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Quantifying hydrologic impacts following dam construction along the Tana River, Kenya

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Daily pre-dam and post-dam discharge data for Kenya's largest river, the Tana, were analysed using flood frequency analysis and computation of various indicators of hydrologic alteration (IHA). Results from these analyses indicated statistically significant ($p < 0{\cdot}01$) augmentation of minimum river flows and reduction in peak flows. We also estimated the frequency of flooding of 71 vegetation sample plots located on various parts of the river floodplain by running a hydrologic water profile simulation program (HEC-RAS). Results from these analyses indicated that plots at elevations greater than 1·80 m above the dry season river level experienced statistically significant ($p < 0{\cdot}01$) reduction in days flooded from the pre- to the post-dam period. This documented magnitude of change in the hydrologic regime will have a negative impact on the unique riverine forest occurring along the Tana River.

Keywords: flood frequency analysis; indicators of hydrologic alteration (IHA); HEC-RAS; Tana River; riverine; forests

Introduction

Background

The natural flow of a majority of the world's rivers has been substantially altered through dam construction (Petts, 1984; Naiman *et al.*, 1993). Great efforts have been expended to tame rivers for transportation, water supply, flood control, agriculture, and power generation. However, the lakes that form behind these dams often destroy upstream riparian habitats (e.g. Ohmart *et al.*, 1988), while in downstream areas, changes in flow regimes have been shown to lead to extensive ecological degradation and loss of biologic diversity (Baxter, 1977; Petts, 1984; Kingsford, 2000; Jansson *et al.*, 2000). The elimination of the benefits of seasonal flooding downstream of dams may be the single most ecologically damaging impact of dam construction. In Africa,

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construction of dams has led to the disruption of many traditional production systems including flood recession agriculture (through changing the availability of water on the surface and in shallow aquifers, and by changes in the distribution of fertile sediments delivered with the flood waters), livestock management, and fisheries production (Adams, 1985, 1992; Scudder, 1989; Thomas & Adams, 1997).

Dam construction and water development projects create wide-ranging social and environmental consequences with impacts extending well beyond the initial planning area. Ecologists and environmentalists are often challenged by the complex interaction of forces at work in these environments, making prediction of overall effects difficult. Unless study of these effects begins 5–10 years before construction, unanticipated consequences are inevitable, some beneficial and others adverse (Biswas, 1978).

There are two main categories of environmental impacts of dams: those which are inherent to dam construction, and those which are due to the specific mode of operation of each dam (McCully, 1996). Impacts due to the existence of dam and reservoir include: (1) upstream change from river valley to reservoir, (2) changes in the morphology of riverbed and banks, delta, estuary and coastline due to altered sediment load (e.g. Park, 1981); (3) changes in downstream water quality which has effects on river temperature, nutrient load, turbidity, dissolved gases, concentration of heavy metals and minerals; and (4) reduction of biodiversity due to the blocking of the movement of the organisms and because of changes in (1), (2) and (3) above. Impacts due to the pattern of dam operation include (1) changes in downstream hydrology which may include changes in total flows and/or in seasonal timing of flows, short-term fluctuations in flows, and changes in extreme high and low flows, (2) changes in downstream morphology; (3) changes in downstream water quality, and (4) reduction in riverine habitat diversity, especially because of the elimination of floods (Johnson *et al.*, 1976; Cadwallader, 1986; Bren *et al.*, 1988; Bren, 1992; Gordon *et al.*, 1992; Jolly, 1996; Richter *et al.*, 1997; Scott *et al.*, 1997; Toner & Keddy, 1997). The elimination of the benefits provided by flooding may be the single most ecologically damaging impact of dam construction. In Africa, elimination of downstream flooding has adversely affected many local production systems including flood recession agriculture, livestock management, and fisheries production (Adams, 1985; Scudder, 1989).

Responsible water management requires sensitivity to a wide range of issues and a starting point is understanding the impacts of present and future water development systems. This study was therefore designed to evaluate the effects of major dam construction on the hydrologic regime of Kenya's largest river, the Tana River, over a 100-km stretch downstream of the town of Garissa (Fig. 1). This reach of the river was chosen because of the availability of the longest discharge record for the Tana, and availability of surveyed river cross-sectional profiles. River discharge data from this station was used to compute various hydrologic parameters that are indicators of hydrologic alteration. The parameters are: magnitude, frequency, duration, timing, and rate of change of river flows (Richter *et al.*, 1997). These analyses coupled with surveys of approximately 70 vegetation sample plots representative of the forests along the river permitted a detailed evaluation of the impact of dam construction on an environmentally important reach of the Tana River.

The Tana River and water development projects in Kenya

A major constraint to development in Kenya is the scarcity of water relative to the expanding population. Over 75% of the country is classified as semi-arid. Most of the population is concentrated in the wetter areas, but with the current annual growth rate of 4%, severe land pressure is forcing expansion into semi-arid areas (Rowntree, 1990). In order to meet the water needs for her growing population, Kenya has embarked on the

Figure 1. Location of the Tana River and its major tributaries in Kenya.

development of multipurpose reservoirs across the main rivers. The Tana River basin has about 20% of the national population, a major portion of the agricultural potential and the highest hydroelectric power generation potential in the country.

The Tana is the largest river in Kenya, rising from the south-western flanks of the central massif of Mount Kenya. It flows for 1012 km, first east to Garissa and then south until it meets the Indian Ocean near Kipini (Saha, 1982). The Tana River drainage basin covers an area of 120,000 km^2 (Ongwenyi *et al.*, 1993*a*). In its upper catchment, the Tana is joined by a dense network of fast flowing perennial rivers that drain the eastern and southeastern flanks of Mount Kenya, and from the Nyandarua (Aberdare) Range and Nyambene Hills (Fig. 1).

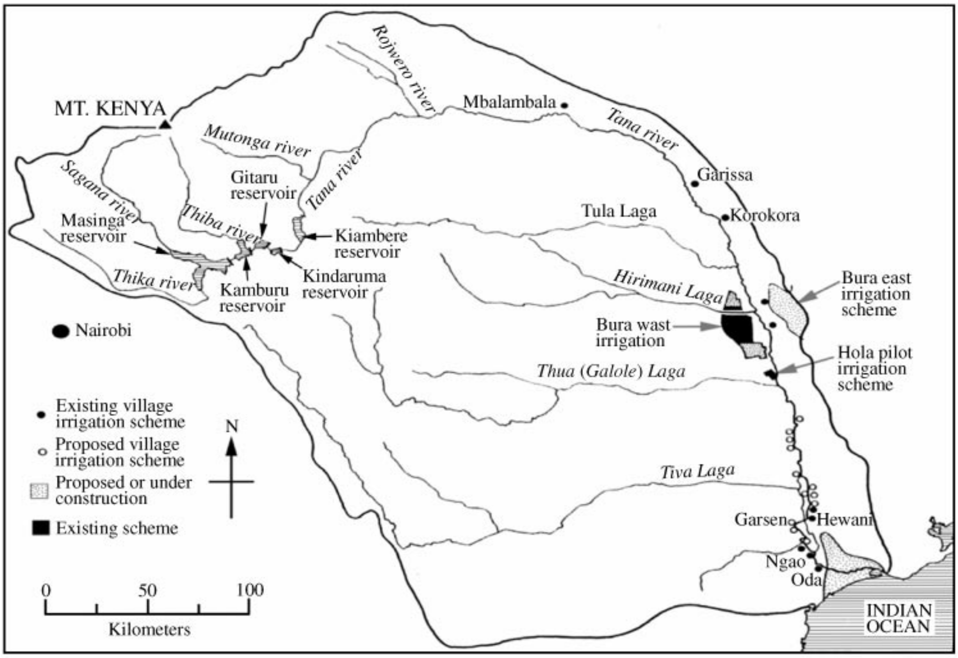

Figure 2. The Tana River Basin and location of present and future planned water development projects.

