HYDRAULIC MODEL STUDIES ON SILT EXCLUSION

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الخلاصة :

هذه الدراسة أجريت على نماذج معملية لمشروع الري بوادي داهماد بالمملكة العربية السعودية، بهدف دراسة الخصائص الهيدروليكية للتصميم المقترح لأعمال الري الصناعية لمأخذ ترعة الري الرئيسية من مياه الوادي .

كذلك إهتمت الدراسة بكيفية التغلب على مشكلة الإطماء عند مأخذ ترعة الري .

وكنتيجة لهذه الدراسة . . . تم إختيار أفضل تصمم لمشروع الري ، وذلك عن طريق إنشاء قناة واحدة فرعية بجانب ترعة الري الرئيسية ، تدفع إليها المواد الرسوبية العالقة بمياه الوادي للتغلب على مسكلة الإطماء . . . ويم ذلك بتحسين وتطوير حركة المياه عند مأخذ ترعة الري من مياه الوادي بإنشاء جزيرة صناعية (بغلة توجية للمياه) بين القناة الفرعية والهدار الرئيسي الذي يقوم بحجز مياه الوادي لتحويلها إلى ترعة الري الرئيسية .

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ABSTRACT

A comprehensive series of model tests were carried out in order to investigate factors affecting the sediment exclusion efficiency and hydraulic characteristics of canal headworks and sediment excluders of the type proposed for the Wadi Dhamad Irrigation Project, Saudi Arabia. The investigation was carried out on a model of a representative structure, for which the prototype canal design discharge was $20\,m^3\,s^{-1}$. The tests confirmed that the most important factors affecting the sediment exclusion efficiency and the hydraulic characteristics of canal headworks located at diversion weirs on wadis are: (*a*) the geometric layout of the canal intake and the sluice channel; and (*b*) the downstream water levels of the canal and at the sluice.

As compared with the original design, it was shown that sediment exclusion could be improved by a number of modifications. The most important involved utilizing only one sluice channel and improving the approach flow conditions by forming an artificial island or guide pier between the sluice channel and the diversion weir.

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INTRODUCTION

In the development and rationalization of spate irrigation schemes in the Arabian peninsular, there has been a requirement for many similar diversion weirs on the numerous wadis involved. Special operation and design factors characteristic of the area are: the steep slopes of the wadis; the high peaks and short duration of the flood flows; and the high sediment loads. The weirs which divert the water into the irrigated areas represent a large proportion of the total cost of the development and in order to keep the cost of the weirs to a minimum it is necessary for the head drop across them to be kept as small as possible.

The head drop is not dictated by the requirements of commanding the irrigated land, as it is usually easy to gain command quite quickly because of the steep catchments. However, it was found that sediment exclusion from the irrigation canals was an important factor in determining the necessary head drop across the weir and hence the height of the weir crest above the wadi bed.

Two series of model tests were carried out to achieve a design which gave efficient sediment exclusion, and a limited third series was used to investigate the reduction of efficiency caused by relatively low head differences across the diversion weir.

A number of model studies have been carried out, but very few field investigations have been undertaken into the detailed hydraulic behavior of prototype excluders. Some general feedback has occurred particularly when sediment exclusion was unsatisfactory. There has been a notable general lack of field studies to establish model/prototype conformity. A fairly comprehensive coverage of available knowledge on sediment excluders can be found in References [1–4].

The layout finally adopted was a form of curved channel excluder, but with a converging curved sluice channel. The convergence had some useful effects on the flow in the sluice channel.

Since the purpose of the investigation was to formulate a type which could be used generally at many headworks, no attempt was made to model the characteristics of individual sites. It was assumed that a particular headworks would be sited in a reach of a wadi at a location most favorable from the sediment exclusion point of view; the curved channel excluder represented a second stage of sediment control, within the internal structure of the headworks.

GENERAL NATURE OF SEDIMENT EXCLUSION PROBLEM IN THE REGION

Many of the wadis in the southwest coastal regions of the Arabian peninsular originated in the mountain ranges which run parallel to the coast, and after the occurrence of the infrequent rainstorms the flows enter the coastal plain. Generally, the floods are of short duration, but peak discharges can be very high. Traditionally these spates have been diverted from the wadis for irrigation using earth banks sometimes as high as 6 or 8 m. These are now being replaced by permanent structures able to withstand the infrequent but large peak discharges. During floods the sediment load may be high as 4 per cent of the total discharge in the steeper parts of the catchment, and the construction of reservoirs or ponding structures is not economically attractive because of the speed with which they will silt up. Thus the effective life of a typical reservoir in the region could well be less than 20 years. The short duration of the floods further exacerbates the diversion problem, as the structures must be able to divert relatively large flows into the canals in order to irrigate the fields in the short time that water is available. At the same time, the high peak discharges combined with the need to minimize the cost of the structures means that only a small head drop can be available across each diversion weir.

