Institute of Hydrology

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Natural Environment Research Council

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HYDROLOGICAL IMPACT ASSESSMENT OF THE PROPOSED RIVER RODING AMENITY BARRAGE

Report to the London Borough of Barking and Dagenham

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Executive summary

1. The London Borough of Barking and Dagenham proposed the construction of an amenity barrage across the River Roding at Four Gates to maintain water levels at low tide. The Institute of Hydrology was commissioned to assess the impact on flood risk in the lower Roding of this barrage.

2. The Institute had just completed an analysis of flood levels along the lower Roding for the National Rivers Authority as part of its review of flood defences in the Thames estuary. The models developed for that study were modified to determine the impact of the barrage.

3. Water levels in the study reach are controlled by the interaction of fluvial flows (including effluent discharges), estuary levels and operation of the Barking barrier.

4. The Barking barrier spans the Roding at its mouth and when closed precludes all water from the Thames and thus removes the tidal influence. The barrier is closed when the Thames barrier is closed and can only be opened when the water levels either side equalise.

5. Beckton Sewage Treatment Works normally discharges its effluent directly into the Thames. However when the Barking barrier is closed and Thames level reaches 4.4 m the effluent is discharged to the Roding.

6. Records of flow in the Roding at Redbridge and water levels at various locations in the Thames estuary were available for the period 1950-1991. Insufficient data on effluent discharges from Beckton STW were available, so data for the period 1950-1991 was generated synthetically.

7. A hydraulic model had been constructed to calculate water levels along the study reach given the fluvial flow, barrier status, effluent discharge and water levels at the Roding mouth. A second version of this model was constructed which included the proposed barrage.

8. For the period 1950-1991 a series of annual maximum water levels at nine critical cross-sections had been derived using the model without the barrage. A statistical distribution had been fitted to each to estimate return period 2 to 1000 years. This required a large extrapolation beyond the available data. For most cross sections the flood defences gave a level of protection greater than 1000 years return period.

9. This frequency analysis was repeated using water levels derived from the model with the barrage in place. The model predicted an increase in flood levels at most cross sections over the without-barrage case, particularly at high return periods (>50 years). However, the maximum increase was only 40 mm. For sections 1001u to 1031 the flood defences still gave a level of protection greater than 1000 years even with the barrage in place. For sections 1041 and 1048 the level of protection was reduced but only by a small amount.

10. It was concluded that the barrage would not have significant influence on flood risk in the lower Roding.

Contents

1. Background

The London Borough of Barking and Dagenham proposed the construction of an amenity barrage across the River Roding at Four Gates near Barking in Essex. The barrage was designed to maintain water levels at low tide thus providing boating/sailing facilities and improving the general appearance of the river.

During the feasibility study of the barrage it was necessary to demonstrate that it would not affect adversely the hydrological regime. In particular there was concern that the barrage might increase flood risk along the lower Roding.

The Institute of Hydrology (IH) had recently completed an analysis of flood levels (up to 1000 year return period) along the lower Roding for the National Rivers Authority (NRA) as part of it's review of flood defences in the Thames estuary (see Institute of Hydrology, 1992).

In collaboration with the NRA, IH was contracted to assess the impact of the barrage on flood risk in the lower Roding below Redbridge, some 10 km upstream of its confluence with the Thames. This was achieved by constructing a new version of the model with the barrage in place. The derived water levels were then analysed and the results were compared with those from the NRA review. The difference between the results indicated the impact of the barrage.

This report has two main parts:

(1) Sections 2 - 12, which describe the model development and the analysis undertaken for the NRA review; and

(2) Section 13, which gives the results of modelling the Roding with the barrage in place and the analysis of the derived water levels.

2. Introduction

The River Roding rises in north-west Essex and flows southward to join the River Thames at Barking, a length of some 50 km (Figure 2.1). The catchment is low lying and underlain by boulder clay on London clay with glacial gravels in the lower part of the catchment. The catchment is narrow with few major tributaries. The middle and upper reaches of the catchment are rural but the lower reaches are heavily urbanised.

The section of river under study is the 10 km reach between Redbridge and the

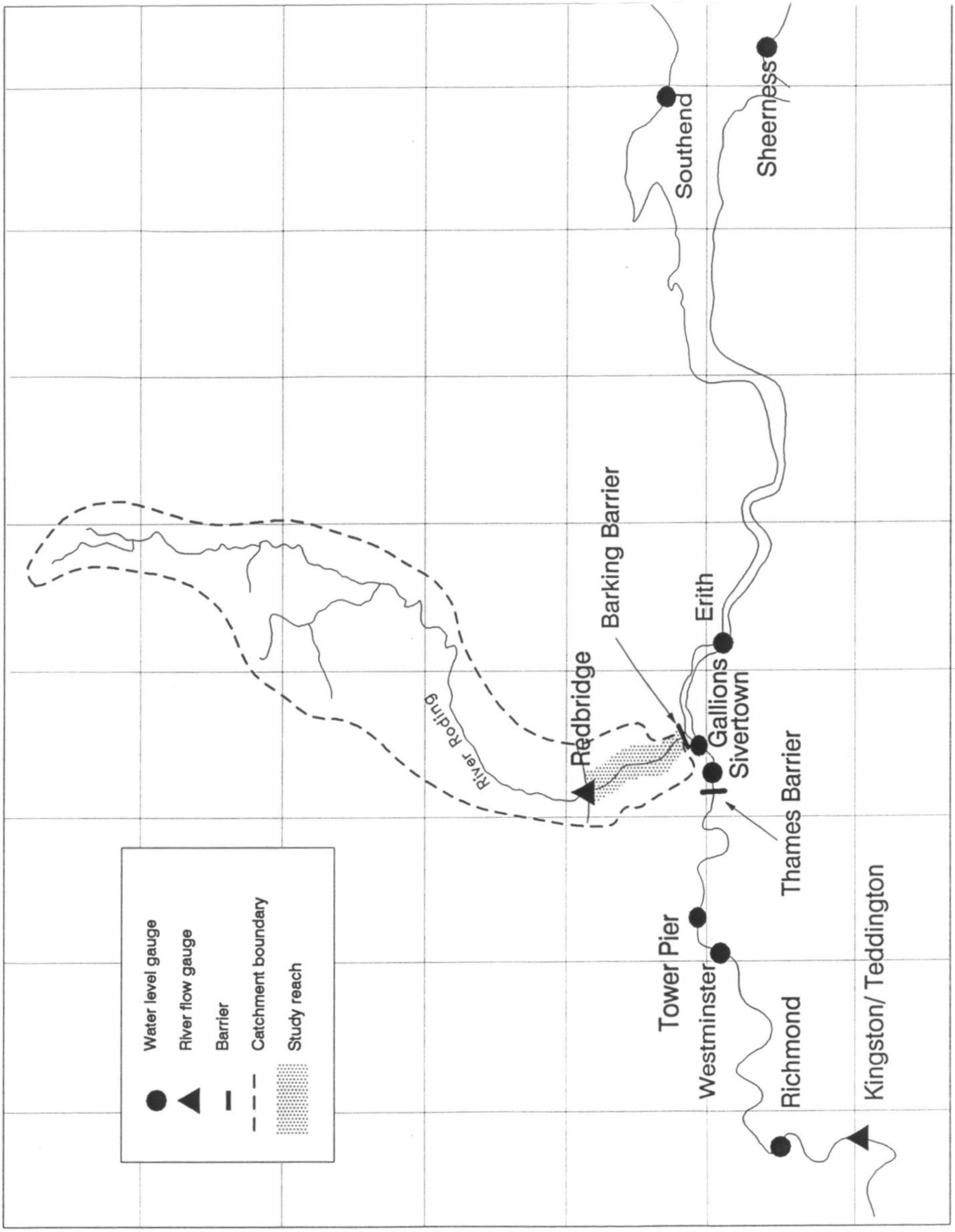

Figure 2.1 *Location map*

confluence of the Roding and the Thames. The lower part of this reach, called the Barking Creek, is tidal, the bed is composed of silt and water levels are influenced mainly by Thames tides. Normal high tide level in the Thames is 5.5 m AODN and the land adjacent to the Creek is, in places, as low as 1.5 m AODN. The Creek banks are protected by earth embankments and by the Barking barrier, which spans the river at its mouth. When closed the barrier precludes water from the Thames entering the Barking Creek. The middle section of the reach is slightly meandering, and its bed is composed of silt and gravel. Water levels are determined by the interaction of tides and fluvial flows. The riparian land is low lying and protected by embankments. Further upstream towards Redbridge, the channel bed is dominated by gravel, the river meandering and there are large variations in channel width. Water levels are controlled by fluvial flows with little tidal influence.

It is clear that stage frequency at the upstream end of the reach will be controlled by the frequency of fluvial events, whereas at the downstream end the stage frequency will follow more closely that of the tidal Thames. Levels at intermediate sections will be controlled by both tidal levels and fluvial flows to degrees according to their location.

No significant tributaries join the Roding along the study reach, but flows may be augmented significantly by effluent discharges from a large sewage treatment works which serves much of north London.

Water levels in the Thames estuary are controlled by three components: the astronomical tide which varies according to a regular, broadly predictable cycle; storm surges caused by adverse weather conditions in the North Sea; and fluvial flows from the Thames catchment.