The lower Tana River basin, stretches from Mbalambala to the delta at Kipini (Fig. 1). The lower basin includes a stretch of the river approximately 625 km long, while the delta occupies an area of about 3000 km^2 (Beck *et al.*, 1986). From Mbalambala onwards, the Tana flows through a vast semi-arid expanse that is drained by a few right-bank seasonal rivers such as Tula, Hirimani, Galole and Tiva (Fig. 2). These rivers (*lagas*), traverse distances of up to 800 km but are dry most of the year. Thus, the river gains no new water in this part of the basin, but loses water continuously through channel losses and evaporation. Rainfall increases from about 350 mm per annum at Garissa to about 470 mm per annum at Hola, and over 1000 mm per annum at the delta. Rainfall in the study area is bimodal with the long rains occurring in April-May and the short rains occurring in November–December. Rainfall is highly variable and averages about 370 mm per annum.

Five dams have been constructed along the Tana River in the upper basin (Fig. 2). The first three dams constructed between 1968 and 1974 were Kindaruma, Kamburu and Gitaru. These first three dams produced rather small reservoirs that left the Tana River essentially unregulated (Hughes, 1985). Construction of the largest dam so far, the Masinga Dam, commenced in 1977 and was completed in 1981. The reservoir had a storage capacity of 1560 million m^3 and an installed generation capacity of 40 MW. Masinga Dam was sited upstream of all potential hydroelectric power generating sites as a regulating reservoir. The objectives of this dam were to: improve electric-power generation during the dry season; increase irrigation potential in the lower Tana basin; and allow increased utilization of dry season flows in the upper Tana. Construction of the fifth dam on the Tana, the Kiambere Dam, started in 1984 and was completed in 1988. The Kiambere has a reservoir capacity of 535 million m^3 and an installed generation capacity of 140 MW (Ministry of Energy, 1987).

Importance of the Tana River to biological conservation

There is a riverine forest along the banks of the Tana River from Mbalambala to the delta at Kipini (Fig. 1). This forest extending 0·5–3·0 km on either side of the river, is an isolated remnant of a once continuous rainforest belt that extended between the Congo Basin and the eastern coast of Africa during moister periods of the Pleistocene (approximately 31,000–26,000 and 8000 B.P.) (Livingstone, 1975 and adapted from Medley, 1990). Severe climatic drying after the hypsithermal (approximately 4000 B.P.) isolated east African evergreen forests in the highlands and riverine localities (Hamilton, 1974; Livingstone, 1975). The depth of the water table, which drops off rapidly from the edge of the river (Marsh, 1978; Hughes, 1985), apparently determines the extent of this forest. A drought-deciduous bushland dominated by thorny shrubs with scattered annual grasses covers extensive areas away from the floodplain. The riverine forest is unique because of its great biologic diversity and occurrence in an otherwise arid environment.

The Tana riverine forest also has a high conservation value because it is home to two endemic subspecies of primate: the Tana River Red Colobus (*Colubus badius rufomitratus*) and the Tana River Mangabey (*Cercocebus galeritus galeritus*). These are both classified as 'rare' and 'critically endangered', respectively by the IUCN. The Tana River poplar (*Populus ilicifolia*) is endemic, occurring in small patches along the Tana, Athi and Ewaso-Nyiro river systems (Dale & Greenway, 1961). It is classified by the IUCN as 'threatened' (IUCN, 1978). The Tana riverine forest is dependent on floods and associated groundwater. The lateral extent of the forest is determined by a decline in water-table depth from the river. Consequently, the composition, structure and dynamics of the forests are influenced by the hydrological characteristics of the river system (Marsh, 1978; Hughes, 1988). Most studies carried out on the Tana riverine forests have indicated a lack of forest regeneration. This lack of regeneration has been attributed to various factors, among them; decreased peak flows (Marsh, 1976; Hughes, 1985; Medley, 1990).

Study goals

Regulation of the Tana River through the construction of Masinga Dam in 1981 and Kiambere Dam in 1988 may have significantly altered the hydrologic regime of the river. Our goals were therefore to: (1) assess the magnitude, timing, rate of change, and frequency of flows for the 100-km reach of the Tana River between Garissa and Hola (Fig. 1), and (2) estimate the frequency of flooding, and consequently, duration of flooding on 71 vegetation plots sampled along the river floodplain in the study area. Outputs from the second goal are a critical input in any ecological study seeking to examine how different forest types along the Tana River floodplain have been affected by dam construction. The methods and results for each of these goals are covered individually.

We approached our first goal by analysing daily discharge data for the Tana River recorded at the Garissa gauging station. These data were used to describe the characteristic pattern of the river's flow quantity, timing, and variability, that is, its natural flow regime. Identifying the flow regime components that have been significantly altered by human actions permits decision makers and river managers to focus on the specific aspects of the flow regime that need to be restored or protected.

Our second goal was accomplished by running a hydrologic water profile simulation program on data generated using surveyed vegetation plot heights, recorded channel cross-section data, and recorded stream flow data from the Garissa gauging station.

Analysis of hydrologic regime

Flood frequency analysis methods

The primary objective of a flood frequency analysis is to estimate a flood magnitude corresponding to any required return period of occurrence (Cunane, 1989). Flood prediction is an assignment of occurrence probabilities to magnitude of floods, which might occur. A reasonable assignment of probabilities requires fitting a probability distribution to observed magnitudes. Owing to the limited duration of quantitative hydrological observations and the infrequent occurrences of flood events, there are in most cases too few floods on record to which a good fit can be made. This difficulty is overcome by fitting the distribution to a series composed of a sufficient number of recorded events.

Flood events can be analysed using either an annual series or a partial-duration series. An annual flood series will consist of only the maximum annual flood peak for each year, irrespective of its magnitude. The partial-duration series consists of all independent flood peaks equal to or greater than a predefined magnitude or threshold (Kite, 1977; Hoggan, 1997).

There are two basic methods of flood frequency analysis: graphic and statistical analysis (Hoggan, 1997). The graphic technique is quick and easy to use but is also imprecise and subjective. The statistical analysis (analytic) method includes the following steps: (1) selection of the theoretical probability distribution for the population to be analysed, (2) estimation of the parameters of the selected distribution from the observed data, (3) computation of points to be used in plotting the curve, and, (4) a comparison between the analytically derived curve and observed data plotted with the graphical method.

Several theoretical distributions are commonly used for fitting the observed sample distributions of annual maximum floods. These include: (1) the Log-Normal distribution, (2) the Gumbel Type I extreme-value distribution, (3) the Gumbel Type III extreme-value distribution (a logarithmic transformation of Gumbel Type I), and (4) the Log-Pearson Type III distribution. The reason for replacing an empirical probability distribution by a theoretical distribution is to allow extrapolation of values of extreme probabilities. Using a theoretical probability distribution avoids subjective extrapolation of empirical probability distribution outside the range of observations (UNESCO, 1987).

In general, a probability distribution will fit the data best and provide the most accurate estimates near the middle values; less so near the tails (Gordon *et al.*, 1992). Although no one distribution will fit all flood data, specifying the distribution and method of fit will allow other researchers to obtain the same results from the same set of data. The procedure is thus much more objective than graphical methods using eye-fitted curves (Gordon *et al.*, 1992). The analytic method is more desirable than the graphic method as more uniform and consistent estimates of population parameters are possible, statistical measures of reliability can be computed, and because there is a theoretical basis for analysing rare events (Hoggan, 1997). The disadvantages of the analytic method include the necessity of selecting a theoretical probability distribution that may not fit the data exactly and the greater potential for a false sense of accuracy, particularly in the estimation of rare events (Hoggan, 1997).