Against this background, there is the almost universal problem of sediment exclusion at an irrigation canal headworks, arising because the lower velocity flows in the irrigation canals are not able to transport sediment at as high a concentration as is sustained in the faster-moving parent river. No general solution has been found to this problem since it is to a large extent dependent on climatic and morphological characteristics in the particular area under consideration. Some factors relevant to design are: (1) river flow variability; (2) sediment transport rates in the river; (3) availability of water for sluicing purposes; (4) availability of head for sluicing purposes; and (5) river mobility, such as the tendency of rivers or low-flow channels to shift course.

In the region considered, the wadi flows transport mostly sand and gravel, although for the majority of the time the wadis are dry and typical storm flows last for only one or two days. Peak flows occurring for only a few hours are followed by an extended recession. The situation changes rapidly from times when (*a*) only sufficient water is available to supply the canals with less than full supply discharge; (*b*) sufficient water is available to supply the canals, with some water available for sluicing, and (*c*) adequate water is available to supply the canals and for full sluicing, although at this stage the sediment concentration in the wadi tends to be higher due to the higher flows.

It was appreciated that with only a small head drop available there would be some conflict in the design most appropriate for all three stages (*a*), (*b*), and (*c*), and this was part of the problem to be investigated. However, the longer period occupied by the recession did suggest that a design which was satisfactory for (*a*) and (*b*) could be acceptable provided performance for (*c*) was not completely unsatisfactory.

DIVERSION WEIR AND HEADWORKS DESIGN

The diversion weirs are located in the foothills and in the coastal plains where the wadis are relatively broad, with beds consisting mostly of sands and varying amounts of coarser materials up to boulder size. The location plan of a typical structure is shown in Figure 1. The weirs have traditionally been constructed of concrete, although more recently alternative types of weir construction have been considered such as permeable face protection using tipped rock. The weirs can be very long—up to 1500 m. This typical diversion structure was used as a basis for the model investigation, having left bank and right bank canal flows of 20 $m^3 s^{-1}$ and 15 $m^3 s^{-1}$ respectively. Corresponding to the sluice gate sizes and arrangements envisaged in a preliminary design, and allowing for the maximum anticipated tailwater levels in the wadi, it was calculated that flows through the sluice at the left and right bank headworks would reach 14 $m^3 s^{-1}$ and 10 $m^3 s^{-1}$, respectively, before the diversion weir was overtopped. Once the diversion weir was overtopped, the sluice flows would increase only slowly due to a rise occurring in the wadi tailwater levels as well as in the water levels upstream of the weir. Tailwater level cannot be predicted very accurately owing to the backing-up effect of shoals and banks at relatively low flows, and to the possibility of degradation downstream of the diversion weirs. During peak flood flows, the wadi will flow full, but at the lower discharges low-flow channels will develop, the positions of which will be variable and unpredictable, especially in the first one or two years of operation of a diversion weir, and different approaches to a canal headworks along the face of the weir would be quite common.

The headworks for the left-bank canal of capacity 20 $m^3 s^{-1}$ was selected for detailed study. The preliminary layout for this headworks can be seen in Figure 2, which of course shows mainly the canal headworks and the sluice channel, only a short length of the 250 m long diversion weir being included in the Figure. A typical section through the weir is shown in Figure 3*a*, and a section through the sluice channel in Figure 3*b*. The preliminary design envisaged a sluice

