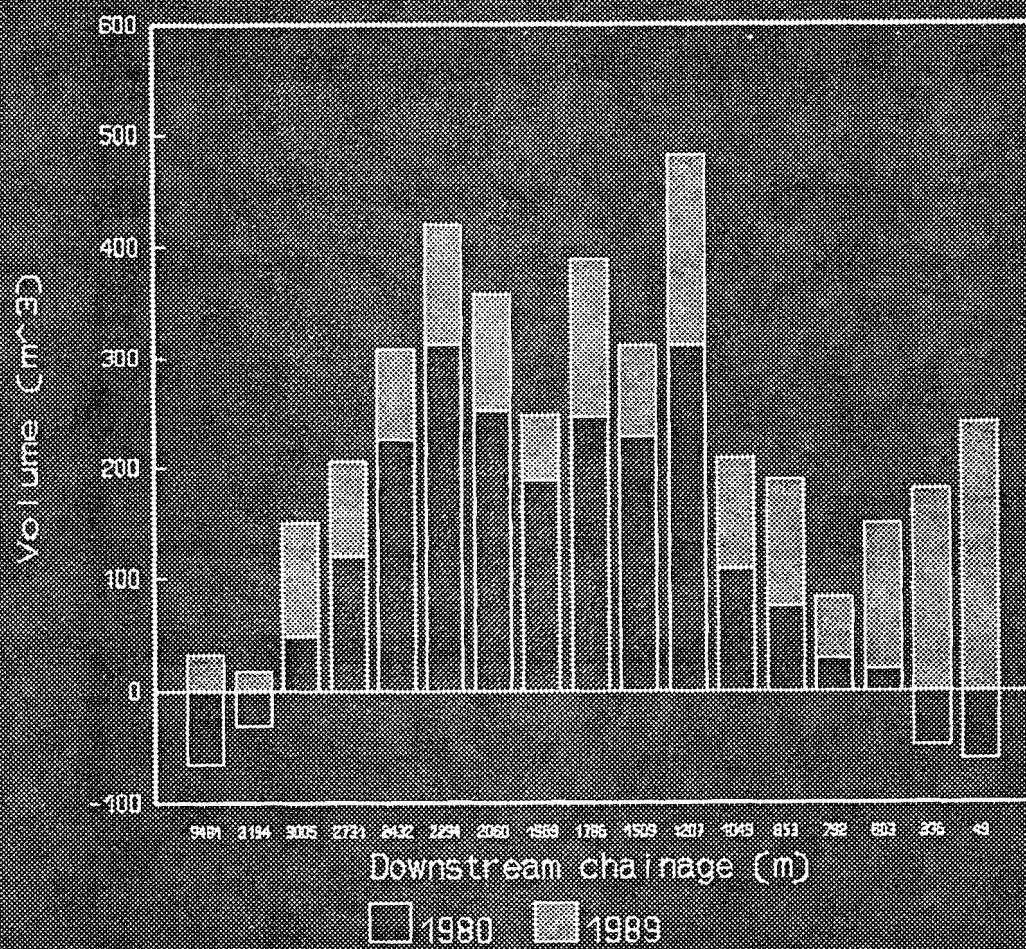


Figure H9 Model predicted deposition of sediment between sections. Calibration 2

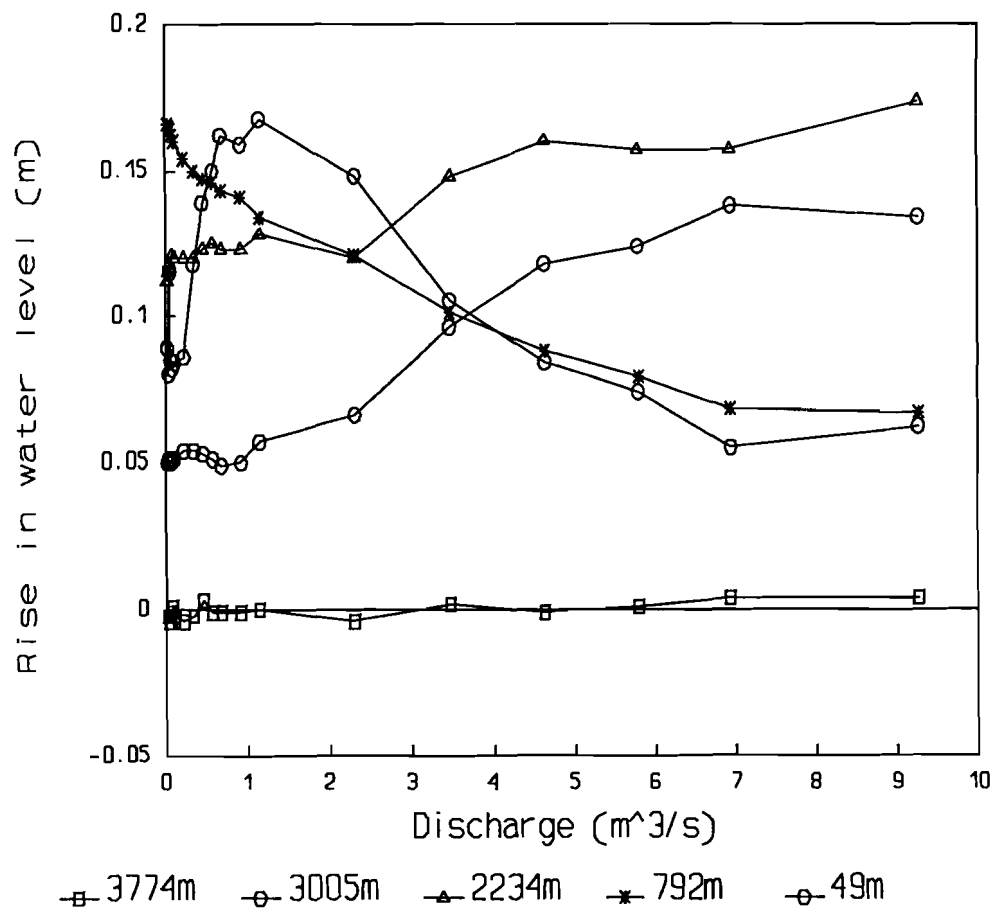