A large variety of distributions have been investigated for application to flood data (Kirby & Moss, 1987; UNESCO, 1987; Cunane, 1989; Bobee *et al.*, 1993; Harktanir & Horlacher, 1993; Ashkar, 1996). Some distributions will be more appropriate for some rivers than for others. The Log-Pearson Type III distribution has been selected as the standard for flood frequency analysis by federal agencies in the United States (Benson, 1968) and Australia (IEA, 1977), while the generalized extreme value (GEV) distribution is the standard in the U.K. (NERC, 1975). Pearson Type III distribution has been recommended for determination of design floods in China (Shi-Qian, 1987).

The two-parameter Log-Normal Distribution has been found best in Italy (Cicioni *et al.*, 1973) and Canada (Spence, 1973). McMahon *et al.* (1992) have shown that the Log-Pearson III distribution fits the flood data of rivers around the world better than any other.

IHA analysis methods

Flood frequency analysis uses the annual maximum series and therefore may include insignificantly small floods and exclude some large floods. Using an annual maximum series means that floods with an average recurrence interval of less than one year cannot be estimated unless a partial duration series is used. Although the two problems outlined above may be overcome by using the partial duration series, it is much more difficult to ensure statistical independence of the data (Finlayson & McMahon, 1995). For this reason, we found it necessary to compute other measures of hydrologic alteration that better described river flows with recurrence intervals of less than a year.

Ecological processes in river ecosystems are regulated by five critical components of the flow regime, namely: the magnitude, frequency, duration, timing, and rate of change of hydrological conditions (Poff & Ward, 1989; Ritcher *et al.*, 1996, Walker *et al.*, 1995; Poff & Allan, 1997). A number of studies examining the issue of hydrologic alteration in rivers and streams have used the Indicators of Hydrologic Alteration (IHA) method proposed by Richter *et al.* (1996). In this method, a series of biologically relevant hydrologic attributes that characterize intra-annual variation in water conditions are computed and their inter-annual variation used as the foundation for comparing hydrologic regimes before vs. after a system has been altered by various human activities. These hydrologic attributes are calculated here, in addition to performing flood frequency analyses on an annual series of maximum floods for the pre- and post-dam period.

Magnitude of discharge at any time interval is the amount of water moving past a fixed location per unit time. The mean river discharge for each month (ms^{-1}) is used to measure changes in flood magnitude from the pre- to the post-dam period. The monthly mean of the daily water conditions describes 'normal' daily conditions for the month, and thus provides a general measure of habitat availability or suitability. The IHA method includes 12 parameters each measuring the mean or median of the daily river discharge conditions for a given month.

Another measure of hydrologic alteration is magnitude and duration of annual extreme conditions. Five minimum and five maximum annual river discharges of various duration ranging from daily to seasonal are computed for this group of measures. The mean magnitude of minimum and maximum flows of various durations provide measures of environmental stress and disturbance during the year, which could impact reproduction of certain species (Richter *et al.*, 1996).

The timing of flows of defined magnitude refers to the regularity with which they occur. The IHA method uses two measures as indicators of changes in the timing of river flows: the Julian date of the 1-day annual minimum water condition, and the Julian date of the 1-day maximum water condition. River discharge data for the Tana River is distinctly bimodal and therefore the average Julian date for annual minimum and maximum discharge would be misleading without splitting the discharge record into two. The Julian dates associated with flood maximum and minimum discharges were therefore averaged separately for January–June and for July–December. The first half of the year contains a minimum and maximum flood centered on February and May, respectively. The second half of the year contains a minimum and maximum discharge centered in September, and November, respectively. The timing of the highest and lowest water conditions within annual cycles provides another measure of environmental disturbance especially with regard to key life cycles such as reproduction (Richter *et al.*, 1997).

The next group of hydrologic measures computed describes the frequency and duration of high and low pulses. Hydrologic pulses are defined as those periods within the year that river discharges fall below or rise above a defined lower and upper threshold. The lower threshold is normally defined as 1 S.D. below mean river discharge for the pre-impact period. Likewise, the upper threshold is defined as 1 S.D. above the pre-impact river flow mean. However, due to the frequently skewed distribution of river discharges, calculation of the lower discharge threshold could result in a negative value. As this was the case with our data, upper and lower discharge thresholds were accordingly defined as flows corresponding to the 75th percentile, and the 25th percentile, of pre-dam river flows, respectively. The number of annual occurrences during which the upper threshold was exceeded is the number of high pulses whereas the number of occurrences the water condition remained below the lower threshold is a measure of low pulses.

Four parameters measuring the rate and frequency of change in river flow conditions were computed. These measures included the number and mean rate of both positive and negative changes in water conditions from one day to the next. These measures provide a measure of the rate and frequency of intra-annual environmental change (Richter *et al.*, 1997).

Rainfall distributions and data sources

Rainfall distribution

It was necessary to determine whether rainfall patterns in the catchment area of the Tana River remained unchanged throughout the period of flood-frequency analysis (1941–1996). This was necessary so that changes in catchment area precipitation could be ruled out as the contributing factor in any observed hydrologic changes.

Rainfall data were available for only three stations in the catchment area. These stations were Meru, Embu, and Murang'a. Meru data is for the period 1914–1985, Embu for 1908–1990, and Murang'a for 1901–1985. Nairobi rainfall station (Jomo Kenyatta International Airport), though not within the catchment of the Tana is close, and has rainfall distribution similar to that of areas within the Tana River catchment. The record from Nairobi is for the period 1951–1990. Each rainfall record was divided into two periods corresponding to the pre- and post-Masinga dam period.

Both parametric and non-parametric tests were carried out to determine whether there was a statistically significant difference in means, medians, and distribution of rainfall between the two periods. Statistical tests carried out included *t*-tests (to compare the means), *F*-tests (to compare variances); Mann–Whitney (Wilcoxon) W tests (to compare the medians, and Kolmogorov–Smirnov test (to compare the distributions of the two samples).

Results for both parametric and non-parametric tests revealed that there were no significant differences in the means, medians, and distributions of rainfall for Meru, Murang'a and Nairobi rainfall stations. Non-parametric tests for the Embu station also indicated no significant differences in rainfall between the pre- and post-dam rainfall. However, the parametric test indicated a significant increase ($p < 0{\cdot}05$) in precipitation from the pre- to the post-dam period. However, an examination of the rainfall data indicated significant departures from normality, which would tend to invalidate this test. Nevertheless, an increase in precipitation in the catchment area would tend to mitigate reduction in river flows resulting from dam construction. Results from these tests imply that any reductions observed in river discharge between the pre- and post-dam period can now be attributed to dam construction.

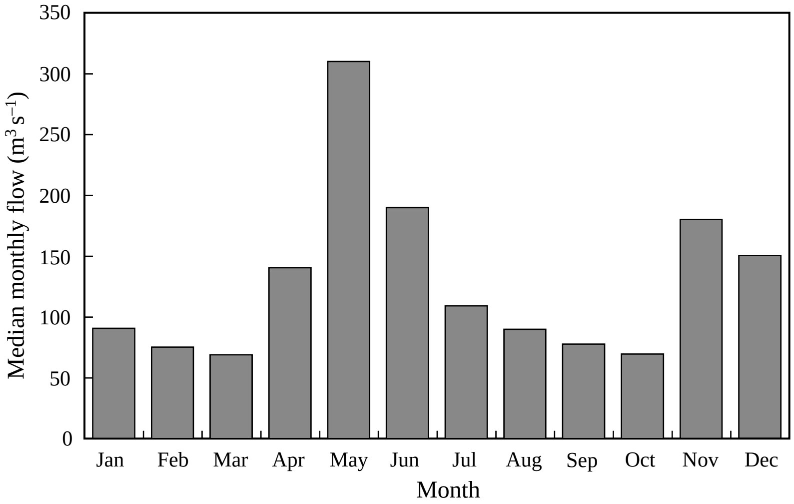

Figure 3. Distribution of median monthly discharges for the Tana River (1941–1996). There are peaks in May and November corresponding to the long and short rainy seasons, respectively, which occur in the upper river catchment basin.