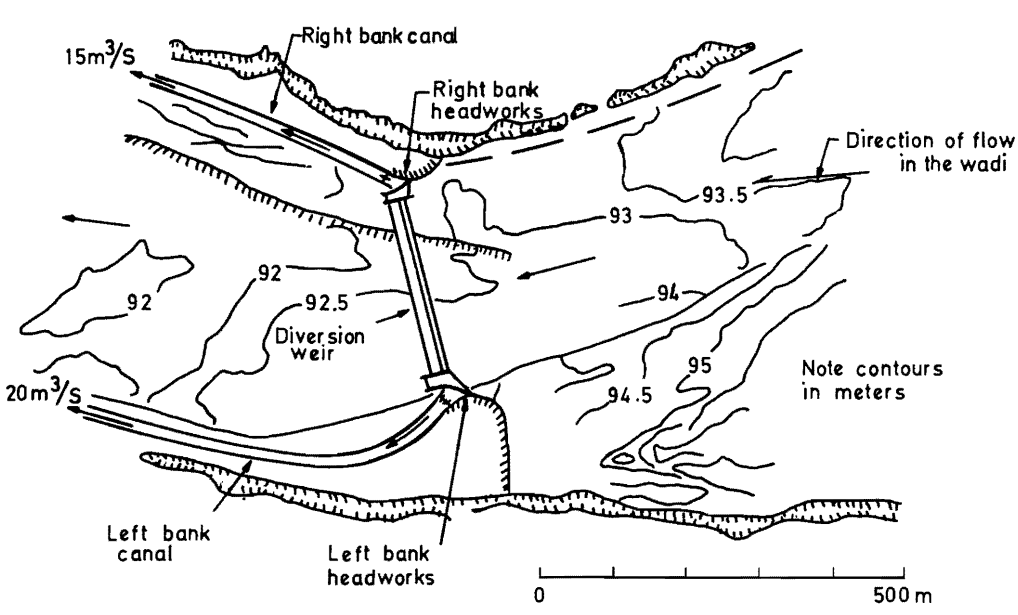

Figure 1. Location Plan for Wadi Dhamad Diversion Weir

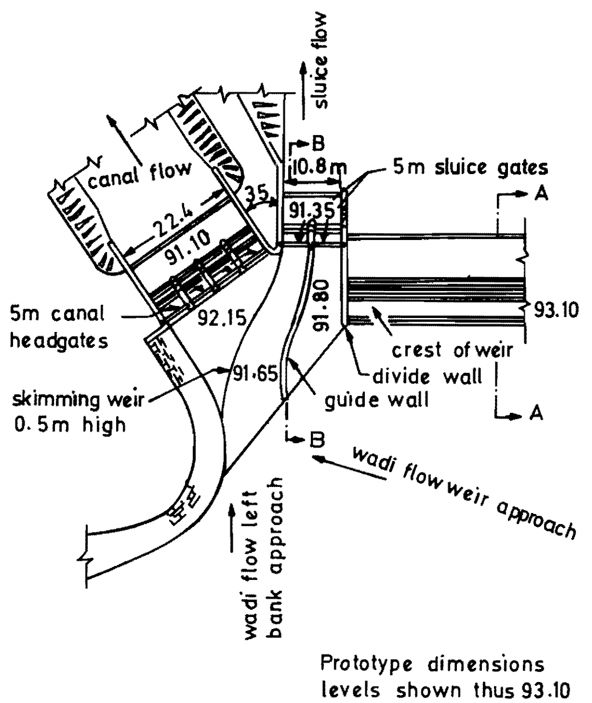

Figure 2. 20 m³ s⁻¹ Headworks: Preliminary Layout

channel, generally 1.45 m below diversion weir crest level, leading to two 5 m wide by 1 m deep sluice gates. The sluice channel was divided approximately into two by a 1.45 m high guide wall, the top level of the wall being at the same height as the diversion weir crest. The guide wall is to be distinguished from the divide wall, which was carried up above flood level and separated the sluice channel from the diversion weir. Between the sluice channel and the approach to the canal headgates (at level 92.15) there was a 0.5 m steep, which was loosely termed a skimming weir. As will be noted, the sluice channel was not very deep, for the purpose of keeping the diversion weir crest level as low as possible.

In operation, it was intended that when adequate flows were available to feed the left-bank headworks, the sluice gate would be fully open. Gravel and boulders would pass down the sluice channel, together with a considerable proportion of the sand fraction of the bed load. As the available wadi flow reduced, it would be necessary partly to close the sluice gates in order to maintain the inflow into the canal at $20\,m^3\,s^{-1}$. When the available wadi flow dropped below $20\,m^3\,s^{-1}$ the sluice gates would normally be shut, except as required for occasional flushing. The low flows occurring in the wadi at this time would probably not transport much gravel, and any gravel and some of the sand in transport would be temporarily deposited in the sluice channel.