Figure H10 Rise in water level over a 15 year period 1973-1989. Calibration 2.

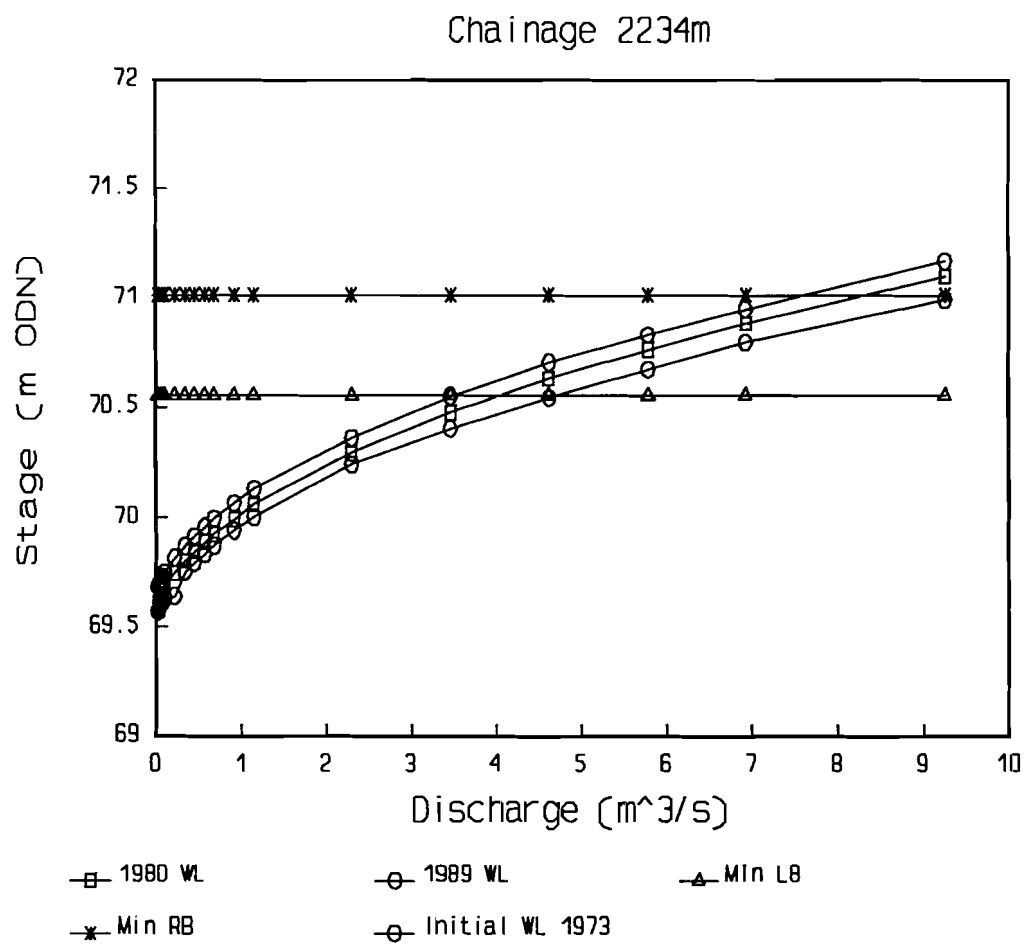


Figure H11 Stage/discharge curve for sample section. Calibration 2.