There are two basic approaches which can be applied to this joint probability problem. The first method is called historical reconstruction. This aims to determine the actual water levels which occurred during some historical period. The available data are analysed in chronological order such that for each day the recorded river flows and estuary levels are applied to the structure functions to derive water levels in the study reach. This reconstructed water level series is then analysed statistically to determine levels of specified return period. The disadvantage of this method is that historical series are normally short and therefore estimation of extreme events, such as the 1000 year flood level, relies on a large extrapolation of the data.

The alternative approach is called synthetic generation. In this method statistical distributions are fitted separately to the observed river flow and estuary level data. Then synthetic series of flow and levels are generated by sampling randomly from these distributions. The resulting series are applied to the structure functions to produce water levels in the study reach which can be analysed statistically to determine levels of specified return period. The advantage of this technique is that there is no limit to the length of the series which can be generated. Extrapolation to extreme events is undertaken using the marginal distributions of flows and estuary levels rather that from the resulting study reach levels. The disadvantage is that the correlation between river flows and estuary levels must be determined since, for example, if high river flows and surge tides occur together as a result of particular meteorological conditions, they can not be considered to be independent. The correlation structure can have a significant influence on the resulting levels and is difficult to quantify, especially where data sets are short. The situation on the Roding is extremely complex since there are a large number of variables: Roding river flows, effluent discharges, Thames river flows and North Sea levels (both the astronomical and surge components). Consequently it was considered that a project involving synthetic generation would be many times more time consuming that one employing historical reconstruction.

In conclusion NRA agreed that historical reconstruction was a more appropriate technique for this study.

3. Requirements for the study

Since water levels in the lower Roding are determined by the interaction of Thames estuary levels and fluvial flows, calculation of a stage-frequency relationship requires an analysis of the joint probability of coincident estuary levels and fluvial flows and the resulting water level. This analysis requires:

(a) a time series of the flows in the River Roding (which would include effluent discharges);

(b) a concurrent time series of levels in the Thames at its confluence with the Roding;

(c) hydraulic models to provide water levels along the study reach given (a) and (b) both for the with and without barrage cases; and
(d) a statistical model to analyse frequency of resulting levels.

Each of these elements is discussed in detail below.

4. Fluvial flows from the Roding

Fluvial flows on the River Roding are measured at the Redbridge (TQ 415884), some 10 km upstream of its confluence with the Thames, where the catchment area is 303.3 km^2. This measurement station defines the upstream limit of the study reach. The station was established in November 1949 with construction of a broad crested weir beneath the road bridge. This was superseded in 1962 by an *Essex* profile (modified flat-v Crump) weir slightly upstream of the previous weir. Calibration of the weir above 35 m^3s^{-1} is based on model tests. All flows have remained within bank during the period of record.

The highest flow on record is 62.4 m^3s^{-1} which occurred on 22 November 1974, however, the peak flow during the 1947 flood (before the station opened) was estimated as 80 m^3s^{-1}.

The *Flood Studies Report* recommends that floods of return period up to twice the record length only (in this case 2n = 82) are estimated from the annual maximum data. Above this point the regional flood frequency curve should be employed. To effect this the two curves were merged producing the flood estimates in Table 4.1.

Table 4.1 *Peaks flows of various return periods at Redbridge*

Return period (years)	2	5	10	50	100	200	500	1000
Peak flow ($m^3 s^{-1}$)	25	33	38	55	70	82	110	140

Figure 4.1 shows the annual maximum instantaneous peak flows (1950-1990) at Redbridge plotted against return period, T, using the Gringorten plotting formula:

$$F = (m-0.44)/(n+0.12) \tag{4.1}$$

where F is the non-exceedence probability (F = 1-1/T), n is the number of years of record and m is the rank of the ith maxima. Also shown on Figure 4.1 are three curves representing:

(a) the generalised extreme value (GEV) distribution fitted to the data by the method of probability weighted moments (PWM),

(b) the results of applying the *Flood Studies Report* (NERC, 1975) regional growth factors to the at-site estimate of the mean annual flood

(c) a 'best' curve representing a merging of (a) and (b).

Daily mean flow data were available from the Surface Water Archive for the period 1950-1991.

Clearly, flow varies during the day so that the daily mean flow series does not exhibit all the characteristics of the flow hydrograph. Nevertheless, provided that the flow

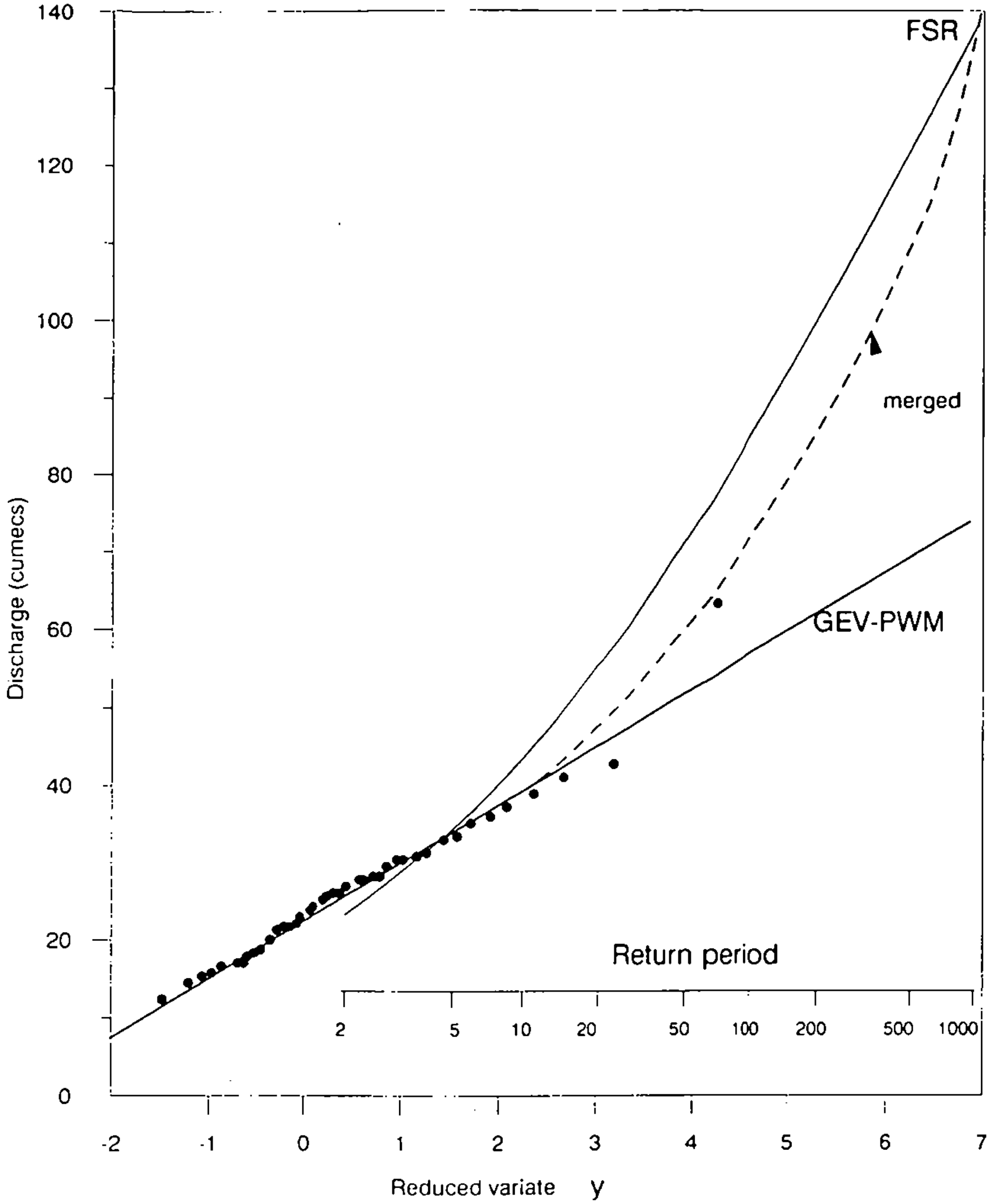

Figure 4.1 *Flood frequency curves for the River Roding at Redbridge*

does not vary significantly, a daily mean flow is adequate for the study. Indeed if the flow can be assumed to be constant over a tidal cycle, the analysis of the interaction of fluvial flows and tides is greatly simplified. This is assumption is considered further below.

Table 4.2 shows the physical characteristics of the catchment. The Roding is in one of the driest parts of the UK with mean annual rainfall only 635 mm and more extreme rainfall, M52D (the 5 year rainfall of 2 day duration) and RSMD (an index of flood-producing rainfall) are also amongst the lowest in the country.

Table 4.2 *Physical characteristics for the Roding catchment*

Morphological	
Drainage area (km^2)	303.0
Main stream length (km)	62.6
Main stream slope, S1085 ($m\ km^{-1}$)	1.22
Stream density (junctions km^{-2})	1.17
Climatological	
Mean annual rainfall, SAAR (mm)	635.
year rainfall of 2 day duration (mm)	42.7
Index of flood-producing rainfall, RSMD (mm)	15.9
Land type	
Urban area (%)	10.
Lakes (%)	0.
Soil WRAP class 3 (%)	80.
class 4 (5)	20.
Hydrological	
Mean flow (m^3s^{-1})	1.86
10% flow (m^3s^{-1})	4.46
Base flow index	0.40

The lower reaches of the catchments are underlain by soils which have a low, class 4, winter rain acceptance potential (WRAP), thus they tend to generate high runoff. The middle and upper reaches of the catchment contain more permeable class 3 soils. The baseflow index provides a indication of the proportion of runoff that derives from stored sources based on an arbitrary separation of the daily flow hydrograph. Catchments draining impervious catchments typically have baseflow indices in the range 0.15 to 0.35 whereas a chalk stream may have a BFI of 0.9 as a consequence of the high groundwater component in river discharge. The value of 0.4 for the Roding suggests that a high proportion of rainfall feeds the quick response component of the hydrograph.