River discharge

River discharge data used in this study were from the Garissa gauging station, located about 60 km upstream of the northernmost part of our study area. The data were obtained from the Ministry of Land Reclamation, Regional and Water Development Headquarters in Nairobi. The Garissa record begins in 1933 and ends in 1996. Daily data values for stage height are available from 1933 to 1993, whereas daily discharge data begins from 1941 and ends in 1996.

The Tana River experiences biannual floods, with peaks in May and in November (Fig. 3). May flows are generally higher and less variable than November flows (Table 1). The highest discharge ever recorded at Garissa is 3568·3 m^3s^{-1}, and occurred on 21 November 1961. The biannual floods are in response to the short and long rainy seasons occurring in the catchment area of the upper river basin. The long rainy season occurs in the months of April–May, and the short rains in October–November. The low-flow period corresponds to the end of the dry season in the upper river catchment, and the lower Tana River floodplain. The lowest river flows occur in February–March and September–October.

A plot of daily discharges (Fig. 4) highlights the high river flows observed throughout the 1960s attributed to higher than average rainfall observed throughout East Africa (Lamb, 1966; Dunne & Leopold, 1978). There is a discontinuity in the discharge record in September and October 1979, in 1980 for the months of January–April, August–September and November–December, and in 1981 from January to March.

Two annual-maximum discharge data series were extracted from the daily discharge data recorded at the Garissa gauging station. The first series is for the period 1941–1979, and corresponds to the pre-Masinga Dam period. Although the Masinga Dam was not completed until November 1981, the data gap in 1980 made it necessary to omit this year from the analysis. The second series is for the period 1982–1996 and corresponds to the post-Masinga Dam period.

Each annual-maximum series was input into the computer program HydroTech (1997) and several probability distributions fitted. The four distributions fitted were: (1) Log-Pearson Type III, (2) 2-parameter log-normal, (3) Pearson Type III, and (4)

Table 1. *Results of fitting annual maximum series data to the Log-Pearson III probability distribution*

Probability of exceedance	Recurrence interval (years)	Pre-Masinga Dam period			Post-Masinga Dam period		
		Estimated discharge (m^3s^{-1})	− 95% confidence limit (m^3s^{-1})	+ 95% confidence limit (m^3s^{-1})	Estimated discharge (m^3s^{-1})	− 95% confidence limit (m^3s^{-1})	+ 95% confidence limit (m^3s^{-1})
0·50	2	719	614	839	782	628	983
0·20	5	1197	1017	1460	1143	916	1566
0·10	10	1597	1324	2037	1361	1070	1969
0·05	20	2050	1654	2737	1551	1197	2350
0·02	50	2748	2138	3887	1773	1340	2825
0·01	100	3365	2549	4965	1925	1434	3165

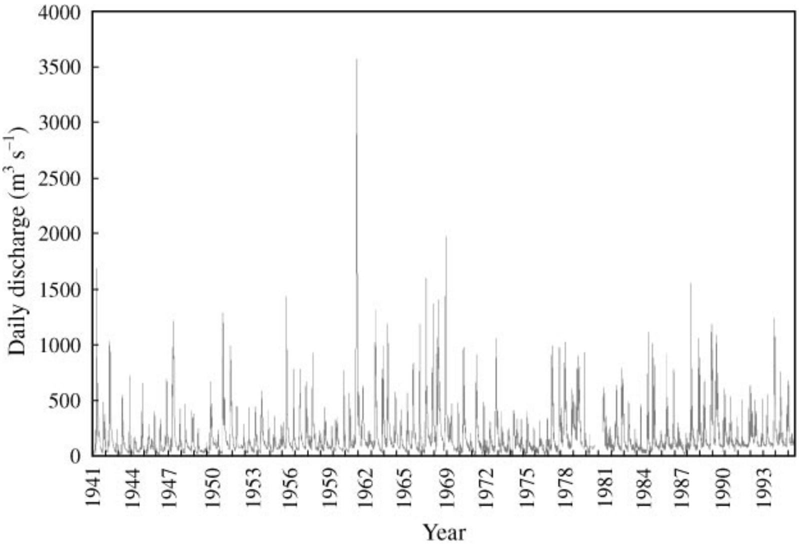

Figure 4. Daily discharge of the Tana River at the Garissa gauging station between 1941 and 1996.

Gumbel (Extreme Value Type I). Subsequent outputs from this program were input into Statistica (1997) to generate charts. The fitted distributions were then evaluated for best fit to each annual maximum series.

A fifth distribution, the 3-parameter log-normal distribution, was also fitted to the pre- and post-dam annual-maximum series. This distribution is not available in HydroTech but was available in Distrib 2·13 (part of the SMADA suite of programs that accompany the hydrology text by Wanielista *et al.*, 1996).

Flood frequency analysis results and discussion

The Log Pearson III distribution was found to best fit the pre- and post-Masinga Dam annual-maximum series data sets (Figs 5 & 6). Observed values for each annual maximum series were plotted using the Weibull formula (the most commonly used plotting formula in the United States). The formula is:

$$P = \frac{m}{n+1} \text{ or } T = \frac{n+1}{m} \tag{1}$$

where;
n = number of years of record
m = rank of the event in order of magnitude, the largest event having $m = 1$.
P = probability of flood being equaled or exceeded in any year
T = is the return period.

Return interval and recurrence interval are used interchangeably to mean the average interval in years within which a given event will be equaled or exceeded (Wilson, 1990). The return intervals for various floods as predicted using the fitted log-Pearson III distribution for both the pre- and post-Masinga Dam period are presented (Table 1).

Predicted 2-year discharge increased from 719 m^3s^{-1} to 782 m^3s^{-1} from the pre- to the post-Masinga Dam period. In order to find out whether this increase in discharge was significant, all floods within the 95% confidence limits of predicted 2-year floods for

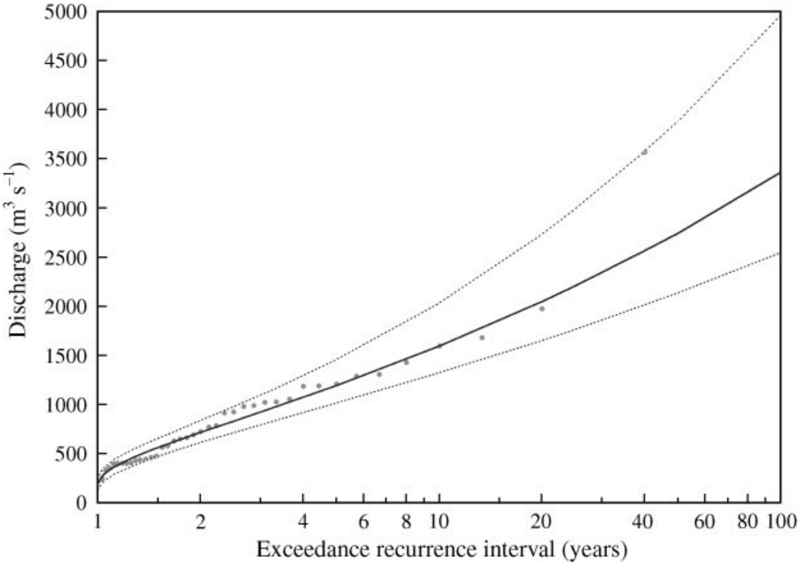

Figure 5. The Log-Pearson Type III distribution fitted to an annual maximum series for the Tana River for the pre-dam period (1941–1979). ● Observed data (Weibull values); —— Log-Pearson Type III estimates; .... 95% Confidence limits.

