RIVER EMBANKMENT PROTECTION STUDIES AT KOHILAWATTA - KELANI GANGA

by

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Introduction

Kelani Ganga drains an area of 2280 km² of the wet zone of the island. The 90 km long river originating in the central hills flows into the Indian ocean at a point just North of the city of Colombo. At present the area north of Kelani Ganga is protected by a 7.6 km long flood bund while the 4 km southern bund protects Colombo city areas from flooding Ref: 1. The bankfull discharge of the river is around 870 m³ (3.0 m msl) at Kohilawatta. Its return period is once a year fig: 1(b) Ref:2.

The single carriageway low level road from Totalanga to Kohilawatta which is well within the flood plain has experienced frequent floods in the past. Several reaches of the road is part of the left bank of Kelani Ganga as shown in Fig: 1(a).

During the 1989 floods, a part of the Totalanga Kohilawatte road which is a part of the left bank was damaged severely along a 100m long stretch and consequently the road width has become too narrow.

To protect this road embankment, sheet piles have been driven along the left bank. But this was no remedy and hence the road embankment erosion continued. Subsequently tyre layers tied with nylon ropes were placed along the bank and bamboo strips were placed transversely to reinforce the bank (French technique), in addition to the sheet piles. But during June 1989 floods, part of the tyre layers was washed out and sheet piles collapsed in to the river fig: 6(a).

Bank Protection - General Comments

The river bank consists of the lower and upper sections. The lower bank, the part below low water, acts as the foundation for supporting the upper bank and generally more susceptible to erosion. Recession of the bank is caused by the erosion of the lowerbank, particularly at the toe. The recession is fast, specially when there is a sandy substratum below and under these condition sand is washed away by a strong current and the overhanging ban collapses. During the high stages of the river, erosion of the upper bank could be caused due to a strong current along the bank.

The causes of bank failure must be fully understood in order to formulate protective works. Erosion of river banks may be attributed to one or more factors. Perhaps one condition may lead to another and accelerate the process of erosion. Some of the causes are (a) Washing away of soil particles from the bank by a strong current, (b) Scouring of the toe of the bank by eddies, followed by a collapse of overhanging materials deprived of support (c) Sliding of slope when saturated with water by floods of long duration (d) Reduction of stability due to pressure of seepage flow (e) Piping in sublayer due to movement of ground water towards the river which carries away materials with it.

A given rate of flow has a sediment carrying capacity and hence the flood discharges are bound to pickup materials from the bed and the banks and this rate can even increase on rising flood. Ref: 3. The erosion of left bank of Kelani at Kohilawatte has taken place during a rising flood in 1989 where the flood level reached 4.3 m msl.


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Scope of the Model Investigations

The scope of this work is the evaluation of the causes of bank failure and evolution of an effective solution to prevent further failure of the bank.

For proper reproduction of these phenomena, geometrically similar models are generally required. Distorted scalar models do not represent correct scouring patterns. Ref: 6.

Field Investigations

The following field studies were carried out before embarking on the model investigation.

(a) River cross sections along a length of 1km of river at 30 m intervals and 15m intervals in some stretches.

(b) Surface flow lines of the prototype.

(c) Velocity observations on the left bank where damage has occurred.

(d) Periodic observations of the profile of the left bank fill.

To study the settlement of the road embankment 9 points were located along the road embankment at 10 m intervals as shown in fig: 3(a) and levels are tabulated below.

<table>
<thead>
<tr>
<th>Level on</th>
<th>Level on</th>
<th>Level on</th>
</tr>
</thead>
<tbody>
<tr>
<td>92-2-16</td>
<td>92-4-5</td>
<td>92-5-3</td>
</tr>
<tr>
<td>HRL2</td>
<td>2.851 m msll</td>
<td>2.850 m msll</td>
</tr>
<tr>
<td>HRL4</td>
<td>3.197</td>
<td>3.197</td>
</tr>
<tr>
<td>HRL6</td>
<td>1.913</td>
<td>1.913</td>
</tr>
<tr>
<td>HRL7</td>
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<td>HRL8</td>
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<td>HRL9</td>
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<td>HRL10</td>
<td>3.107</td>
<td>3.107</td>
</tr>
<tr>
<td>HRL12</td>
<td>2.906</td>
<td>2.906</td>
</tr>
<tr>
<td>HRL1</td>
<td>3.107</td>
<td>3.107</td>
</tr>
</tbody>
</table>

From above tabulation it can be seen that there is no observable settlement along the road embankment during the short period of observation.