As indicated above the urban 10% of the catchment is concentrated near to the gauging station. Hydrologists who have analysed data from the Roding suggest that, on some hydrographs, runoff from the urban part of catchment can be distinguished from main rural portion. Flow hydrographs typically exhibit a steady rise to a plateau with the main peak following with a lag of around 24 hours. The rising limb may be steep, of the order of 10 hours (eg. Figure 4.2), but is normally less severe, rising more evenly to a peak after around 48 hours (Figure 4.3). This indicates that a flow of slightly less than the peak is sustained over a least one tidal cycle.

Individual hydrographs show the response to a particular rainfall profile. The effect of rainfall profile shape may be eliminated by deriving a unit hydrograph for the catchment. In his review of the *Flood Studies Report* unit hydrograph rainfall-runoff analysis, Boorman (1985) included 13 flood events from the River Roding for analysis of percentage runoff. However, unit hydrograph parameters were available only for six of these due primarily to a lack of short duration rainfall data, although some were rejected because the unit hydrograph was double-peaked. Time-to-peak of the one hour unit hydrograph ranged from 26.5 to 39.0 hours with an average of 33 hours. These results support the assumption that the Roding exhibits a relatively slow response to rainfall and that daily mean flows provide an adequate description of the flow hydrograph.

An alternative method of assessing the variability of flow in the catchment is to consider the ratio of mean to peak flow. Figure 4.4 shows a graph of the highest instantaneous flow in each month of the record (1950-1990) plotted against the mean flow for the day of the peak. It is noteworthy that even for the highest daily mean flow, 56.1 m^3s^{-1}, the peak was only 62.4 m^3s^{-1}, ie. 11% higher. The solid line in Figure 4.4 represents a one-to-one relationship, whereas the dotted line resulted from a least squares regression:

$$Qmax = 1.99 + 1.10\ Qmean \qquad (r^2 = 0.92) \tag{4.2}$$

The line slope of greater than unity shows a tendency for a greater difference between the peak flow (Qmax) and the mean (Qmean) for that day. The relationship is influenced by a large number of small events where the peak is only slightly greater than the mean flow, but provides an adequate model. Below Redbridge no major tributaries enter, but there is some lateral inflow from a higher urbanised riparian area. Runoff from this area will most likely be very rapid, reaching the Thames before the main flood peak arrives from Redbridge. Hence it is unlikely that the peak flow will be increased.

It is concluded that, ideally, flows of shorter duration than one day should be used since many hydrographs show a rapid variation in flow on the rising limb which could coincide with a surge tide. However, a flow slightly less than the peak is normally sustained for longer than a tidal cycle and the peak flow itself tends only to be a few percent higher than the plateau. Model runs (see section 8) indicated that predicted levels in the Roding were not very sensitive to this assumption. Hence, the daily mean flows, adjusted to estimate the peak using equation (4.2), were accepted as providing an adequate time series for the analysis.

In addition to lateral inflows below the gauging station, flows may be augmented from effluent discharges. This is considered next.