PHYSICAL MODEL LAYOUT, SCALING AND TEST PROCEDURES

The movement of sediment in the region of a complex structure such as a sediment excluder is so complicated that at the present time it can only be studied by means of a physical model. Because of the considerable length of the diversion weir (about 1200 m), it was clearly impossible to model the whole weir and headworks to an acceptable scale. The left-bank headworks was therefore modeled, together with the adjacent part of the diversion weir. In curtailing the diversion weir in this way, the importance of the

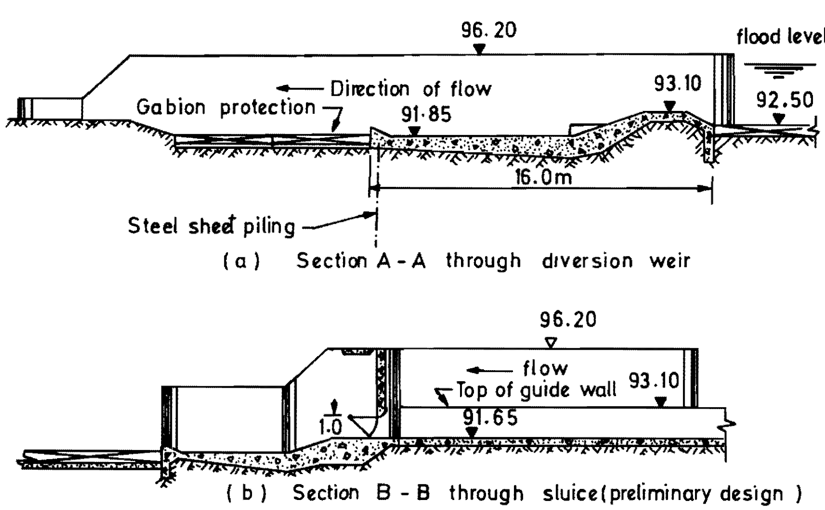

Figure 3. Sections Through Diversion Weir and Sluice

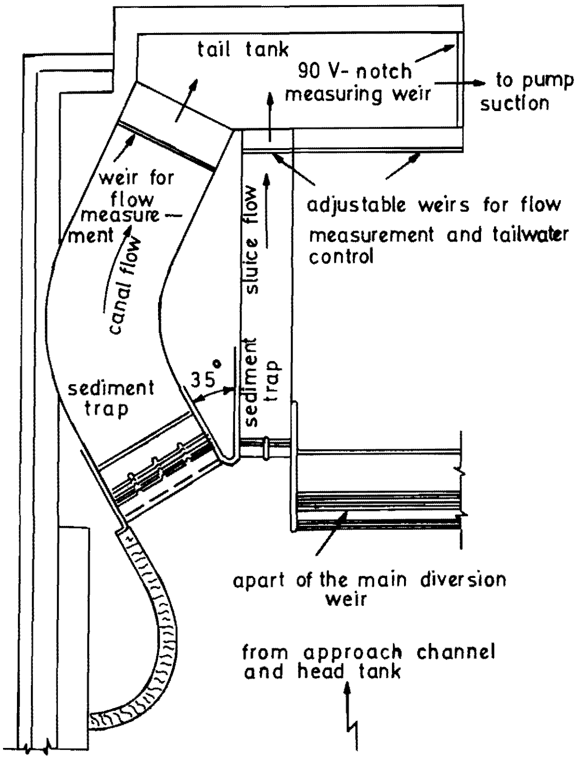

Figure 4. General Layout of Model Area for Wadi Dhamad.

actual direction of approach of the flow to the headworks was not forgotten, and this was varied during the testing programme. The general layout of the model is shown in Figure 4. With the space available, a horizontal scale of 1/30 was appropriate. Since the flow through the headworks involved gravity phenomena such as flow over weirs and sills and through gates, Froudian scales were used in modeling the structure and the corresponding discharges.

Many sediment modeling problems involved simulating the behavior of a river or channel with mobile beds and boundaries. In such cases, correct simulation of the variable bed formations and bed roughness is a major part of the similarity requirements. However, in the present investigation this was not considered to be the major similarity problem. Except when the sluice gates were shut, it was anticipated that sediment movement in the sluice channel would be in the form of isolated particles or small groups of particles, sliding or saltating along the concrete bed of the sluice channel. It was evident that movement of sediment over the skimming weir would necessarily be in suspension, although subsequently some of the particles would fall to the concrete bed of the platform approaching the canal headgates and move by sliding or saltation. Similarity in suspension and in movement over solid surfaces were based on the parameter w/v^*, where w is a characteristic fall velocity of the sediment particles and v^* is the shear velocity $\sqrt{\tau_0/\rho}$. In this expression τ_0 is the bed shear stress and ρ is the fluid density. A prototype sand size of approximately 0.4 mm was assumed to be reasonably representative of the sand beds of the wadis in the region, and taking into account the model and prototype water temperature conditions, a granulated bakelite having a specific gravity of 1.39 and a median diameter of approximately 0.5 mm was found to satisfy the modeling criterion sufficiently closely.