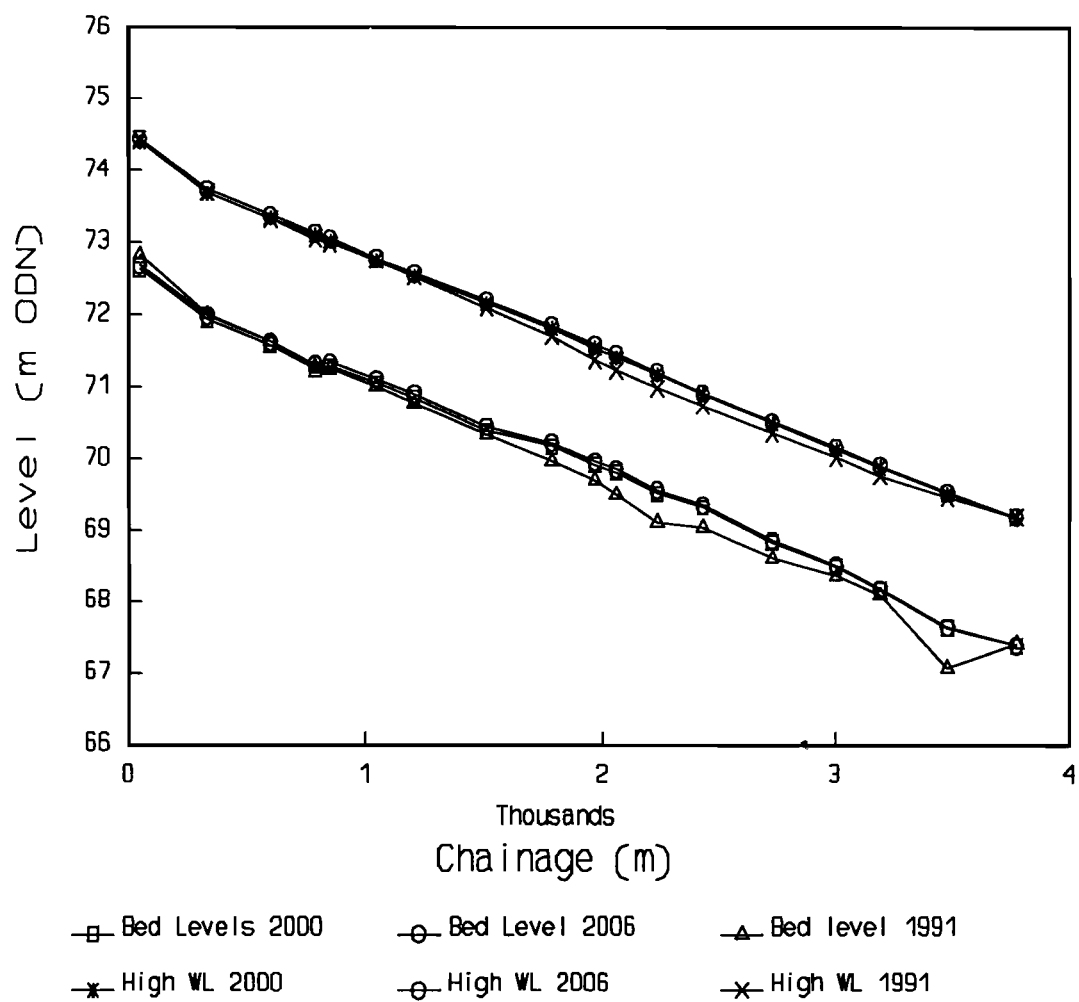


Figure H12 Long profile: bed and water levels. Simulation run 1

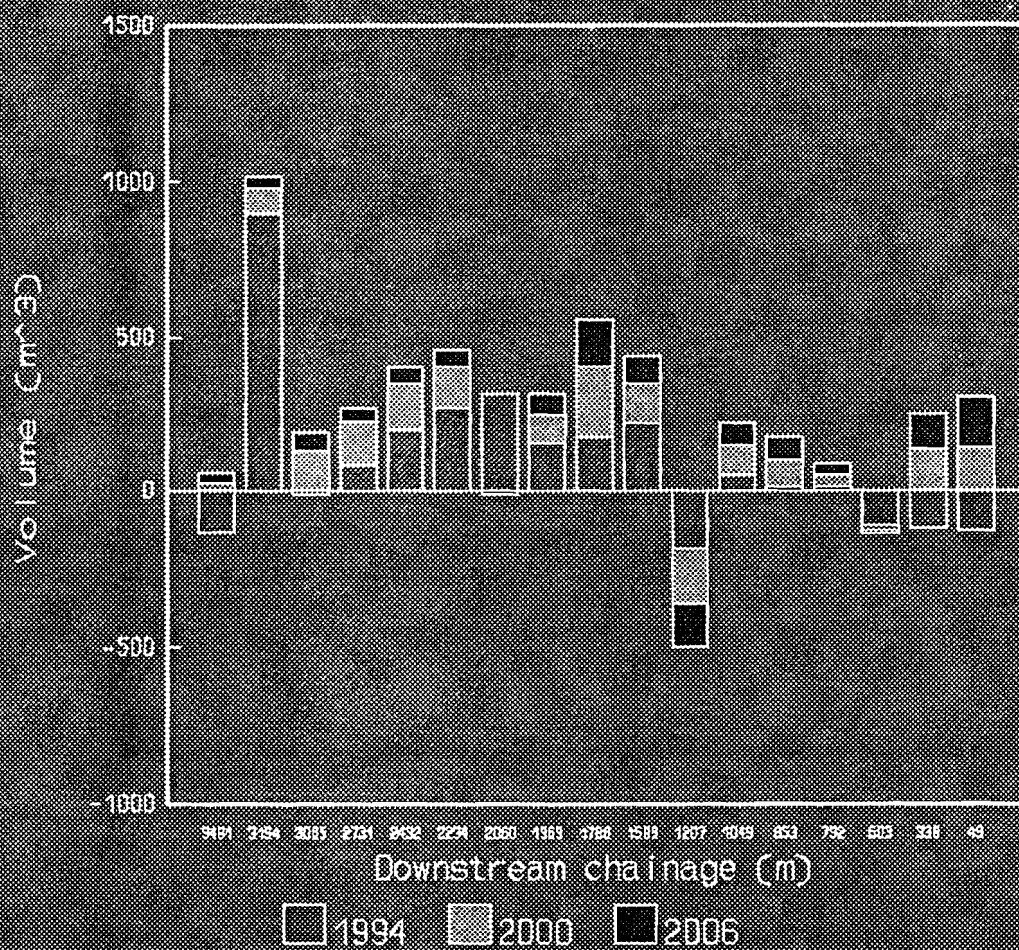