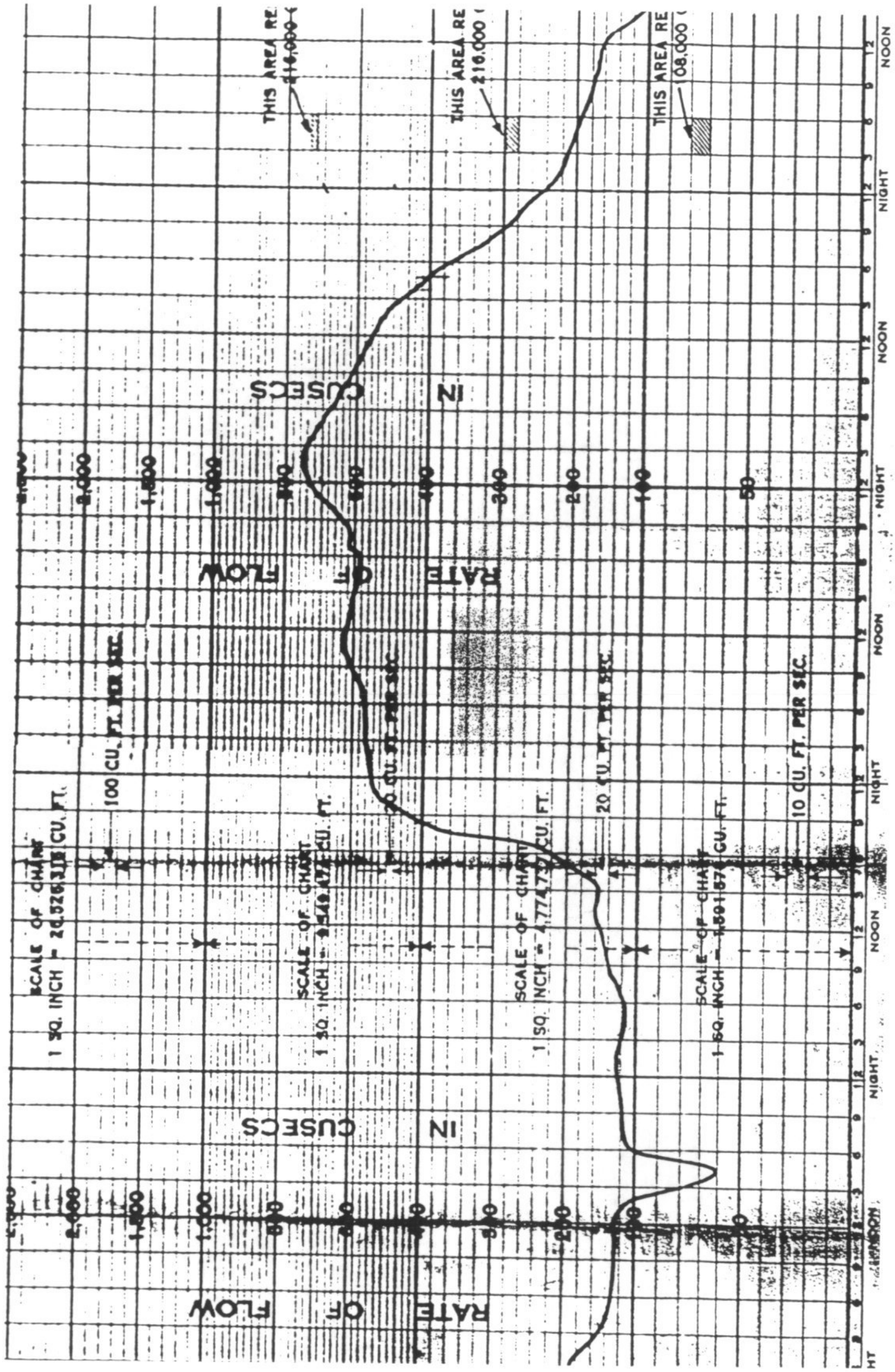

Figure 4.2 Flow hydrograph for the Roding at Redbridge 2-6/3/54

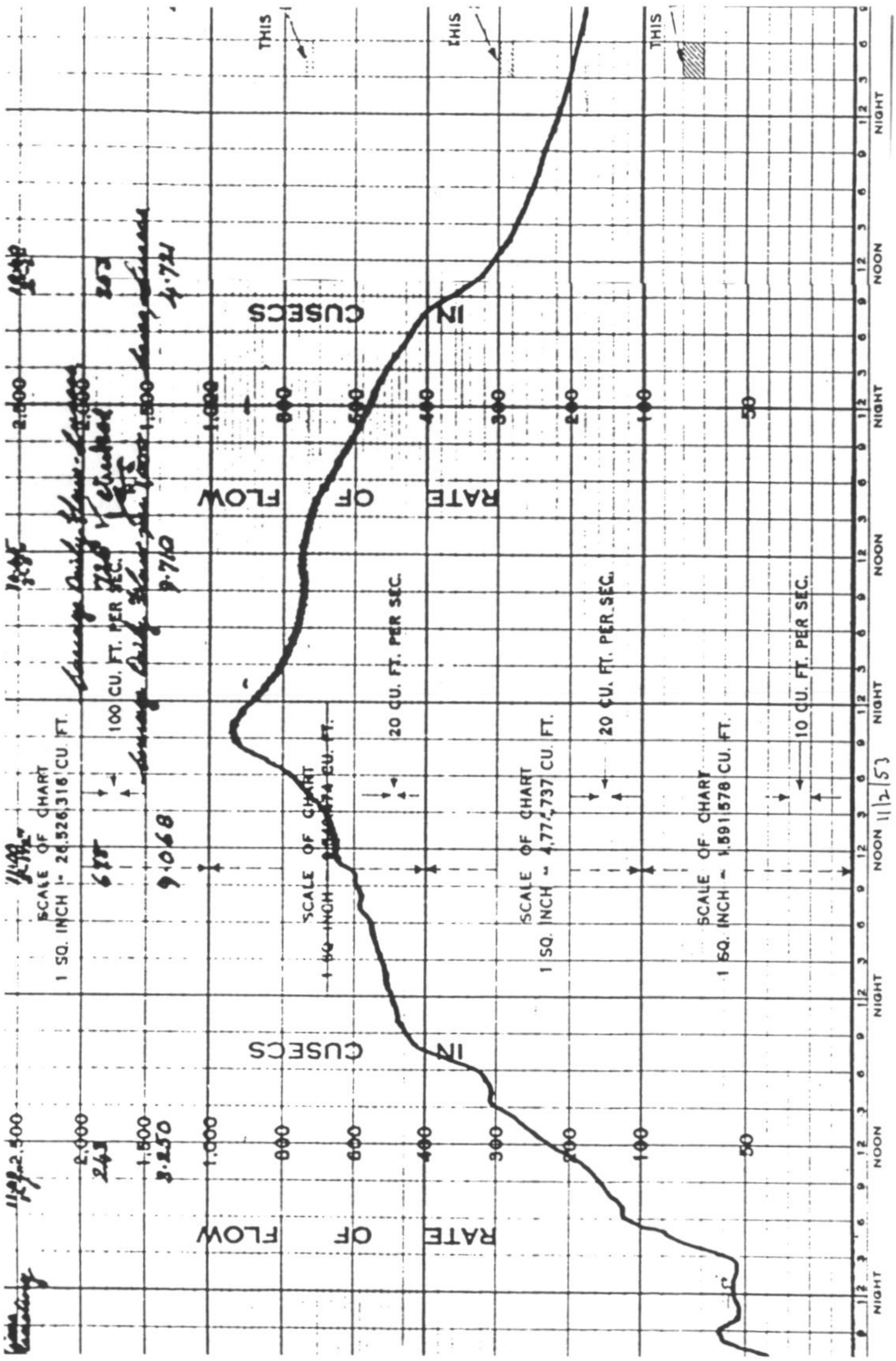

Figure 4.3 *Flow hydrograph for the Roding at Redbridge 9-14/2/53*

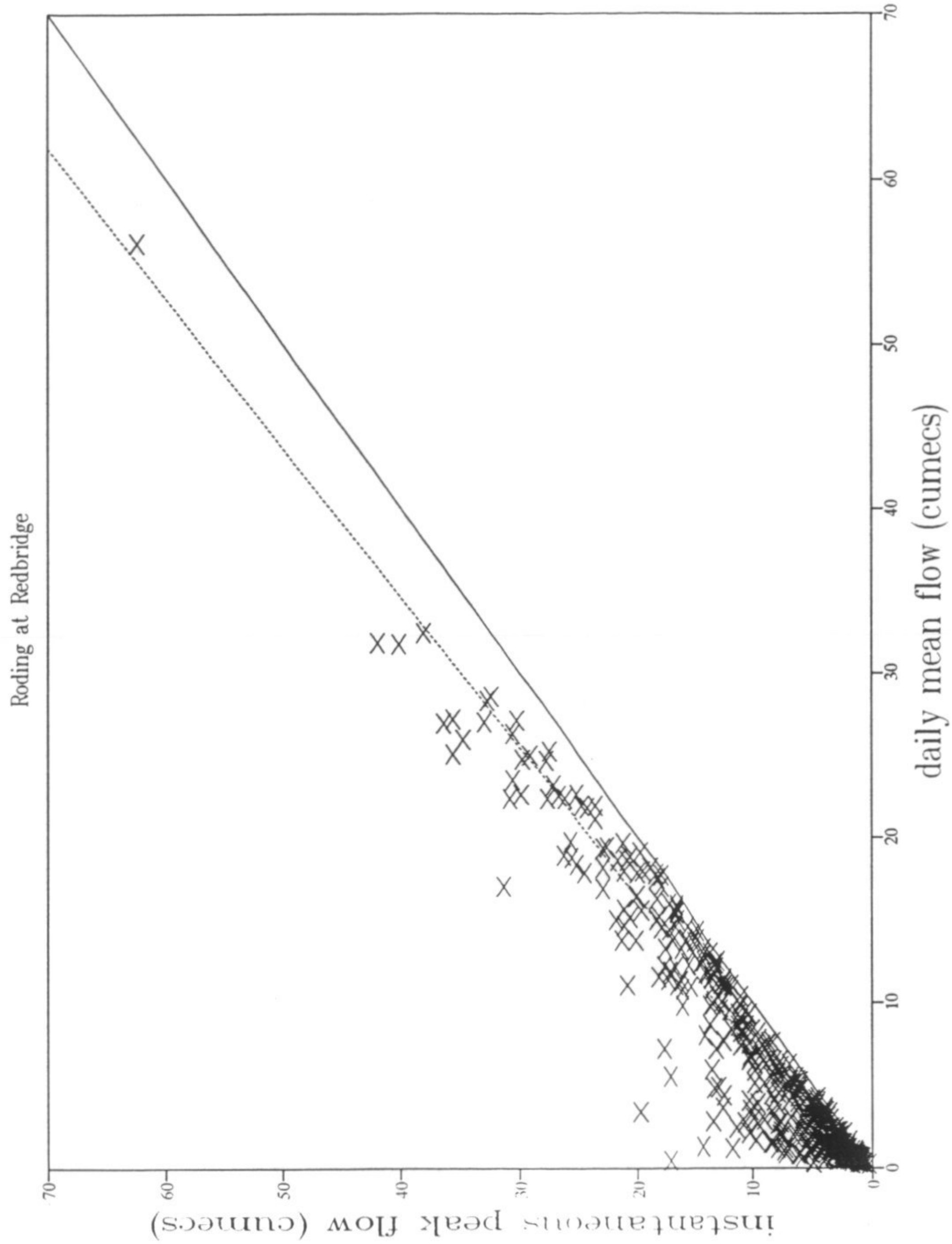

Figure 4.4 *Highest instantaneous flows at Redbridge against mean flow for the day of the peak (m^3s^{-1}).*

5. Discharges from Beckton Sewage Treatment Works

Beckton STW is sited on the right bank of the Roding at its confluence with the Thames. It has a design peak output capacity of 31.25 m^3s^{-1}. At low tide this is discharged directly into the Thames via the Beckton outfall. However, when the Barking barrier is closed, effluent from Beckton is discharged into the Roding via the auxiliary outfall, about 150m upstream of the barrier. The discharge is not diverted at low water when the Barking barrier closes, but is delayed until the Thames reaches a level at which effluent stops flowing from the treatment works and above which reverse flow would occur. As soon as the Thames level falls back below this critical level, discharging directly to the Thames is resumed. These actions minimise water levels behind the barrier. Unfortunately there are no records to quantify this critical Thames level at which water would start flowing into the works, although the bed of the outlet culvert is thought to be at 4.0 mAODN. However, according to the operators at Beckton, the Barking barrier had always been closed on past occurrences. The Thames and Barking barrier close when the level at London bridge will reach 4.87 mAODN (see section 7 below), this is equivalent to a level at the mouth of the Roding of between 4.4 and 4.7, depending on the combination of tide level and Thames flows. Thus the minimum level that effluent discharge from Beckton is diverted to the Roding is assumed to be 4.4 m. This datum is used below.

Flow into Beckton STW comes from the main interceptor sewers in London north of the Thames. When flows to Beckton STW are very high, the Abbey Mills pumping station diverts some of the flow (up to 20 m^3s^{-1}) bound for Beckton into the Channelsea river, a tidal embayment in the River Lea system. A new overflow was being considered at Beckton to permit this extra flow to be discharged directly into the Thames. The sewer 'catchment' draining to Beckton STW is around 300 km^2 (about the same as the Roding but all urban) extending from Hammersmith and Brent to Barking and its response time is considered to be around 6-7 hours. Effluent takes about 14 hours to be treated before it is discharged.

Daily mean discharge data from Beckton were available for all of 1987, most of 1989 and some of 1990. Figure 5.1 shows the 1987 discharges together with flows at Redbridge for the corresponding days. It can be seen that average discharge from Beckton is around 12 m^3s^{-1} and increases coincide with increases in flow at Redbridge as a result of storms crossing north London. However, it is also evident that there is a poor correlation between absolute values: high discharges from Beckton occurred during June, whereas flows at Redbridge were generally low. In contrast the high flows experienced at Redbridge in October and November were associated with only moderate increases in discharge from Beckton. Figure 5.2 shows discharge from Beckton for each day in 1987 plotted against flows at Redbridge. There is not a strong relationship between the two data sets, apart from the apparent decrease in variability of effluent discharge with increase in river flow, which may be a consequence of fewer data at high flows. The response of the sewer catchment is clearly much quicker than the river catchment, thus discharge from Beckton on one