the pre- and post-dam period were extracted from the daily discharge record and compared. Since distributions of extracted discharges deviated significantly from normal, the Mann–Whitney and the Kolmogorov–Smirnov tests were used to compare medians, and distributions, respectively, of the pre- and post-dam discharges. Tests

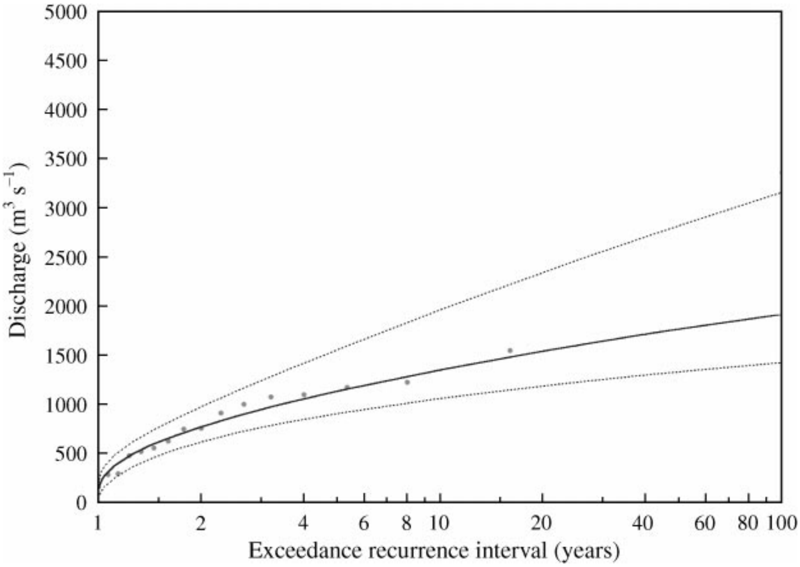

Figure 6. The Log-Pearson Type III distribution fitted to an annual maximum series for the Tana River for the post-dam period (1982–1996). ● Observed data (Weibull values); —— Log-Pearson Type III estimates; .... 95% Confidence limits.

revealed that the distributions of the two discharges were significantly different ($p < 0{\cdot}01$) and that the observed increase in predicted 2-year discharges from the pre- to the post-dam period was statistically significant ($p < 0{\cdot}01$).

Magnitude of predicted 5-year floods decreased from 1197 m^3s^{-1} to 1143 m^3s^{-1} from the pre- to the post-dam period, respectively. The medians and distributions of 5-year pre- and post-dam discharges were extracted as described above and compared. Pre-dam median discharges were significantly greater ($p < 0{\cdot}01$) than post-dam discharges, and the distributions of discharges from the two periods were also significantly different ($p < 0{\cdot}01$). Similar tests performed on extracted 10-year and 20-year floods also indicated significant differences ($p < 0{\cdot}01$) in both magnitude and distribution of floods from the pre- to the post-dam period.

The major impact of construction of the Masinga Dam along the Tana River has been to augment minimum river flows while reducing peak floods. This observation is consistent with that of other studies, e.g. Petts (1984), Toner & Keddy (1997).

IHA analysis results and discussion

A major downstream impact of construction of the Masinga Dam on the Tana River has been a significant ($p < 0{\cdot}01$) reduction in May flows (Fig. 7). May, the month of highest median discharge during the year, has experienced flow reductions of up to 20% while April, June and July have seen virtually no change in median flows since dam construction. The remaining months have seen increases in median flows from the pre- to the post-dam period. The low flow months of February and March have seen increases of 73·4% and 68·3%, respectively (Fig. 7).

Variability of monthly discharges has also declined since dam construction (Table 2). Reductions in variability of up to 50% have been observed in the low flow months of February-March, and October, and in the high flow month of November. Mean monthly river discharges have increased significantly ($p < 0{\cdot}05$) for January, February, March, October and December from the pre- to the post-dam period.

Minimum 1-day, 3-day, 7-day, 30-day, and 90 day river discharges increased significantly ($p < 0{\cdot}01$) from the pre- to the post-dam period (Table 3). Maximum 1-day and

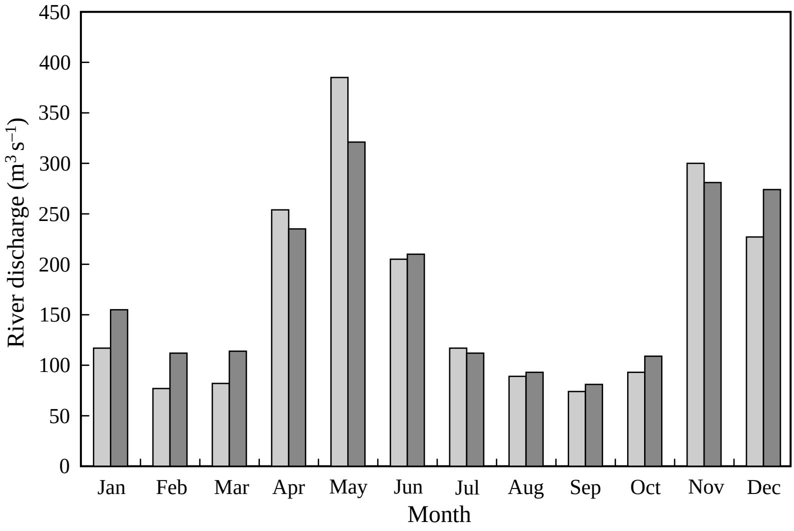

Figure 7. Mean monthly flows for the Tana River for pre- (□) and post-dam (■) periods.

Table 2. *Monthly mean flows for the Tana River*

Month	Pre-dam period (1941–1979)					Post-dam period (1982–1996)				
	Mean (m^{3}s^{-1})	Standard error (m^{3}s^{-1})	− 95% confidence limit (m^{3}s^{-1})	+ 95% confidence limit (m^{3}s^{-1})	C.O.V. (%)	Mean (m^{3}s^{-1})	Standard error (m^{3}s^{-1})	− 95% confidence limit (m^{3}s^{-1})	+ 95% confidence limit (m^{3}s^{-1})	C.O.V. (%)
January	113·7	2·513	108·8	118·6	76·0	163·4	4·603	154·4	172·4	58·7
February	77·5	1·805	74·0	81·0	76·9	108·2	2·773	102·8	113·6	49·2
March	83·3	2·568	78·3	88·3	105·7	108·4	3·846	100·9	115·9	73·9
April	253·8	8·259	237·6	270·0	111·2	233·8	11·16	211·9	255·7	99·7
May	385·0	7·605	370·1	399·9	68·7	302·1	8·589	285·3	318·9	61·3
June	204·7	3·619	197·6	211·8	60·5	199·2	4·924	189·5	208·9	52·4
July	117·1	1·546	114·1	120·1	45·7	108·0	1·332	105·4	110·6	25·7
August	88·3	0·887	86·6	90·0	34·5	87·8	0·882	86·1	89·5	20·9
September	74·1	0·875	72·4	75·8	38·9	79·9	0·949	78·0	81·8	24·3
October	92·8	3·526	85·9	99·7	130·5	107·5	4·013	99·6	115·4	77·2
November	300·0	12·495	275·5	324·5	142·3	291·8	12·115	268·1	315·5	84·8
December	227·4	6·623	214·4	240·4	101·0	286·0	9·848	266·7	305·3	71·7

Table 3. *Magnitude and duration of annual minimum and maximum discharges for the Tana River*