In order to assess the effectiveness of any proposed design changes, it was necessary to have quantitative measures of the sediment entering the canal, and of the sediment passing through the sluice. The arrangement shown in Figure 4 worked satisfactorily. Referring to Figure 4, the flow from the head tank passed over a bed of bakelite about 6 m long, in which at the lower discharges a low-flow channel developed. At higher flows the whole area carried flow, as would occur in the prototype.

The slope of this area could be adjusted to ensure a satisfactory channel, with a sediment load representative of the wadi flows. The flow in the channel was measured by a simple rectangular weir and the area upstream of this weir was used as a sediment trap. Only very fine sediment passed out of the trap and this was measured by occasional sampling of the outflow. The areas downstream of the sluice gates and downstream of the length of the diversion weir were also used as sediment traps and were followed by adjustable measuring weirs which were used to simulate possible tailwater levels in the wadi downstream of the diversion weir. The right-hand wall of the sluice channel sediment trap was not a feature of the prototype and was incorporated in the model in order to form a flow measurement channel and sediment trap.

Typically, test runs lasted one hour, as this time period usually resulted in adequate volumes of sediment being trapped. Because of the inevitable scatter of results most tests were duplicated, with higher and lower sediment concentrations in the approach channel. As well as the effectiveness of sediment exclusion, head losses were also of interest, through the approaches to the headworks and through the canal headgates, etc. Even at high floods the approach to the headworks itself will be relatively low and it was considered that boulders and gravel were unlikely to jump the skimming weir, and so could be omitted from the model investigation.

A further consideration was that during the passage of a number of floods, the low-flow channels would reform, so that the direction of approach to the canal headworks and sluices would vary unpredictably. This was allowed for in the model studies by deliberately initiating flows having substantially different directions of approach to the headworks. There seemed no reason to investigate the exclusion of very fine material approaching wash load size, as this would easily be carried through the canal system with a resulting benefit to the fields from nutrients and improved soil structure.

In making comparisons of performance, it was necessary to standardize conditions, always with sufficient flow for full supply to the canal, and with varying amounts of sluice flow and varying tailwater levels in the wadi downstream of the weir.

Series 1 Tests: Optimization of the General Configuration of the Sediment Excluder

This investigation was carried out using the physical model described earlier. The preliminary layout, shown in Figure 2, was used as a starting point. Most of these tests were carried out with the water level upstream of the diversion weir just at weir crest level, resulting in $20\,m^3\,s^{-1}$ entering the canal and $7\,m^3\,s^{-1}$ passing down each sluice channel when the corresponding sluice gates were fully open (discharges are quoted in prototype values). The skimming weir was intended to form one boundary of the left-hand sluice channel, as well as a step which would inhibit the passage of gravel and boulders.

During the course of these tests, various cross-sections were tried for the skimming weir, but it was found that none of them really gave better performance than a simple 0.5 m step.

Considerable guidance for effective layouts was obtained by observing the effects on sediment paths of changes (e.g. to guide walls) carried out in a quick and temporary way. Although these short-term changes could not be recorded quantitatively, such observations were probably the most productive part of the optimization process.

In the first place it was necessary to establish a basis for comparison and so a reference set of tests were carried out on the preliminary layout, with different sediment concentrations in the wadi flow, different directions of approach of the scoured channel, high and low tailwater levels for the sluice flow, etc.