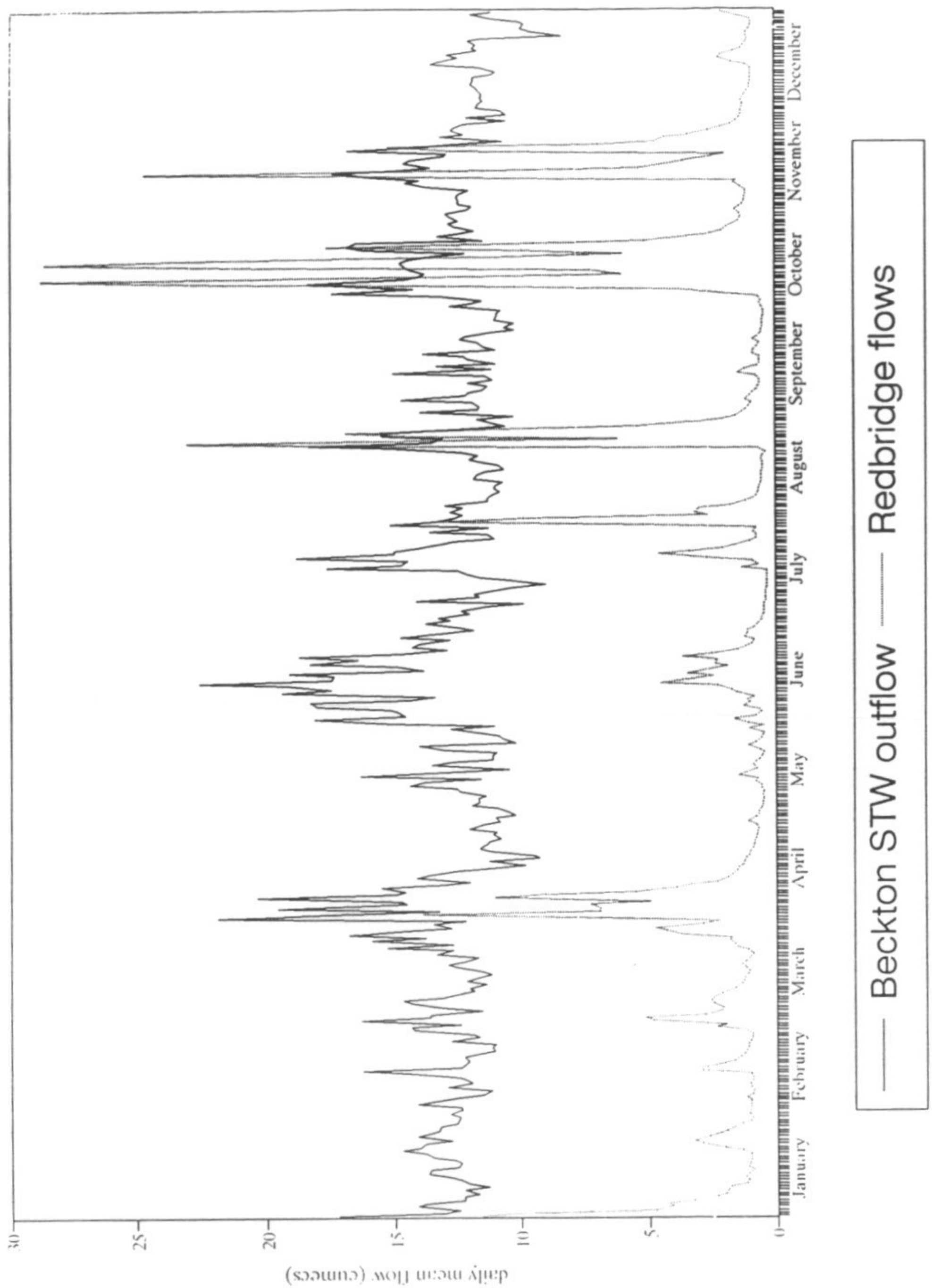

Figure 5.1 *Daily mean discharges from Beckton STW for 1987 together with daily mean flows at Redbridge for the corresponding days (m^3s^{-1}).*

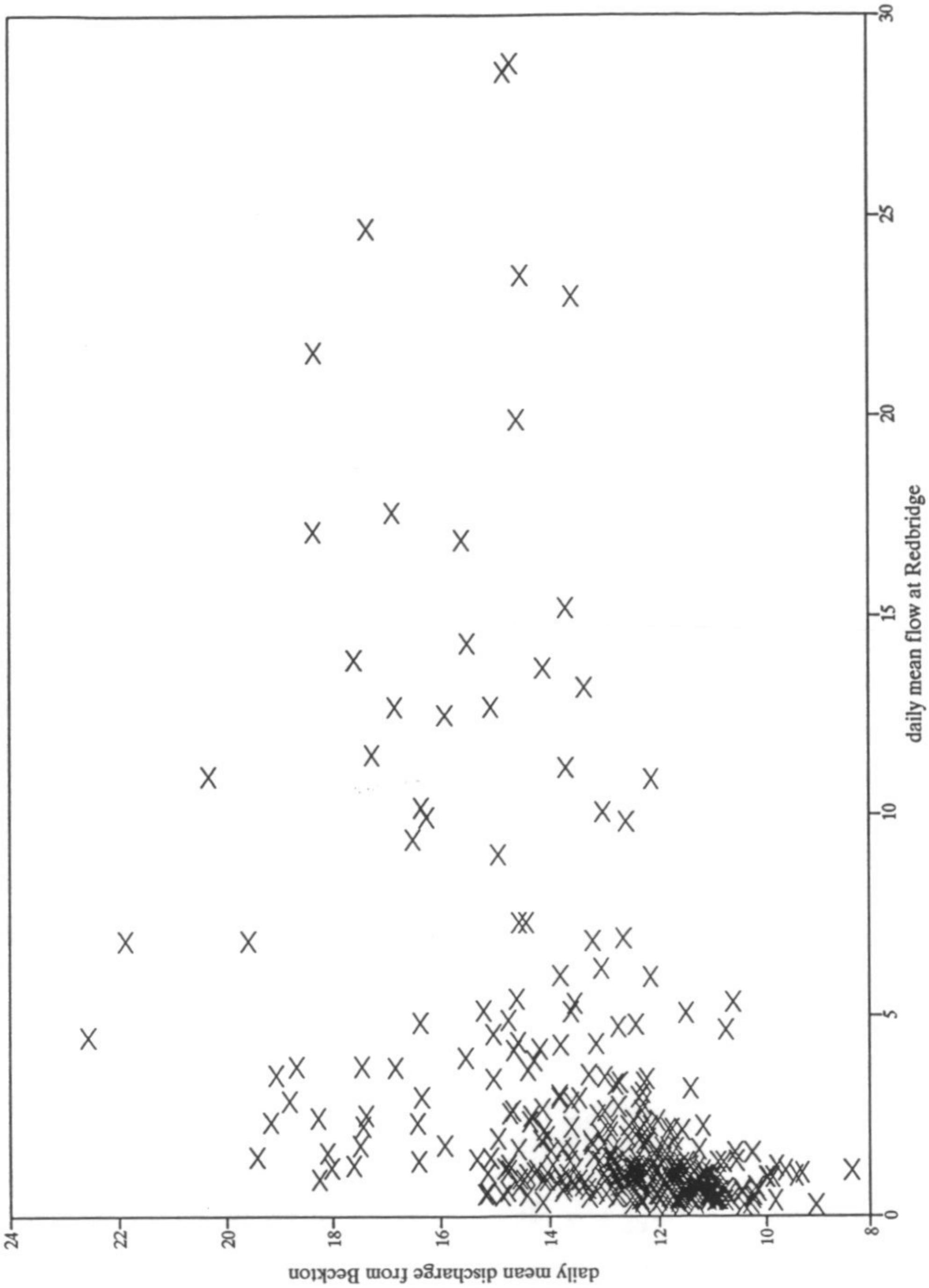

Figure 5.2 *Daily mean discharges from Beckton STW for 1987 against daily mean flows at Redbridge for the corresponding days (m^3s^{-1}).*

day might be related to flows at Redbridge on the following day. However, similar graphs to Figure 5.2, produced at lags of one and two days, showed a similar wide scatter of data points. Figure 5.3 shows histograms of discharge from Beckton for 1987 and for 1989. The two distributions have similar shapes and both exhibit slight positive skewness, but it is clear that discharges in 1987 were significantly higher. No changes to operating capacities or procedures were introduced between 1987 and 1989, thus it is assumed that the difference is due to meteorological conditions. Indeed 1987 was considerably wetter than 1989, for example the total rainfall over the Roding catchment in 1987 was 727 mm (Institute of Hydrology, 1988), compared with 554 mm for 1989 (Institute of Hydrology, 1990). Insufficient data were available to compare these with 1990.

It was concluded that, since records of discharge from Beckton were not available for the same period as flows at Redbridge, they would need to be generated synthetically by sampling randomly from a frequency distribution for each day. The distribution was derived by combining data for 1987 and 1989. The observed cumulative distribution function is shown in Figure 5.4 together with a that for a normal distribution with

mean	=	11.835
standard deviation	=	2.504

This function was used to generate synthetically the mean flow from Beckton for each day of record required.