River Morphology and Sediment Transport

It has been estimated that annually 952,000 tons of bed load and 145,000 tons of suspended load is carried by Kelani Ganga and the rate of sediment discharge is given by Qs = 1.65815 * 10^-6 * Q^4.1239 kg/s Ref: 4.

According to Henderson if the river slope is steeper than s where s = 0.517 * D1.14 * Q^-0.44 then it is likely to be braided whilst those with gentler slopes are more likely to be meandering. In this case where D50 = 1.51 mm, according to Henderson, S varies from 0.0421 and 0.02406 from bankfull discharge (870 m^3/s) to maximum discharge (3100 m^3/s) and the river slope is 0.0004. Hence it is likely to be meandering. Due to meandering effect the bed close to left bank (on the outer curve) is deeper than the other portion as shown in fig 3(b).

Hence it is essential to change the flow characteristics in order to reduce the damaging effect of the current and change the bed profile, where scour is high along the bank. This type of change can be brought about by introducing groynes. Type of groynes, length and spacing should be decided upon carefully based on engineering knowledge, experience and judgement.

The Model

River training works normally designed based on empirical methods are model tested or the designs are evolved using hydraulic models. Considering the scope of studies and the hydraulic parameters under investigation, a model to a scale of 1/35, was constructed fig. 6(b). It represents a river reach of 600 m upstream of the eroded area and 300 m downstream.

In mobile bed modeling it is nearly impossible to scale sediment particle diameter and obtain realistic transport of sediment and hence other techniques of using low density (hence larger diameter) materials are adopted. Taking initiation of bed movement, the relationship has been derived. Ref: 7

\[ d = D \times e \left( \frac{W-1}{W-1} \right)^{3/2} \]

Where
- \( d \) = mean diameter of model bed material
- \( D \) = mean diameter of real bed material
- \( e \) = Linear scale of model
- \( W \) = specific gravity of model bed material
- \( W \) = specific gravity of real bed material

However the model scales have been derived without resorting to movable bed modeling.

Tests carried out in this model operated on the Frounud law yield only qualitative results and as such observation made enable comparison of behavior of different variants.

Bed materials used in the model were sea sand of D50 = 0.40 mm and specific gravity 2.65 and some tests were run using saw dust treated with lime Ref: 7. A spherical fruit
(jamfruit) of diameter 7 mm to 10 mm was used to observe the bed flow lines while floats were used for surface flow lines. River cross sections were obtained to setup the model. Four surface flow lines observed in the river and the velocities observed near the left bank were used to prove the model. Periodic observations of the bank profile were taken to observe any further settlement of the bank. The model Reynolds number was greater than $5 \times 10^5$ when the discharge was greater than 300 m$^3$/s and hence the model was operated in the rough turbulent zone simulating the correct roughness.

**Hydraulic Model Tests**

In the process of the river adjusting itself to transmit as much water and sediment as it receives from upstream, there will be changes of the width, plan form and the slopes. There is historical evidence of these changes taking place in the lower reaches of this river. In this stretch under investigation, the river section has moved to the left while producing a typical scour hole on the left bank having a steep slope. Hence this bank has collapsed during floods.

The preliminary tests conducted in the model included surface Fig: 2(a) and the bed flow lines Fig 2(c), velocity observations near the left bank Fig 4(a) and velocity profiles across the river Fig 4(b) and the profile across the river Fig 3(c) for the bankful discharge of 870 m$^3$/s to observe the general flow characteristics and their effects on the river profile.

The concentration of surface flow lines towards the left bank and the bed flow lines towards the right bank is typical for a bend in the river.

Two types of groynes-attracting type and repelling type—both having same length, angle and the spacing were tested in the model. It was observed that attracting type groynes having angle of inclination 45 degrees gave lower velocities along the bank in comparison with the repelling type. The flow conditions evaluated with respect to the surface flow lines and the velocities behind the groynes are proof of unfavorable flow conditions with repelling type groynes. Hence a series of attracting type groynes were selected for detail investigations.