Figure H13. Model prediction of sediment between sections. Simulation run.

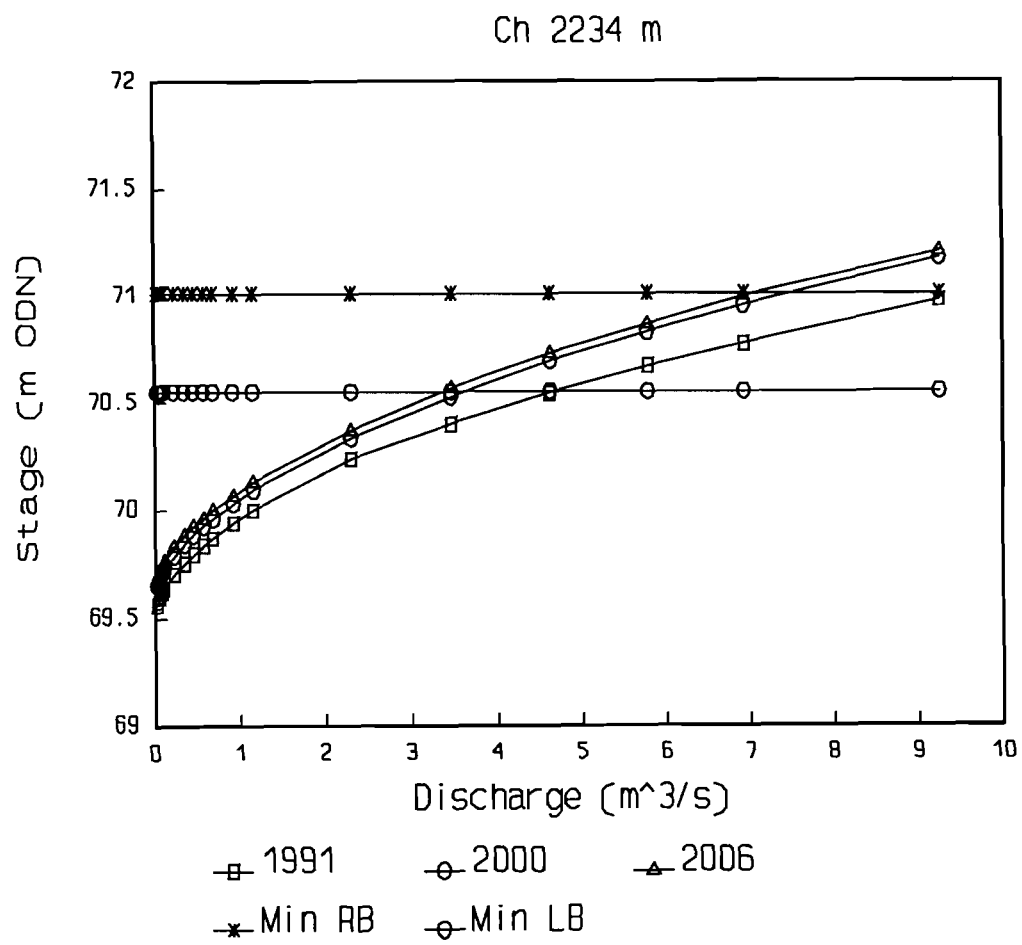


Figure H14 **Stage/discharge curve for sample section.**
Simulation run 1.