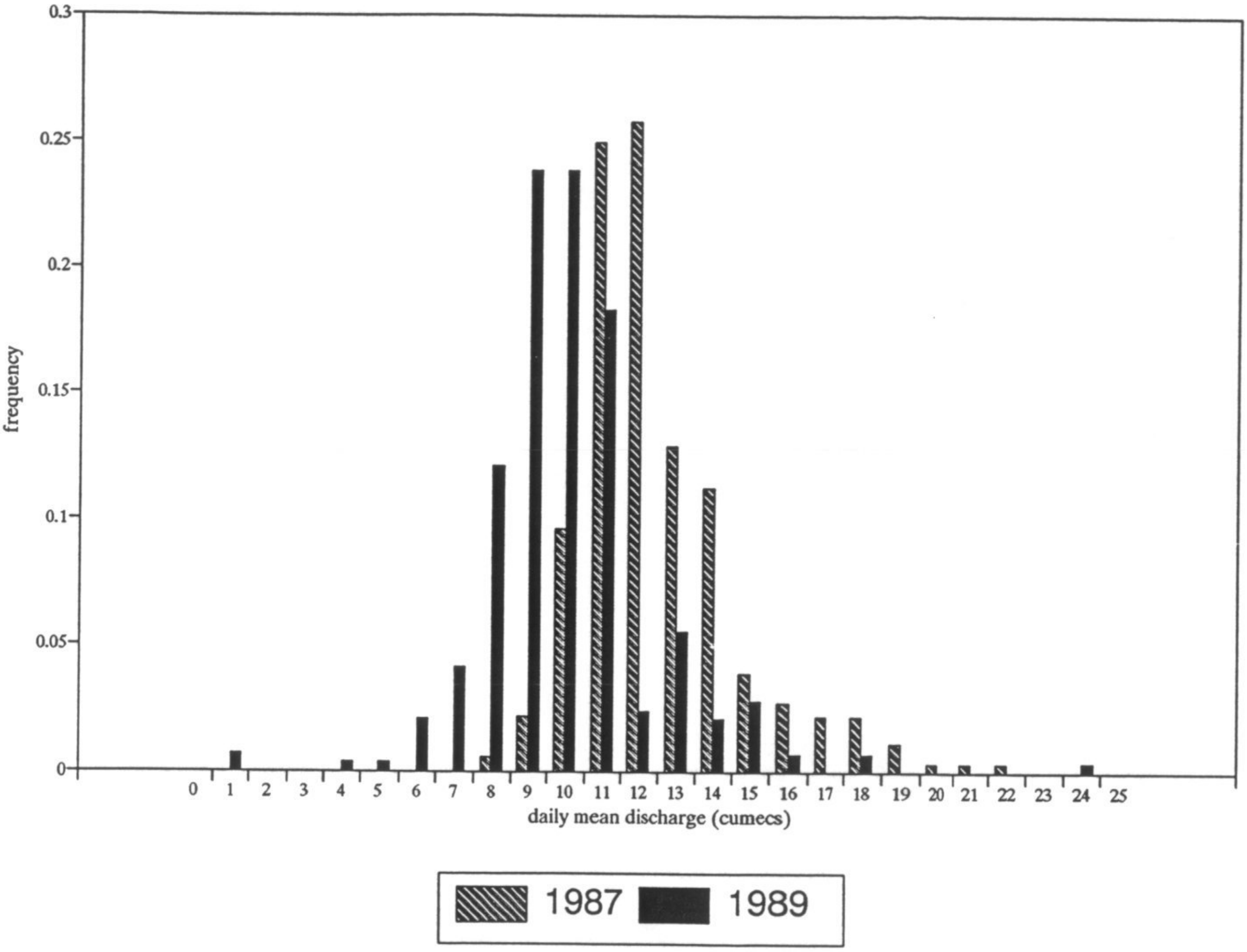

Figure 5.3 *Histograms of daily mean discharges from Beckton STW for 1987 and for 1989 (m^3s^{-1}).*

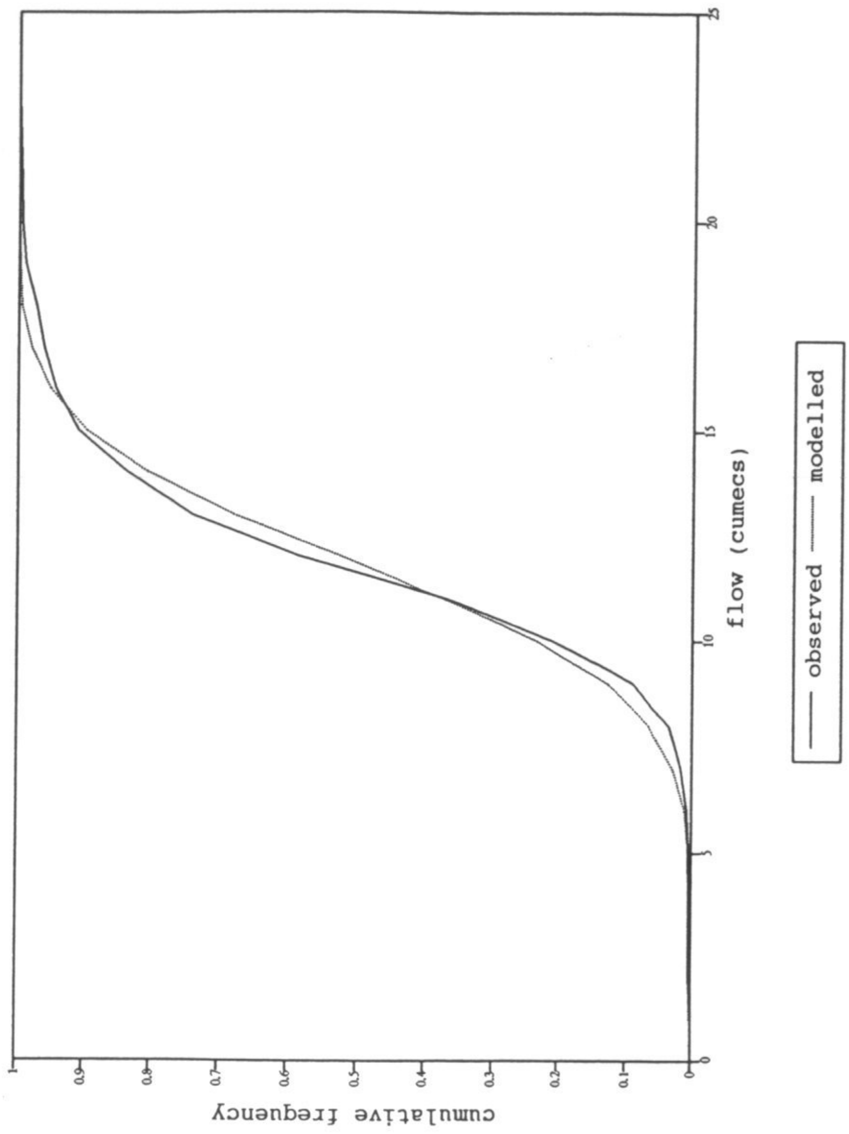

Figure 5.4 *Cumulative frequency of daily mean discharges from Beckton STW for 1987 and 1989 combined together with a normal distribution*

6. Water levels at the mouth of the Roding

A number of gauges have recorded tide level in the Thames estuary from 1950 to the present. The closest gauge to the River Roding mouth is Gallions, approx. 1.5 km upstream from the mouth and 4 km downstream of the Thames Barrier. Adjacent tide gauges are (u/s) Silvertown, just downstream of the Thames Barrier, and (d/s) Erith, approximately 8.5 km downstream of the River Roding mouth. Figure 2.1 shows the location of the Thames tide gauges.

For the following tide gauges, water level maxima (one for each high water, two per day) were available (short missing periods are ignored):

Table 6.1 *Available tide maxima data in the Thames estuary*

Gauge Name	Period of record
Southend	1 1 1939 - 31 12 1985
Gallions	1 1 1975 - 31 12 1985
Tower Pier	1 1 1939 - 31 12 1985
Richmond	1 1 1939 - 31 12 1985

Half hourly tide levels, from which maximum water levels can be extracted, were available as given in Table 6.2 (ignoring short missing periods).

Table 6.2 *Available half hourly tide data in the Thames estuary*

Gauge Name	Period of record
Southend	26 5 1988 - 31 12 1991
Sheerness	28 8 1987 - 31 12 1991
Erith	1 4 1987 - 30 4 1991
Silvertown	1 9 1987 - 30 4 1991
Westminster	1 10 1987 - 30 4 1990

Table 6.2 shows that the Sheerness record begins before the Southend record. In order to extend the Southend record to the same duration, a correlation analysis was performed between Southend levels and Sheerness levels on data for the overlapping period. The use of all data resulted in slight underestimation of levels above 2.5 mAOD, and a model using just the higher levels (above 2.5 mAOD) was derived with the-following result:

$$\text{Southend} = 0.146 + 0.943 * \text{Sheerness} \ (R^2 = 0.92) \tag{6.1}$$

No direct measurements have been made of water levels at the mouth of the River Roding. However, in an earlier study (Tidal Thames Defence Levels, Final Report, 1988) a hydraulic model of the Thames from Kingston to Southend had been developed. This study produced a series of structure functions, which give water levels at a series of cross section in the tidal Thames, including the mouth of the Roding, as a function of water level at Southend level and river flow at Teddington/Kingston. Two structure functions exist for each cross section: one with open Thames Barrier and one with closed Thames Barrier. For application of the structure functions, daily mean flow data were available for Teddington/Kingston, from 1883 to the present. Also available was an indication of whether the Thames Barrier was open or closed during each tidal cycle.

The structure functions may be used directly to calculate the Roding mouth level from Southend tide level and flow at Teddington/Kingston, or they may be used to derive the difference between a known level, for example at Erith, and the Roding mouth. This difference is then applied to the observed Erith level to determine the Roding mouth level. On the advice of the NRA Thames Hydraulic Modelling Section the latter procedure was preferred to the direct calculation from Southend levels and Kingston/Teddington flows because of the limited accuracy of the structure functions. The structure functions are given in Annex A.

For the recently installed tide gauges Silvertown and Westminster no structure functions were available. In order to be able to use these data, the difference between the structure functions at the nearest two sites were interpolated, using relative distances as weighting factors, to obtain a structure function of differences with the Roding mouth and with Tower Pier respectively. The structure functions that were derived in this way are presented in Annex B Tables A1 to A4.

The maximum level reached at Southend over the period 1950-1985 was 4.6 m (with a Thames flow of 72 m^3s^{-1}); this is equivalent to around 5.3-5.4 m at the Roding mouth.

7. Operation of the Barking barrier

The Barking barrier is sited at the mouth of the Barking creek. It was designed on tidal surge levels with a 1000 year return period in the year 2030 AD and came into operation in 1982. The barrier comprises of three vertical drop gates. The central gate is parked high to allow the passage of shipping, whereas the two side gates are parked just above normal high water level.