**Attracting Type Groynes**

8 m long attracting type groynes having an angle of inclination of 45 degrees were tested with different spacing and it was observed that velocity distribution along the bank is low for spacing of 8 m, 16 m and 24 m as shown in fig 4(a), and surface flowlines indicate that the bank is safe from the action of direct current.

**Velocity Distribution Across the river**

Velocity distribution across the river were observed in three cross sections with and without groynes as shown in the fig 4(b). From these observation it is seen that there is considerable reduction in the velocity on the left.

**Erosion**

The river bed profiles were observed by taking 3 cross sections and the longitudinal section and it was observed that erosion is taking place on the river bed towards the right side while the left side deeper section got silted up and the right side started to scour as shown in fig 3(c).

**Stability of Groynes**

Considering the maximum velocity of flow to be 4 m/s and weight of stone to be 400 kg, the calculated mean diameter is 0.666 m (Ref. 8)

**Side slopes**

Maximum height of water in the river is 8 m and hence the critical shear stress on the sides $T_c = 0.75 \times (\rho_w \cdot g \cdot y) = 23.54$ N/m$^2$ due to flow of water. The critical shear stress according to shields diagram is $T_c = 0.06 \times (\rho_s - \rho_w) \cdot g \cdot d = 666.41$ N/m$^2$ to move the 0.666 m diameter rubble packing on level surface. Therefore for a side slope of 1 on 1 (45 degree) and angle of repose of 0.666 m diameter rubble of 50 degree, the shear stress required to move the rubble on the slope is $666.41 \times k$ N/m$^2$.

where $k = (1 - \sin^2 45 / \sin^2 50)^{0.5} = 0.3955$

Therefore shear stress needed to move the stones on side slopes = $666.41 \times 0.3955 = 263.57$ N/m$^2$. This is more than 23.54 N/m$^2$ caused by flowing water. Hence taking side slopes 1 on 1 and the nose of the slope as 1 on 1.5 the rubble packing is safe against movement of stones (Ref: 9 & Ref: 10).

**Conclusions**

1. Damage to the left bank has been definitely caused due to high velocity current and particularly because of the bend in the river.
2. With the construction of the proposed groynes there will be a change of river bed configuration facilitating the stability of the left bank. Stopping of sand mining within a distance of 0.75 km upstream and 0.5 km
downstream will be essential to prevent further erosion even after the construction of the groynes. Periodic observation of this area should be made to monitor the behavior of the groynes.

3. The proposed preventive measures may some times lead to adverse conditions downstream and hence the behavior of the river reach should be monitored after implementation of the proposals.

References

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5. Coast Conservation Department management programme - 1984

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10. Sediment transport by H N C Breusers-Delft-1980
LOCATION PLAN

FIG. 1 (a)

SIMULATED WATER LEVELS AT KOHILLAWATHE (21-05-1991)

FIG. 1 (b)
SURFACE FLOW LINES
WITHOUT GROYNES
FIG. 2 (a)

SURFACE FLOW LINES
WITH GROYNES
FIG. 2 (b)

BANK FULL DISCHARGE (870 Cumecs)
BED FLOW LINES WITH OUT GROYNES

FIG. 2 (C)

BED FLOW LINES WITH GROYNES

FIG. 2 (D)

(BANK FULL DISCHARGE)
VELOCITY DISTRIBUTION ALONG THE LEFT BANK

VELOCITY DISTRIBUTION ACROSS THE SECTION L12_R13

VELOCITY DISTRIBUTION ACROSS THE SECTION L14_R14

FIG. 4(a)

FIG. 4(b)
ROAD EMBANKMENT PROTECTION
AT KOHILAWATTA - KELANI GANGA

LAYOUT
FIG. 5
SCOUR DAMAGES TO THE LEFT BANK OF KELANI GANGA AT KOHILAWATTA AND SUBSEQUENT FAILURE OF SHEET PILE PROTECTION

FIG. 6 (a)

1/35 GEOMETRIC SCALE MODEL OF LEFT BANK PROTECTION WORKS KELANI GANGA - KOHILAWATTA

FIG. 6 (b)