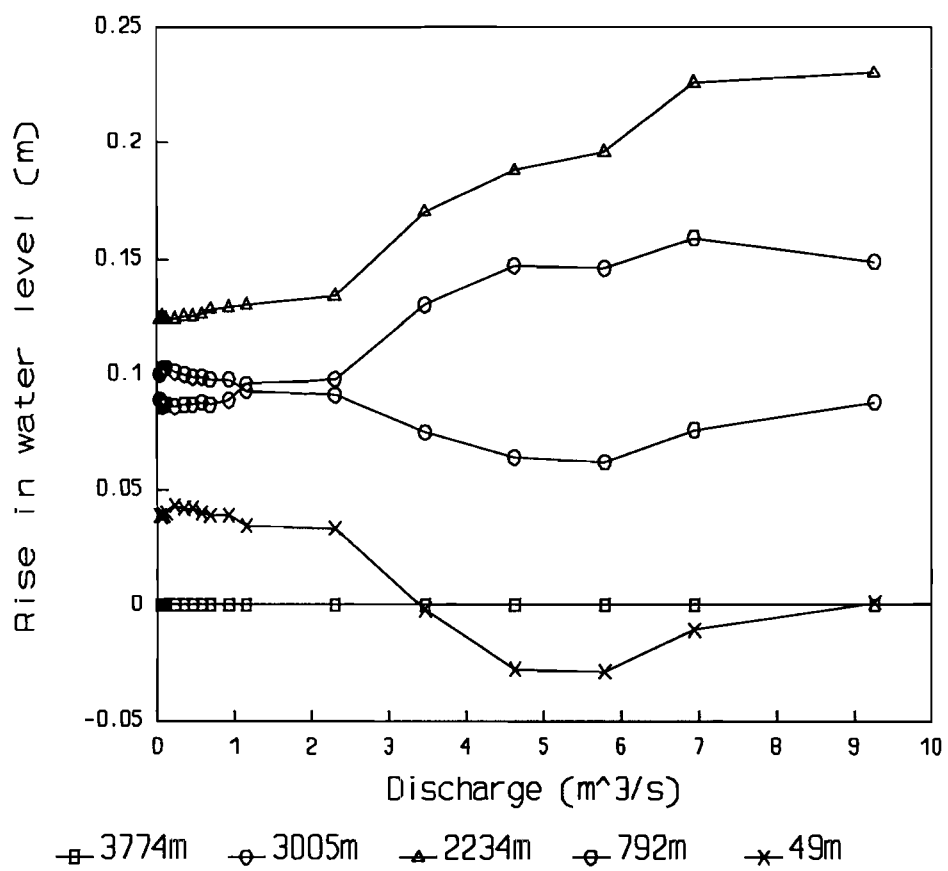


Figure H15 **Rise in water levels for a 15 year period 1991-2006. Simulation run 1.**