The Barking barrier is closed when the Thames barrier is due to be closed. The Thames barrier is closed if the controller believes that, without closure, the water level at London Bridge will reach 4.87 mAODN. The assessment is made on forecast levels at Southend produced by the Storm Tide Warning Service (STWS) and flows at Teddington/Kingston as given in Table 7.1.

A list of actual closures of the Thames barrier was examined. The barrier had been closed routinely at low water for testing and on 10 occasions for flood protection purposes. Only during one closure would the level at London Bridge have exceeded 4.87 m. On the other occasions the actual level would not have reached 4.87 m.

Table 7.1 Rules for closing Thames and Barking barriers

Thames flow ($m^3 s^{-1}$)		Southend level (mAODN)	
mgd		Action level	Closure level
1000	52.6	3.55	3.85
2000	105.2	3.50	3.80
3000	157.8	3.50	3.80
4000	210.4	3.45	3.75
5000	263.0	3.40	3.70
6000	315.6	3.35	3.65
7000	368.2	3.30	3.60
8000	420.8	3.25	3.55
9000	473.4	3.15	3.45
10000	526.0	3.05	3.35
11000	578.6	2.95	3.25
12000	631.2	2.85	3.15

Once the decision to close the Thames barrier has been made staff at the Barking barrier are alerted. The main gate of the barrier weighs 300 tonnes and takes 45 minutes to close ie. 35 minutes for the central gate to fall from the 'parked' position to the water level and a further 10 minutes to reach the river bed. When closed the barrier precludes all water.

Ideally the barrier should be closed at a time which maximises the storage capacity behind the barrier for fluvial flows from upstream. Overall, the optimum time for closure would be when water begins to flow from the Thames into the Roding mouth. During low flows from the Roding this will be soon after low tide. However, during high fluvial flows it would be beneficial to delay closure and allow water to flow from the Roding. Modelling of the effects of closure timing on storage capacity behind the Barking barrier has been undertaken by NRA Thames Region. Preliminary results suggest that, in general, the best time is about two hours after low tide.

It was recommended to NRA that further modelling of the effects on storage upstream of when in the tidal cycle the Barking barrier is closed should be undertaken to clarify the optimum time of closure. The results should be included in the operational procedures for the barrier.

In practice, the Barking barrier has been closed around low tide, ie. about six hours before high water. The Thames Barrier closes four hours before high water, thus if the STWS steps down its warning within two hours of the Barking barrier closing, the Thames barrier may not close (although this has not happened in practice). In contrast, the Barking barrier will always be closed if the Thames barrier is closed.

The Barking barrier can only be opened when water levels on either side are equal. Normally this would be on the next low tide after closure (ie 12 hours later), since surges do not last for two tidal cycles, but the exact timing will depend on the level of water which has built upstream behind the barrier.

It has been found unnecessary to close the barrier during normal tides but only if a high water level is enhanced by a surge.

The barrier operation rules have important consequences for the stage frequency relationship. At the downstream end of the study reach water levels are controlled by levels in the tidal Thames up to a level where the Barking barrier will close (about 4.4 - 4.7 m depending on the combination of tidal level and Thames flow) above which water levels in the lower Roding will be controlled by fluvial flows ponding behind the barrier. Flows will be augmented by discharges from Beckton STW when the level exceeds 4.4 m. The hydraulic model will need to be run continually for an entire tidal cycle with constant inflow to see if, following a barrier closure, significantly high water levels result.

It is clear that closure of the barriers relies on the judgement of the operations manager. To determine whether, given perfect forecasts of water levels, the barrier would have been closed during historical events, had it been built, Table 7.1 can be used. However, in practice the barriers are closed more often. To investigate the implications of this a sensitivity analysis was undertaken. This involved repeating the water level/frequency analysis on the River Roding assuming that the barriers close at a lower level in this case when the level at London Bridge reached 4.76 m AOD (rather than 4.87 m). This sensitivity analysis is described in section 13.

8. The hydraulic model of the River Roding

A study was initiated in 1988 to investigate the hydraulic performance of lower Roding from Redbridge to its confluence with the Thames in order to determine flood defence levels, to examine the effectiveness of the Barking barrier operation and to investigate potential improvements in flow control. The model used was ONDA which was developed by Sir William Halcrow and Partners and the study was undertaken by the NRA Thames Region Hydraulic modelling group. ONDA is a one dimensional model which uses the St Venant flow equations to relate stage and discharge at nodes about 200 m apart along the study reach.

Output from the model takes the form of a series of structure functions each of which indicate the water level that will result at a particular cross section on the river resulting from any combination of fluvial flow, effluent discharge and tidal level. Since it can be assumed that fluvial flows are constant over a tidal cycle, the relative timing of tidal cycle and flood hydrograph can be ignored.

The ONDA model was used to define the following sets of structure functions:

1. When the Barking barrier is closed:

relating water level to:

(a) fluvial flows from Redbridge over the range 1 to 64 m^3s^{-1} (the highest daily mean flow adjusted by Equation 4.2);

(b) discharges from Beckton STW works over the range 0 to 25 m^3s^{-1} (the highest recorded daily mean discharge), added for a time period for which the Thames water level exceeds 4.4 m.

(c) tidal levels over the range 4.0 to 5.5 m (from slightly below the level at which the barrier will be closed up to a level slightly above the maximum recorded between 1950 and 1985).

A further assumption made is that low water in the Thames (when the Barking is closed) is always sufficiently low that its precise level does not influence the initial storage available behind the barrier.

2. When the Barking barrier is open:

relating water level to:

(a) fluvial flows from Redbridge over the range 1 to 64 m^3s^{-1} (the highest daily mean flow adjusted by Equation 4.2);

and

(b) tidal levels over the range 0 to 5.0 m (slightly above the level at which the barrier will be closed).

When the barrier is open effluent from Beckton STW is discharged directly to the Thames.

All model runs were undertaken by NRA and structure functions were provided for the 10 locations on the River Roding as given in Table 8.1 and Figure 8.1. The structure functions are given in Annex B.

The model was calibrated using surveyed cross-sections of the river. Each cross section covers the main river channel and either bank, stopping at the bank top. Thus no information is provided on the geometry of the flood plain beyond the bank top, except for a flood defence level. In the model, therefore, the bank was assumed to rise vertically to the flood defence level (if greater than the surveyed level). The bank top was treated as a weir allowing spillage on to the flood plain once the water had exceeded the bank top level. Furthermore, due to lack of data to the contrary, the flood plain was assumed to have an infinite storage capacity, thus water levels are unable to exceed significantly the bank top level. Clearly, at some locations there are likely to be retaining walls or other structures on the flood plain which would limit its storage capacity leading to significant increases in water level once the storage has been filled. In this way water levels may be underestimated by the model.

Table 8.1 *Cross sections selected for detailed analysis*

Reference number	Location
1001u	upstream of Barking barrier
1006	Longreach wharf
1012	A13 Alfred Way bridge
1017d	downstream of Fourgates
1017u	upstream of Fourgates
1023b	Gurney Close rail bridge
1031	Ilford High Road bridge
1041	Wanstead park footbridge
1048	downstream Redbridge

It was recommended to NRA that flood plain storage should be evaluated to permit more confident extrapolation of water levels beyond bank top height.

For the calibration events, the hydraulic model predicted the observed peak level to within 10-20 mm. It is assumed that the peak levels for various combinations of river flow, estuary level and effluent discharge in the structure functions are accurate to within about 50 mm.

In all the above model runs it was assumed, as indicated in section 4, that the flow from the River Roding would be constant throughout any day. The sensitivity to the model results was tested by undertaking further runs using flows distributed through the day according to observed hydrographs. Three sets of runs were made with the peak flow early in the day, at mid-day and late in the day to investigate the influence of the relative timing of the hydrograph and tide graph (Thames water level). In each case the tide graph peaked at 3.0 m and the flow hydrograph peaked 35 m^3s^{-1}. The differences in maximum levels are given in Table 8.2. It is clear that for most

Figure 8.1 *Location of critical cross-sections*

Table 8.2 *Difference in level (mm) from using a constant flow in the Roding and from a variable flow peaking at different times of the day*

Section	Constant flow	Early peak	Mid-day peak	Late peak
1001u	2.997	-0.007	-0.007	-0.007
1006	2.994	0.000	0.000	0.000
1012	2.999	0.000	0.000	0.000
1017d	3.004	0.000	0.000	0.000
1017u	3.322	-0.083	-0.041	0.000
1023b	3.648	-0.155	-0.085	-0.007
1031	4.228	-0.117	-0.117	-0.009
1041	5.888	-0.009	-0.009	-0.007
1048	6.824	-0.006	-0.006	-0.006

sections the use of a hydrograph makes little or no difference, whereas for 1023b and 1031 the levels are slightly lower. It was concluded that, ideally, flow hydrographs should be used but this would increase the complexity of the modelling many times. Using a constant flow can be viewed as providing an extra margin of safety since it produces slightly higher levels at the two cross sections. Hence, this was accepted as providing an adequate representation of flow for the analysis.

9. The nightmare scenario

As indicated in section 7, once it has been closed the Barking barrier can only be opened when water levels on either side are equal. It has been anticipated that the water level on the Thames side of the barrier would always be higher, thus normally the barrier would be opened on the next low tide after closure (ie 12 hours later).