Early tests on the preliminary layout showed that sediment entry to the canal with both sluice gates operating (i.e. canal flow $20\,m^3\,s^{-1}$, each sluice gate discharging $7\,m^3\,s^{-1}$) was very much worse than when only the left-hand sluice gate was operating (at $7\,m^3\,s^{-1}$). The reason for this was that under the former conditions the action of the right-hand sluice gate was to attract the scoured channel towards the right (i.e. towards the diversion weir). A considerable part of the flow moving towards the canal and the left-hand sluice gate was then forced to make a very tight turn around the upstream nose of the guide wall. The resulting vertical vortices took sediment up into suspension, and this sediment moved with the upper part of the water flow into the canal. The slower-moving bottom water turned into the left-hand sluice channel. Subsequently during the tests on the preliminary layout, the right-hand sluice gate was kept shut, and the majority of the tests were carried out with a sluice discharge of $7\,m^3\,s^{-1}$. When this was done, it was apparent that the direction of approach of the flow to the headworks was still important. At various times after the construction of the diversion weir, the flow would approach the canal headworks along the upstream face of the weir and observations on the model showed that this approach condition led to poor sediment exclusion from the canal, for essentially the same reasons as described above in connection with the right-hand sluice channel. The model observations also showed that a pronounced left-bank approach to the headworks resulted in poor sediment exclusion from the canal, because the anti-clockwise curvature of flow around the left guide bank caused sediment to move towards the left and hence into the canal.

As the result of observations on the preliminary design, it was decided to use a single sluice channel with a downstream width corresponding to two 3.5 m wide sluice gates. A right-hand guide wall was formed by introducing a curved pier between the sluice channel and the main diversion weir, and the right-hand curvature of the sluice channel in front of the canal approach platform was increased. The shape of the sluice channel as modified can be seen in Figure 5. Effectively a converging curved channel had been formed, with right-hand curvature, followed by a reverse curve leading to the sluice gates. The plan shape of the sluice channel was chosen partly by observing the effects of changes to the boundaries, carried out rapidly. However, it is evident that the converging shape of the curve sluice channel had some hydraulic advantages. First, as the discharge in the sluice channel decreases along its length, the velocity on the channel is maintained because of the reduction in the cross-section area of flow. Secondly, some recent work

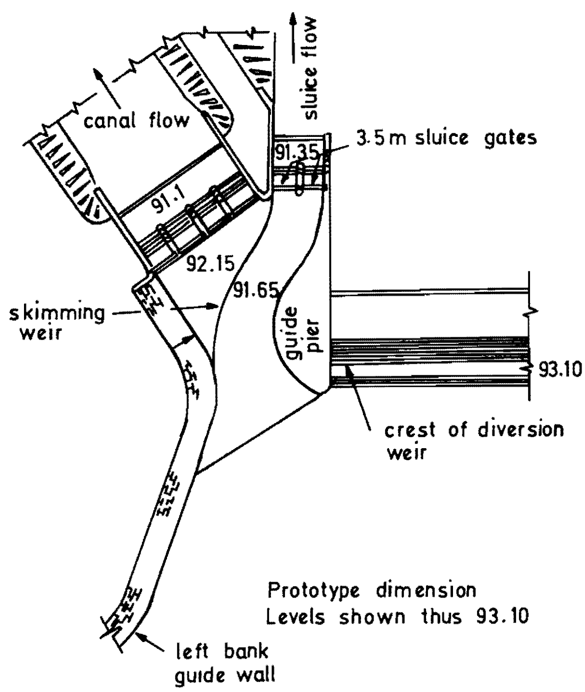

Figure 5. 20 $m^3 s^{-1}$ Headwork as Modified

[5] has shown that, in a curved converging channel, the thread of maximum velocity remains close to the inside curve of the channel, in contrast to the behavior in concentric curved channels. This means that as the bed load is moved transversely towards the inside curve of the channel, there is a relatively high velocity flow which transports it towards the sluice gates.

As well as these changes to the sluice channel, a left-bank guide wall was formed, extending upstream, so as to prevent a pronounced left-bank approach. These modifications resulted in substantially improved performance. However, with a pronounced weir approach, the situation still arose in which the flow moving towards the canal had to make a very sharp turn around the upstream nose of the guide pier, resulting in the same unfavorable condition as encountered before, i.e., a large proportion of the sediment entered the canal rather than moving into the sluice channel. An obvious way to avoid this very unfavorable approach condition was to move the main diversion weir upstream relative to the position of the headworks. A similar movement of the diversion weir in an upstream direction had in fact been recommended as the results of model studies carried out in connection with the Diyala Diversion Weir, Iraq [4]. After carrying out these modifications, shown in Figure 5, the percentage of the total sediment carried towards the headworks which entered the canal was—in broad terms—reduced from about 60 to 30 percent, a very significant improvement. No doubt further improvements could have been made if time and budgets had allowed an extensive programme of further tests, but this was not possible. However, any further improvements would show diminishing returns and must be considered in the context of the other assumptions necessary in the testing programme.