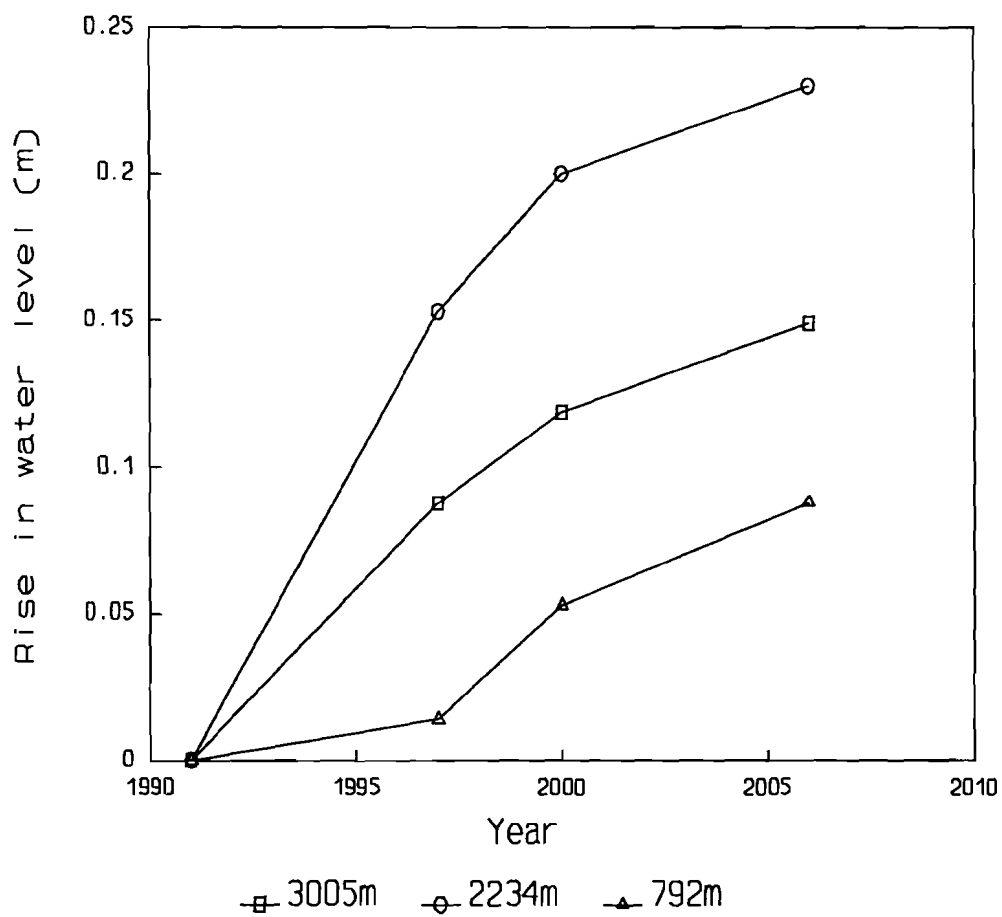


Figure H16 Rise in water level over a period of 15 years: 1991-2006 at different cross-sections. Simulation run 1.

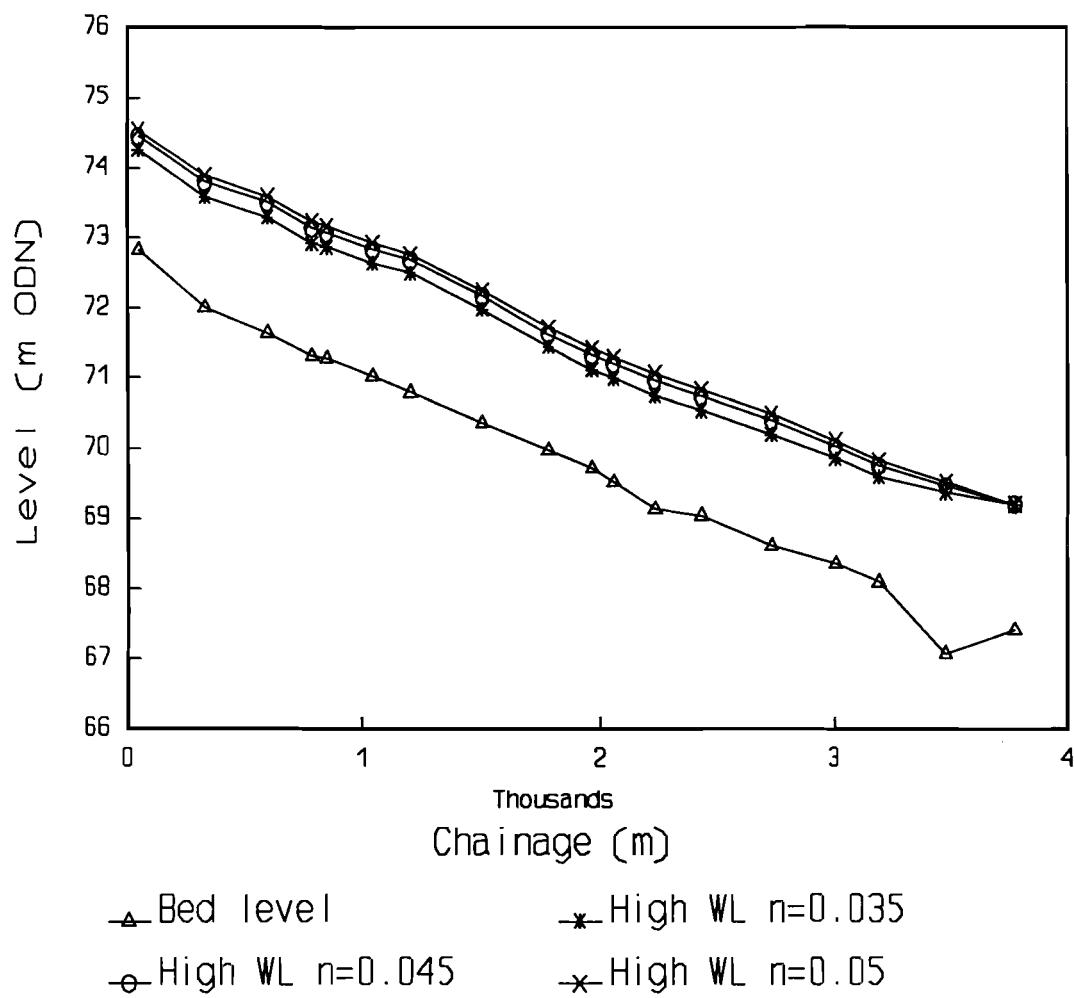


Figure H17 Comparison of water levels at different roughness values and no sediment input. Simulation run 2.