	Pre-dam (1941–1979)				Post-dam (1982–1996)			
	Mean (m^3s^{-1})	− 95% confidence limit (m^3s^{-1})	+ 95% confidence limit (m^3s^{-1})	C.O.V (%)	Mean (m^3s^{-1})	− 95% confidence limit (m^3s^{-1})	+ 95% confidence limit (m^3s^{-1})	C.O.V (%)
1-day min.	32·0	26·6	37·4	51·9	50·7	38·1	63·3	46·6
3-day min.	33·2	30·1	36·3	51·2	55·6	48·9	62·2	41·2
7-day min.	35·0	32·9	37·0	49·5	59·8	55·4	64·1	38·6
30-day min.	41·0	39·9	42·0	45·6	68·1	66·0	70·2	34·2
90-day min.	52·2	51·4	53·0	44·4	78·0	76·8	79·2	30·6
1-day max.	900·7	704·8	1096·5	67·1	815·9	624·9	1006·9	43·9
3-day max.	854·9	752·6	957·2	65·3	734·5	636·3	832·6	46·0
7-day max.	782·8	720·0	845·6	67·3	668·8	610·0	727·6	46·9
30-day max.	593·4	567·9	618·9	75·0	502·0	478·3	525·7	52·7
90-day max.	392·1	381·2	403·0	84·1	352·2	341·1	363·2	60·6

Table 4. *Timing of maximum and minimum river flows for the Tana River*

Parameter	Pre-dam period (1941–1979)				Post-dam period (1982–1996)			
	Mean	– 95% confidence limit	+ 95% confidence limit	Coefficient of variation (%)	Mean	– 95% confidence limit	+ 95% confidence limit	Coefficient of variation (%)
Julian date of annual minimum (Jan–Jun)	77.7	70.1	85.4	30.5	80.7	56.4	105.0	56.6
Julian date of annual minimum (Jul–Dec)	278.1	267.9	288.3	11.3	278.9	268.0	289.8	7.0
Julian date of annual maximum (Jan–Jun)	123.5	116.0	130.9	18.6	108.8	85.5	132.2	40.3
Julian date of annual maximum (Jul–Dec)	322.8	314.7	330.9	7.8	323.5	304.9	342.0	10.4

Table 5. *Frequency and duration of high and low pulses on the Tana River*

	Pre-dam period (1941–1979)				Post-dam period (1982–1996)			
	Mean	– 95% confidence limit	+ 95% confidence limit	Coefficient of variation (%)	Mean	– 95% confidence limit	+ 95% confidence limit	Coefficient of variation (%)
Low pulse count	6·0	5·1	6·9	44·2	4·5	2·7	6·4	60·1
High pulse count	5·7	5·0	6·4	36·5	6·1	4·9	7·3	37·2
Low pulse duration (days)	14·6	12·5	16·6	42·0	7·9	3·9	11·9	75·3
High pulse duration (days)	15·0	13·2	16·8	37·2	15.1	12·3	17·9	34·8

3-day flows remained virtually unchanged from the pre- to the post-dam period (Table 3). However, the 7-day, 30-day and 90-day mean maximum annual discharges all declined significantly ($p < 0{\cdot}01$) over the same period.

Neither the timing of highest or lowest water conditions within a year changed (Table 4) nor have the low or high pulse counts changed from the pre-dam and post-dam period (Table 5). Duration of high pulses also remained virtually unchanged over the same period. However, the mean low pulse duration has decreased significantly ($p < 0{\cdot}01$) from 14·6 days to 7·9 days, from the pre- to the post-dam period, respectively.

Both the annual rises and the annual falls of the Tana River have increased significantly ($p < 0{\cdot}01$) from the pre- to the post-dam period (Table 6). Mean rise rates from the pre- to the post-dam period remained virtually unchanged while mean fall rates increased significantly ($p < 0{\cdot}01$) from 15·7 ms^{-1} to 21·6 ms^{-1}.

Hydrologic impacts on riverine forests: HEC-RAS model analysis

Data sources and methods

Water surface profiles associated with peak discharges on the Tana River were calculated to determine the frequency of flooding of the vegetation sample plots. Water profiles were computed using the recently released HEC-RAS computer program (HEC-RAS, 1995). The main objective of the HEC-RAS program is to compute water surface elevations at all locations of interest for given flow values.

HEC-RAS performs one dimensional water surface profile calculations for steady, gradually varied flow in natural or constructed channels. The computations begin at the upstream end of a study reach and proceed cross-section by cross-section, to the lower end of the reach (Hoggan, 1997). The measured distances between cross-sections are referred to as the reach. Water surface profiles are computed from one cross-section to the next by solving the energy loss equation with an iterative procedure called the ‘standard step method’. The energy head loss between cross-sections is comprised of friction losses (computed using Manning’s roughness coefficient, n) and contraction or expansion losses (HEC-RAS, 1995). The required information for a cross-section consists of: the reach of the river; a description of the X and Y coordinates (station and elevation points); downstream reach lengths; roughness coefficients; main channel bank stations; and contraction and expansion coefficients (HEC-RAS, 1995). A major assumption made in all water profile calculations performed here was that the flow of the Tana River through the various reaches was steady, i.e. the depth, velocity, and discharge remained constant with time at a particular location on the river.

In September of 1995, the heights of 71 vegetation sample plots above the dry season river level were measured. These plots were chosen to be representative of the riverine forest found within 3 km of the river channel (Maingi, 1998). A David White Level (tilting dumpy level) and an accompanying leveling rod were used in this survey. Each plot was surveyed and leveled to the nearest river section. Within each 25 × 25 m plot, six measurements were taken to determine an average plot height. The lowest and highest lying plots were at a mean height of 0·50 m and 5·02 m, respectively, above the dry season river level.

By comparing discharge records between Garissa gauging station and a temporary gauging station at Bura in 1981–1982 period, Hughes (1985) established that a flood observed at the Garissa gauging station would take approximately 24 h to reach Bura (Fig. 2). Therefore, flows observed at Bura lag Garissa flows by a day. All vegetation sample plots were surveyed between 18 September and 28 September, 1995. The corresponding flows at Garissa for these days would be those for the period 17 to 27 September 1995. The maximum discharge during this period was 99 $\mathrm{m}^3\mathrm{s}^{-1}$, the

Table 6. *Rate and frequency of change in the Tana River flows*

	Pre-dam period (1941–1979)				Post-dam period (1982–1996)			
	Mean	– 95% confidence limit	+ 95% confidence limit	Coefficient of variation (%)	Mean	– 95% confidence limit	+ 95% confidence limit	Coefficient of variation (%)
Fall rate (m^3s^{-1})	– 15·7	– 18·3	– 13·0	51·9	– 21·6	– 25·8	– 22·6	33·0
Rise rate (m^3s^{-1})	31·3	25·5	37·1	57·4	26·8	21·3	32·4	38·9
Fall count	67·2	63·0	71·5	19·5	82·3	74·6	89·9	17·4
Rise count	40·0	33·0	47·1	33·0	78·1	70·6	85·5	17·9

minimum 89 m^3s^{-1} and the mean 93·3 m^3s^{-1}. We therefore assumed the discharge equivalent to 'dry season' river level to be 93 m^3s^{-1}.

Cross-section data for the river were obtained from a published report on the results of river survey investigations carried out in May–June 1978 by Sir McDonald and Partners Limited for the Republic of Kenya (NIB, 1979). In addition to the channel cross-section data, there were several corresponding surveys across the floodplain. Four of the 10 channel cross-sections surveyed were within a stretch of the river where the vegetation plots were located. Individual river channel cross-sections were scanned and the following determined and entered into the HEC-RAS program: station and elevation points defining each of the four cross-sections, downstream reach lengths, and, main channel bank.