Series 2 Tests: Required Minimum Sluice Flow

In optimizing the plan-form of the typical headworks it was not possible to vary too many parameters—otherwise it would have been impossible to detect the effect of specific changes. One major parameter which was kept constant in most tests was the quantity of water used for the sluice flow, which was normally maintained at 7 $m^3 s^{-1}$ prototype. This discharge was maintained by operation of the sluice gates, whether the sluice outlet was drowned or not. The particular sluice flow of 7 $m^3 s^{-1}$ was selected partly by observations on the model, which could easily show whether there was a strong sluice action or not, and partly on the basis of some experience which suggested that the sluice discharge should not be less than about one-third of the canal discharge. At least the value selected seemed sufficiently realistic for the purposes of the Series 1 tests.

However, in view of the variability of the wadi flow and the period of time which was occupied by recession flows, it was clearly of interest to investigate further the effect of the magnitude of sluice flow on sediment exclusion efficiency. The Series 2 tests represented the condition where there was sufficient flow in the wadi for full supply discharge in the canal, and varying amounts of sluice flow. The canal discharge was kept constant at 20 $m^3 s^{-1}$ and the sluice discharge was varied (with the upstream water level at diversion weir crest level), usually by varying the tailwater level downstream of the sluice outlet. It was assumed in this series of tests that the canal headworks would have been sited as favorably as possible with ɪrespect to waddi plan curvature, etc. Accepting that the sediment entering the combined canal and sluice approach channel was thereby kept to a feasible minimum, the problem then became one of reducing as far as possible the proportion of the sediment entering the canal.

In Figure 6, the percentage of sediment approaching the headworks which enters the canal (η_s) has been plotted against the sluice discharge, for a canal dis-

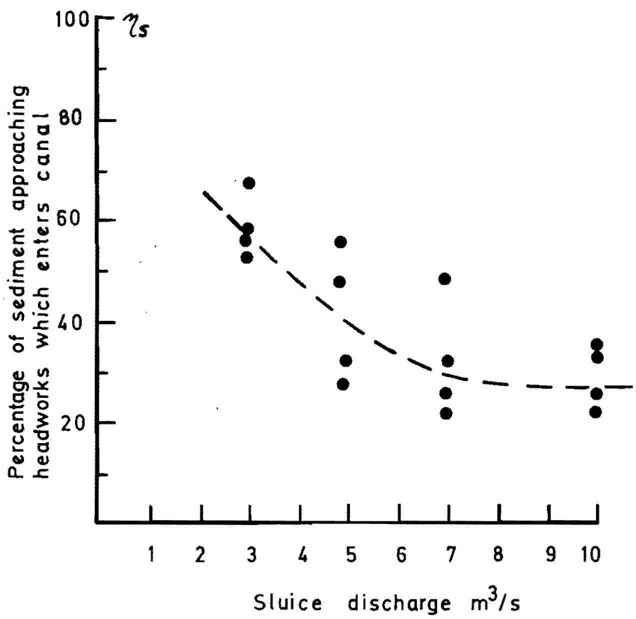

Figure 6. Series 2 Tests: Effectiveness of Sediment Exclusion Related to Sluice Discharge. Canal Flow = 20 $m^3 s^{-1}$

charge of 20 $m^3 s^{-1}$. The approach conditions for the observations plotted included center and weir approaches, but left-bank approaches were excluded as unrealistic. Despite the inevitable scatter, Figure 6 shows a reduction in the percentage η_s as the sluice discharge is increased, but η_s levels off at about 30 percent as the sluice discharge approaches 7 $m^3 s^{-1}$, or about a third of the canal flow. For this particular type of sediment excluder it appears that a sluice flow of about one-third of the canal flow is of the right order, but it is not known to what extent this value might apply to other types of excluder.