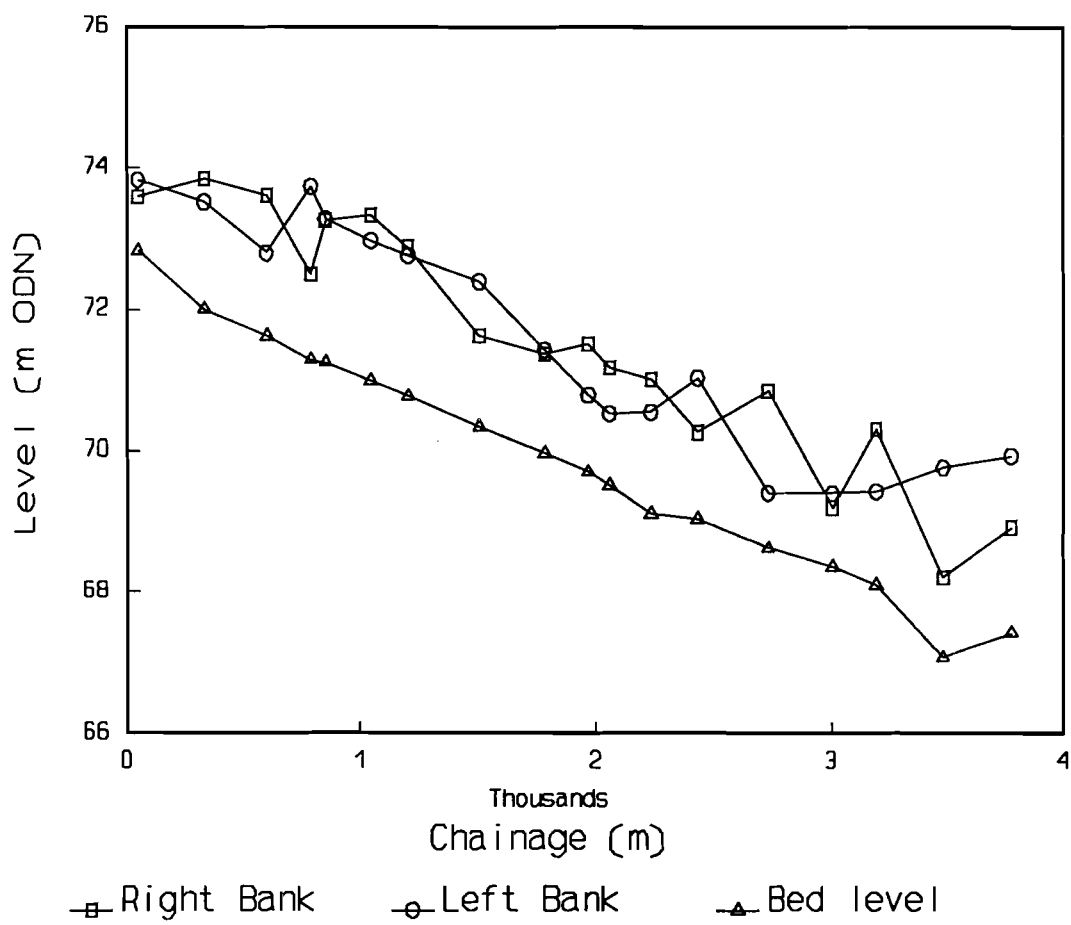


Figure H17a. Left and right bank levels and bed level

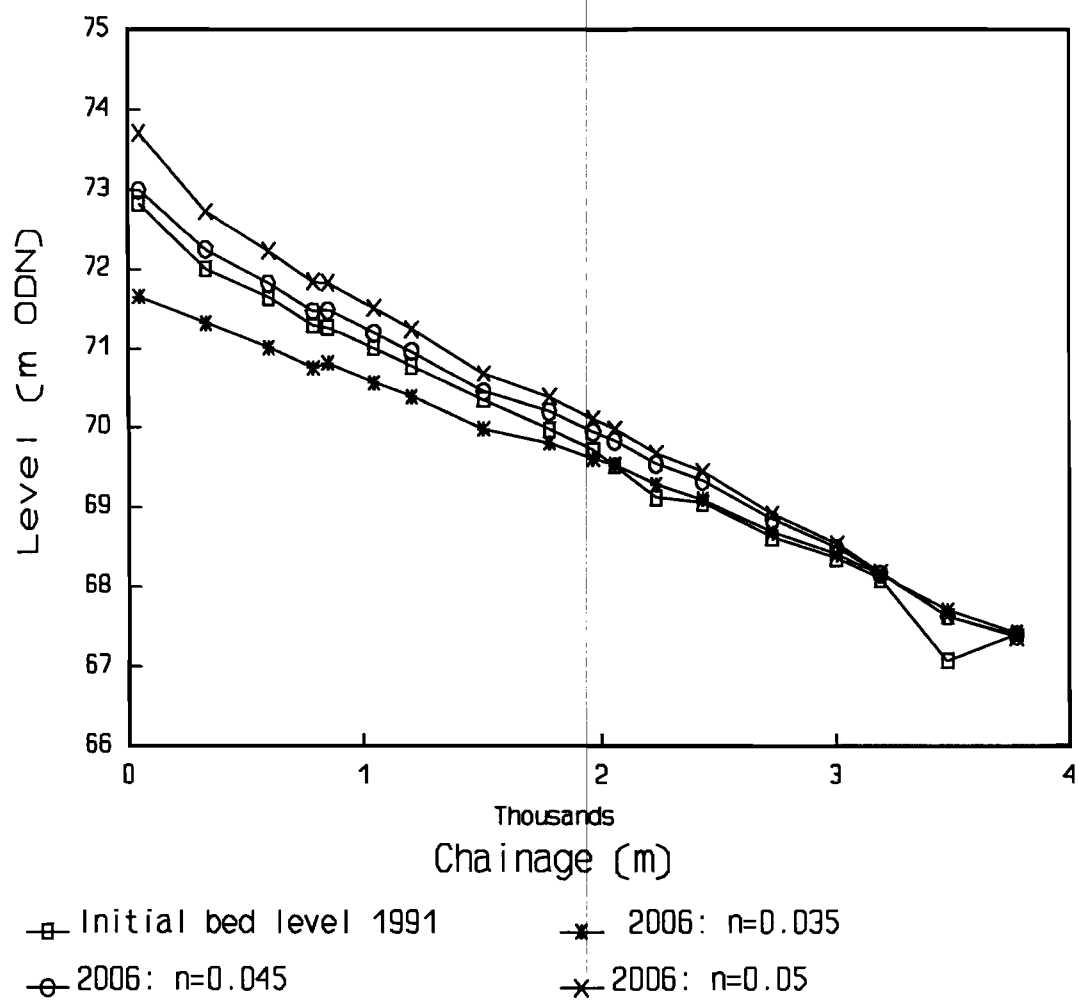


Figure H18 Comparison of bed levels at different roughness values over a 15 year