Along with channel cross-section data, the river survey report (NIB, 1979) also included water surface elevations corresponding to two flows: the floods of April 1978 (with a maximum discharge of 1025 m^3s^{-1}) and a flow of 150 m^3s^{-1}. These marks were also converted to actual elevations that would provide useful reference points in calibrating the HEC-RAS simulation model. Manning's n of between 0·020 and 0·027 were used for the various reaches of the river. The default contraction and expansion coefficient values (0·1 and 0·3 respectively) were used, as they are typical of gradual transition conditions (HEC-RAS, 1995).

In all analyses, flow was considered to be sub-critical and therefore only boundary conditions for the downstream end of the river needed to be entered. An energy slope of 0·00033 was entered for calculating normal depth (Manning's equation) at that location. In general, the energy slope can be approximated by using the average slope of the channel or the average slope of the water surface in the vicinity of the cross-section (HEC-RAS, 1995). The average slope of the water surface in the downstream reach of the river was calculated as approximately 33 cm per kilometer (NIB, 1979).

Once boundary conditions had been entered, the steady flow data were entered. Flow values for which water elevation profiles were required were entered for the most upstream reach. Initial analysis began using the two reference discharges: 150 m^3s^{-1} and 1025 m^3s^{-1}. Water surface elevations corresponding to each of the flows were known for each cross-section. Using this information, some minor adjustments in Manning's n were made in order for the computed water surface elevations across each cross-section to best match the observed flows. The largest difference between observed and calculated water surface elevation was for the most downstream cross-section and corresponded to a discharge of 150 m^3s^{-1}. The observed water surface elevation at this cross-section for a discharge of 150 m^3s^{-1} was 103·8 m above sea level whereas the calculated water surface was 103·27 m above sea level. The observed water surface elevation for a discharge of 1025 m^3s^{-1} was 107·0 m while the calculated one was 107·41 m. Thus, the HEC-RAS simulation model was successfully calibrated (to within 0·5 m), and ready for calculation of water profiles.

Since all plot heights had been measured from the river surface during the low flows of September (taken for this analysis to correspond to a discharge of 93 m^3s^{-1}), the water surface elevation corresponding to this discharge needed to be computed. The calculated water surface at the most downstream cross-section corresponding to this flow was 102·76 m above sea level. Therefore, the height of each plot above dry season river level was added to this value (102·56 m) in order to convert each to a height above sea level at the most downstream river cross-section. The lowest plot, which was only 0·5 m above dry season river level, was at an elevation of 103·26 m above sea level while the highest plot, which was 5·02 m above the river, was at an elevation of 107·78 m above sea level.

The next task was to determine the discharge necessary to raise the water surface elevation to the height of each plot (as measured from the most downstream river cross-section). Various discharge values ranging from 75 m^3s^{-1} to 1200 m^3s^{-1} were entered at the most upstream cross-section and the computed water surface elevations corresponding to each discharge read at the most downstream cross-section. By comparing the computed water surface elevations with the elevation of each plot (above sea level), it was possible to determine the discharge necessary to raise the water surface to the elevation of the plot. Any discharge greater than that value would result in the plot being inundated.

Once the minimum discharge necessary to inundate each plot had been determined, the number of days the plot had been inundated for any period between 1941 and 1996 could be extracted from the daily discharge record at Garissa. One main assumption made here is that there is minimal loss of water through evaporation, transpiration, and through the river bed between Garissa and the first river cross-section used in this analysis.

In order to determine whether there has been a significant change in the number of days each plot was inundated since construction of Masinga Dam, the Garissa daily discharge record was divided into a pre- and a post-Masinga Dam period (as described in earlier sections). The number of days the flow at Garissa exceeded the minimum discharge required to inundate each plot was extracted from each period.

Results

The four lowest lying vegetation sample plots (19, 46, 55 and 31), at elevations of less than 2 m above dry season river level and requiring flows less than 350 m^3s^{-1} to be inundated, showed an increase in days flooded from the pre- to the post-dam period (Fig. 8). The remaining vegetation sample plots (at elevations greater than 2 m above dry season river level) experienced a decrease in days flooded over the same period. Paired-comparison tests (parametric and non-parametric) of average days flooded for each plot before and after dam construction revealed significant differences ($p < 0{\cdot}01$). Any vegetation sample plot at an elevation less than 1·80 m above dry season river level has experienced more flooding since the construction of Masinga Dam, while plots above this elevation have experienced a reduction in days flooded (Fig. 9). The

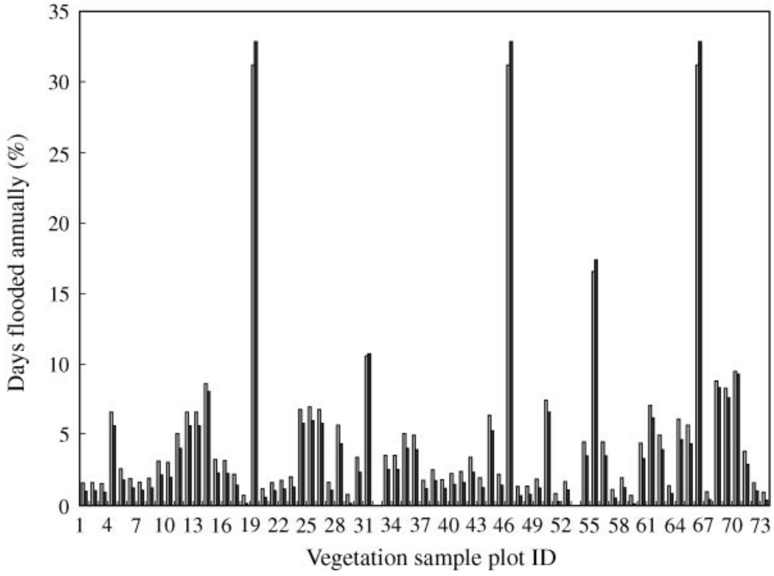

Figure 8. Frequency of flooding for each of the surveyed 71 vegetation sample plots. Pre-dam period (□); post-dam period (■).

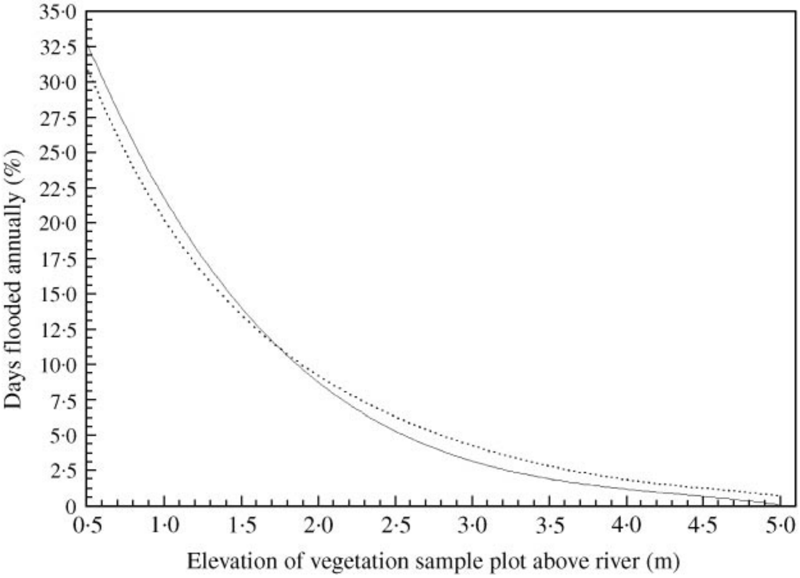

Figure 9. Frequency of flooding (measured as the percentage number of days each plot was flooded annually) plotted against elevation of each vegetation sample plot above dry season river level. Pre-dam period (··········); post-dam period (———).

average increase in days flooded for plots below 1·80 m above dry season river was 4·7%, while higher elevation plots (elevations greater than 1·80 m) experienced an average decrease of 67·7%.