Series 3 Tests: Effects of High Tailwater Levels on Sediment Exclusion Efficiency

In the Series 2 tests the discharge in the wadi were restricted and the water level upstream of the diversion weir was at weir crest level. The water level in the sluice channel was therefore relatively shallow and for 7 $m^3 s^{-1}$ sluice discharge the velocity was relatively high. The sluice channel was curved and converging, apparently giving a velocity distribution which was particularly appropriate to the movement of sediment away from the canal intake. However, it was obvious that for a sluice flow of 7 $m^3 s^{-1}$ at depths which were not shallow, the velocity would not be high and the curved channel effect on sediment exclusion would be considerably impaired. These greater depths would occur in the sluice channel at the higher wadi flows, when water would be spilling over the diversion weir and tailwater levels were rising. The extent to which tailwater level would increase the depth of flow in the sluice channel is dependent on the height if the sluice channel floor above the wadi bed, which in turn is dependent on the high of the diversion weir crest level above the wadi. The effect of tailwater levels was investigated by means of tests in which wadi discharges in the range 100 to 300 $m^3 s^{-1}$ prototype were used. In the Series 3 tests, the tailwater levels were varied by means of the adjustable weirs in Figure 4.

The main uncontrollable variable in these tests was

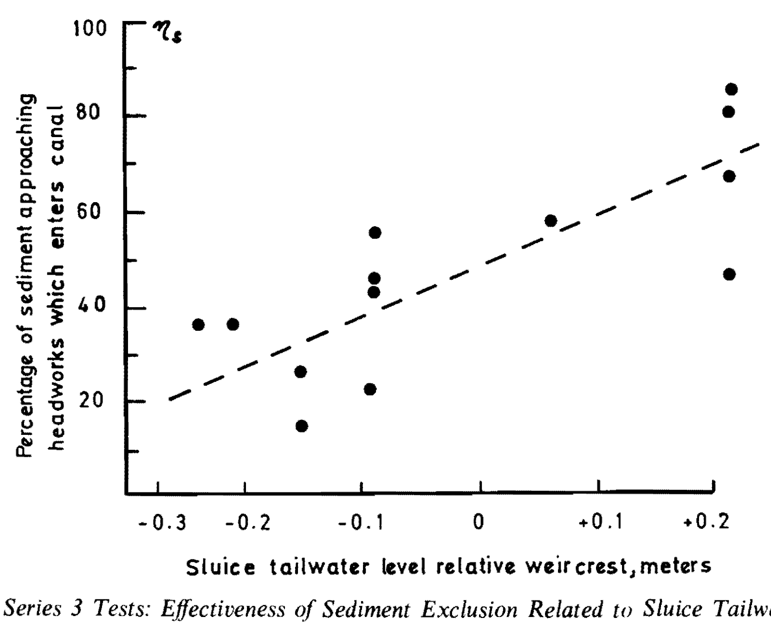

Figure 7. Series 3 Tests: Effectiveness of Sediment Exclusion Related to Sluice Tailwater Level

the approach direction of the deep water channel to the headworks. The actual sluice flow was dictated by the upstream and tailwater levels. Because of this random variation (which was nevertheless realistic) the tests results were somewhat scattered, as is evident in Figure 7. Despite this it can be seen that the percentage of the total sediment entering the headworks which subsequently entered the canal varied from about 25 percent when tailwater levels were about 200 mm below diversion weir crest level to about 70 percent when tailwater level were about 200 mm above diversion weir crest level. This could obviously be attributed to the depth flow in the sluice channel, as affected by tailwater levels. For large depths of flow in the sluice channel, the skimming weir would no longer act as an effective boundary, and the curved channel effect was lost. The ideal solution to this problem would be to keep the tailwater low relative to the structure by constructing the weir crest height well above wadi bed level. This would lead to high costs and it was evident that a compromise had to be reached between costs and sediment exclusion efficiency at high rates of flow. It can be assumed that the quasi-stable behavior of regime canals will be followed in this case, but to a shorter time scale, i.e., during times of high sediment loads entering the canals some deposition will occur, but for the majority of the time when the sediment load is lighter the canal will revert to its normal bed levels.

CONCLUSIONS

1. Model studies have enabled the performance of a sediment excluder to be considerably improved. By the elimination of one sluice channel, the necessary sluice flow was also substantially reduced.
2. The modifications to a preliminary design entailed emphasizing the curvature of the sluice channel and controlling the approach conditions by moving the diversion weir upstream and constructing a guide bank on the wadi bank side. It must of course be emphasized that the action of the excluder is local, and the headworks must be correctly located in relation to the characteristics of the wadi.
3. For the particular type of excluder investigated, efficient sluicing required a sluice flow equal to approximately one-third of the canal flow.
4. In order to preserve the curvature effect of the sluice channel, velocities should not be too low, hence depths of flow should not be too large. So as avoid excessive cost of the diversion weirs, it may not be possible to set the diversion weir crest at a level which is ideal for sediment exclusion at the higher discharges.

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