period: 1991 to 2006. Simulation run 2

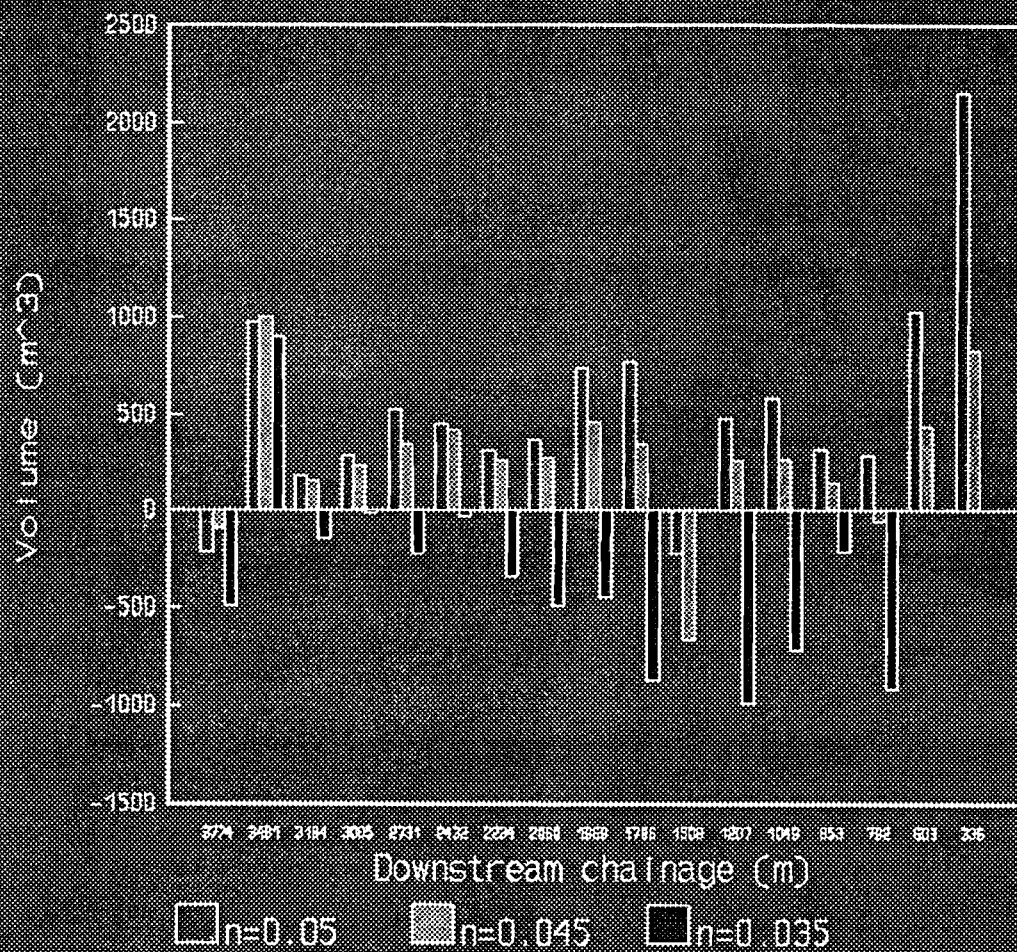


Figure H19 **Comparison of deposition of sediment over a period of 15 years 1991 to 2006 for three roughness values. Simulation run 2**