Duration of flood pulse for all vegetation sample plots in the pre-dam period averaged 8·2 days and ranged from 4·9 days to 16·5 days. However, since construction of the

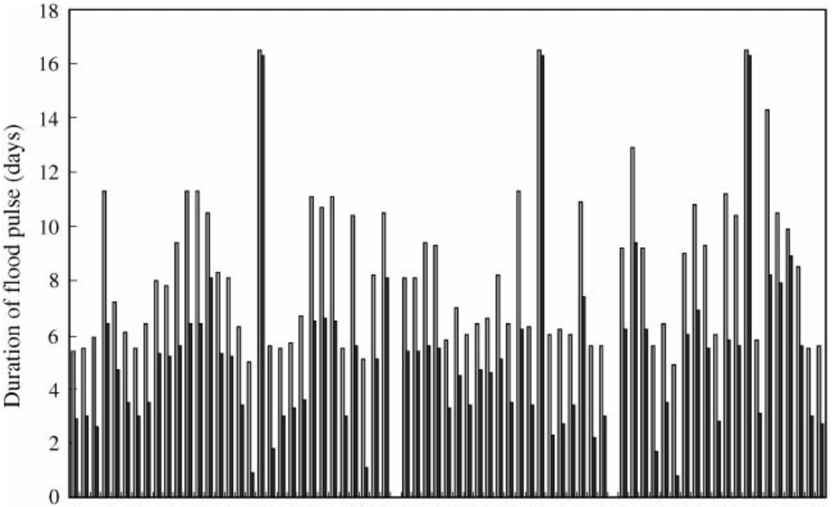

Figure 10. Mean duration of flood pulses for 71-surveyed vegetation sample plots for the pre- (▢); and post-dam (■) period.

Masinga Dam, the average duration of flooding decreased to 5·2 days (with a range from 0·8 days to 16·3 days). Duration of flood pulse has declined for all vegetation sample plots by an average by 87·6%, since construction of Masinga Dam (Fig. 10).

Discussion and conclusions

Through flood frequency analyses and computation of various indicators of hydrologic alteration, we have demonstrated a significant modification in the hydrologic regime of the Tana River since the construction of Masinga Dam in 1981. We also estimated the frequency and duration of flooding for 71 vegetation sample plots along the Tana River floodplain by running a hydrologic water profile simulation program (HEC-RAS). Results from these analyses indicated that plots situated less than 1·8 m above the dry season river level experienced significant increases in number of days flooded, while those situated above this elevation experienced significant declines in days flooded. We also found that the mean flood pulse duration for the plots declined from 8·2 days to 5·2 days from the pre- to the post-dam period. Results from flood frequency analyses indicated a significant reduction in floods with a recurrence interval of five years or greater. This observation is consistent with that from other studies that have documented reduced peak flows in the downward reaches of dams (Johnson, 1994; Kondolf, 1997; Brandt, 2000).

In addition to reduction in peak discharges, most dams will trap large amounts of sediment in the reservoir, releasing only a proportion of the sediment into the downstream reaches. Sedimentation rates in reservoirs behind the earlier smaller dams along the Tana River have been very high. For instance, about 12·6 million m^3 of sediment were deposited in the Kindaruma reservoir between 1968 and 1970 (Ongwenyi *et al.*, 1993*b*). It is expected that this sedimentation rate has continued within the Masinga reservoir. Along meandering rivers, the primary effect of diminished peak flows and sediment loads is a reduction in the meandering rate stemming from the rivers' decreased capacity to erode and deposit sediment (Petts, 1996). Ox-bow lakes are formed following the cutting off of meanders. Meander cutting and abandonment of river channels leads to a geomorphologically diverse floodplain, and consequently the development of a highly complex forest. Less meandering decreases the formation of establishment sites and, thus, reduces recruitment of pioneer species (Bradley & Smith, 1986; Johnson, 1992; Scott *et al.*, 1997; Friedman *et al.*, 1997).

Using Landsat MSS (1975, 1984), Landsat TM (1985), and SPOT HRV (1989, 1996) data for the study area, Maingi & Marsh (2001) demonstrated a reduction in the meandering rate for the Tana River since construction of the Masinga Dam. Various studies along the Tana River floodplain have documented poor regeneration of many of the forest species (Marsh, 1976; Hughes, 1985; Medley, 1990; Maingi, 1998). We can therefore expect the poor regeneration problem to be further exacerbated by the less frequent meander migration and cutoff. Riverine species that become established along ox-bow lakes will continue to decline as new ox-bows are no longer established, and also because the old ox-bows are no longer replenished with floodwater and silt as a result of diminished peak flows. Likewise, species occurring on higher elevations that are now flooded less frequently may also experience changes in composition favoring those that can better tolerate the increasingly xeric conditions. Overall we can expect the Tana riverine forest to continue narrowing and become less diverse if current hydrologic conditions remain or are further modified with more upstream dam building.

Seasonal flooding promotes the exchange of materials and organisms among the mosaic of habitats in the floodplain, and it is the hydrological regime that plays the key role in determining the level of biological productivity and diversity (Bayley, 1995). Variability of monthly discharges for the Tana River has been drastically reduced, with reduced peak discharges and augmented low flows. Mean monthly discharges for the December–March period are now 17–44% higher in the post-dam period than in the pre-dam period. Peak flow months of April–May and November–December have been affected differently by dam construction. May floods have seen declines of up to 22%, while November flows remained virtually unchanged.

Higher and prolonged low season flows imply a higher water table during the dry season with possible water logging in the low-lying areas of the floodplain. In deed, the mean low flow pulse duration has changed from about 15 days to 8 days from the pre- to the post-dam period. Mean fall rates for the river have also increased since dam construction. Rapid rises and recession, or irregular and unpredictable floods offer little resource value and disturb the floodplain environment (Petts, 1996). Low season flows are now higher than before dam construction, a development likely to reduce flowering, and fruit production of many species within the riverine forest. Kinnaird (1992) found that fruiting of many important riverine tree species along the Tana River floodplain was triggered by dry conditions created by low river flows rather than low rainfall. The altered flows demonstrated in this paper may have affected flowering phonologies and potentially reduced fruit and seed production, and thereby significantly reduced available primate food sources.

Diminished peak flows and deposition of sediments along the Tana River floodplain suggests a corresponding reduction in the area subjected to flooding, and an overall reduction in the floodplain fertility. This being the case, we would eventually expect a reduction in the extent of the floodplain forest. In a related study using remote sensing data, Maingi (1998) found that the average extent of the riverine forest from the river channel declined by at least 200 m between 1989 and 1996. This statistic was supported by field observations indicating that many of the large trees towards the edge of the riverine forest showed signs of die-back. In addition, computed forest landscape measures derived for the riverine forest between 1989 and 1996 indicated increasing fragmentation. Mean patch size and area-perimeter ratio of the riverine forest decreased by 31% and 4%, respectively over the same period.

The results of this study have documented the magnitude of change in the hydrologic regime of the Tana River due to dam construction. These changes will have a negative impact on the unique riverine forest occurring along the Tana. There are plans for construction of two additional dams in the upper Tana River basin. Such developments can only exacerbate the impact on the riverine forest and flood recession agricultural activities in the lower floodplain. Previous environmental impact studies on dam construction across the Tana River have focused on the reservoir area and ignored the downstream impacts. This study has demonstrated that adverse environmental effects resulting from damming of the Tana River extend beyond the reservoir and therefore there is a need for future studies addressing downstream effects as well.

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