However, during the runs of the hydraulic model it was found that if high flows occur in the Roding at a time when the barrier is closed, the water level on the Roding side can exceed that on the Thames side. The structure functions in Annex B, Table B1, show that for most cases of a flow of 65 m^3s^{-1} when the barrier is closed, the peak level on the Roding side will exceed the peak level on the Thames side.

An example is shown in Figure 9.1 for a flow of 65 m^3s^{-1} from the Roding and a Thames level of 5.0 m AOD. Under current barrier operating procedures the water level upstream of the barrier would keep rising. This is because there is no outlet for flood water from the Roding to the Thames below the defence levels, once the barrier is closed. Since the flood hydrographs usually have a duration of 20-30 hours water levels may continue rising for many hours inundating areas adjacent to the Roding

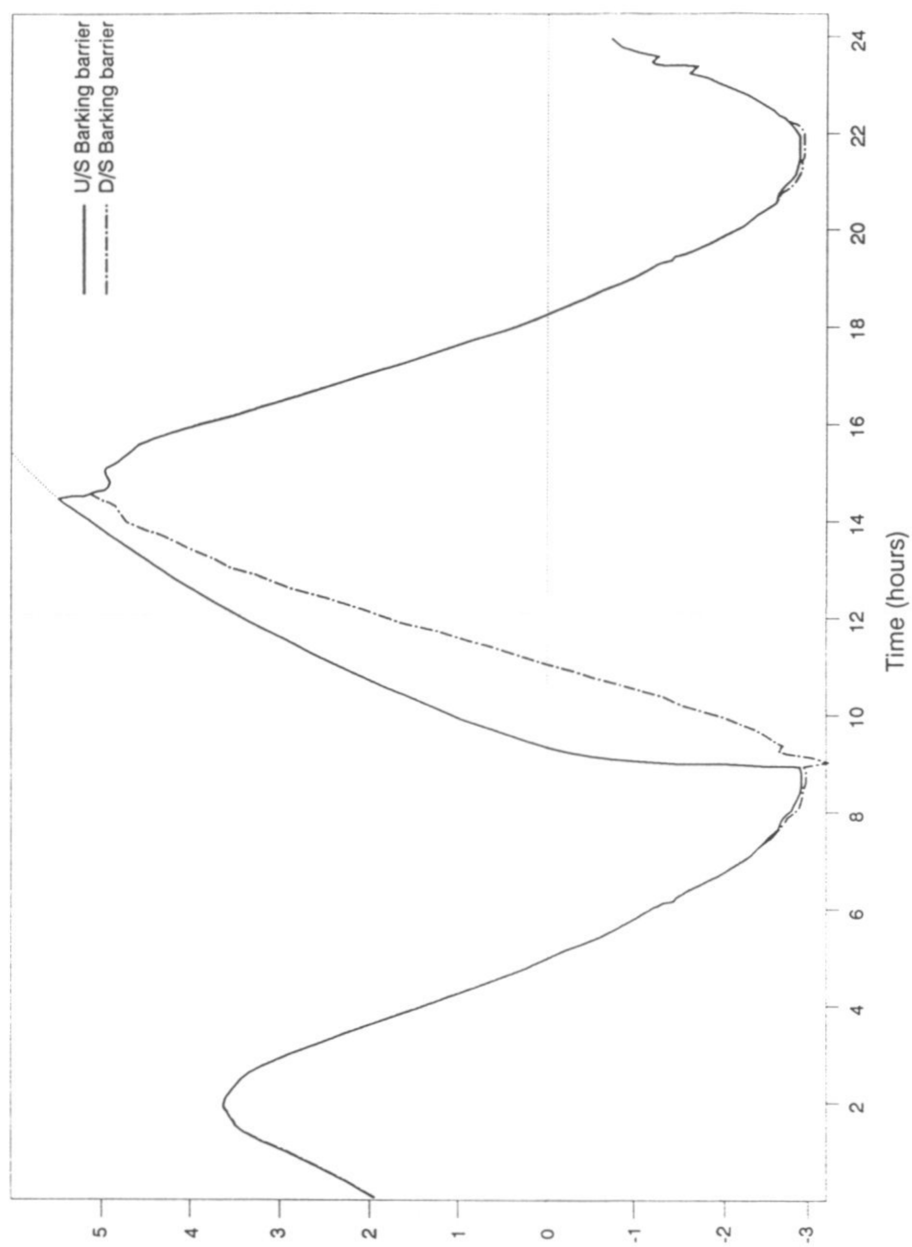

Figure 9.1 *Nightmare scenario example*

upstream of the barrier. In the model the barrier is permitted to open when the water levels on either side are not equal, as indicated in Figure 9.1. This is clearly the critical situation for major flooding in the lower Roding.

At Redbridge a flow of 65 m^3s^{-1} has a return period of around 50 years, however the probability of a flow of this magnitude or greater occurring when the barrier is closed is not known.

This problem is hinted at in the current operating procedures for the Barking Barrier. An inflow/storage curve is provided which indicates the length of time that it would take to fill the channel behind the barrier given the flow in the Roding and the current water level. The procedures state that, if this time is " .. less than the time to high water, advice the controller." It is not clear what action would be taken.

It was recommended to NRA that the Barking barrier operating rules should be reviewed to consider appropriate actions for events where the barrier would be closed and flows in the Roding exceed about 50 m^3s^{-1}. In these cases closing the barrier may increase the flood risk.

10. Historical reconstruction of annual maximum water levels

To undertake the water level/frequency analysis on the River Roding, for the historical period 1950-1991, daily maximum water levels were reconstructed for each critical cross section using the structure functions described in section 8. To apply these functions the following four data series were derived:

1. Water levels in the Thames estuary at the mouth of the Roding were derived for each high tide using the daily mean flows at Teddington/Kingston from the Surface Water Archive (1883-1991), data for tide gauges in the Thames estuary supplied by Thames Water and structure functions derived as part of the 1988 tidal Thames study.

2. Dates on which the Barking barrier was closed, or would have been closed (had it been build). It was assumed that the Barking barrier will have closed if the Thames barrier was (or would have been) closed, hence any effect on Thames levels effected by closure of the Thames barrier can be ignored.

3. Daily mean flows on the Roding at Redbridge. These data were provided by the Surface Water Archive (1950-1991). Mean flows were adjusted to estimate the peak flow by using Equation 4.2.

4. Daily mean effluent discharges into the Roding from Beckton STW. These data were generated by random sampling from a normal distribution fitted to the available data as described in section 5.

For each day of period 1950-1991 the four data series were accessed.

If the Braking barrier was closed then:

water levels at each critical cross section in the Roding were derived by applying the flow at Redbridge, the level at the Roding mouth and the discharge from Beckton to the appropriate structure function. The discharge was assumed to be zero if the level of the Thames is less than 4.4 m AODN and positive if the Thames is equal to or above that level.

If the Barking barrier was open then:

water levels in the Roding were derived by applying the flow at Redbridge and levels at the mouth of the Roding to the appropriate structure function.

Annex F gives a list of occasions when the barriers would have been closed (according to the model).

From the series of daily water levels, the annual maximum water levels were extracted for each cross section. These are given in Annex C. Without exception the annual maximum levels occurred as a combination of high levels in the Thames and/or high flows in the Roding *while the Barking Barrier was open*. This was because in the series of available data 1950-1991, the combination of high flows in the Roding and Barking barrier closure did not occur. However, if the barrier operating rules are revised this situation may never happen (see section 11).

To derive water levels of various return periods, extreme value analysis of the annual maximum levels was undertaken.

11. Statistical analysis of annual maxima

A GEV distribution was fitted by PWM separately to the annual maximum water levels at each chosen cross section within the River Roding. The resulting parameters are given in Table 11.1.

It is noteworthy that the value of k is positive in each case This denotes a downward curvature of the frequency curve, ie the slope of the line decreases with increasing return period and implies and upper bound on water level This is realistic since the channel width will increase with stage and becomes effectively infinite when the flood defences are exceeded, hence the water level cannot the flood defences to any great depth.

Table 11.1 *GEV parameters for each cross-section*

Section	Max	Mean	u	Alpha	K
1001u	4.76	4.41	4.352	0.156	0.208
1006	4.78	4.42	4.355	0.160	0.207
1012	4.80	4.44	4.370	0.163	0.205
1017d	4.82	4.45	4.386	0.164	0.205
1017u	4.80	4.45	4.390	0.158	0.224
1023bu	4.80	4.47	4.413	0.149	0.221
1031u	5.09	4.57	4.506	0.147	0.176
1041u	6.9	5.5	5.359	0.302	0.086
1048	7.7	6.4	6.256	0.326	0.126

12. Water levels of various return periods

Annual maximum water levels of return periods 2 to 1000 years are given for each cross section are given in Table 12.1. The standard error of each estimate is given in brackets below. An example level/frequency curve is shown in Figure 12.1.

In the lower parts of the study reach the level/frequency curve is very shallow with only a small range in level, 0.5 m, between the 2 and 1000 (the standard error ranges from around 30 to 150 mm), thus a small change in level relates to a large change in return period. The consequence of this is that any small error in the model translates to a large error in return period.

Estimates beyond the 100 year level are based purely on extrapolation of the data shown. Under the present Barking barrier operating rules a discontinuity would be expected in level/frequency curve when barrier close coincides with high flow (the 'night-mare scenario). As indicated above this did not happen in the historical data, so the return periods of such events can not be determined under the current scheme.

Clearly the results rely on the assumptions made in constructing the hydraulic model, particularly the assumption of infinite flood plain storage. This increases the uncertainty of the estimates above the bank top levels.

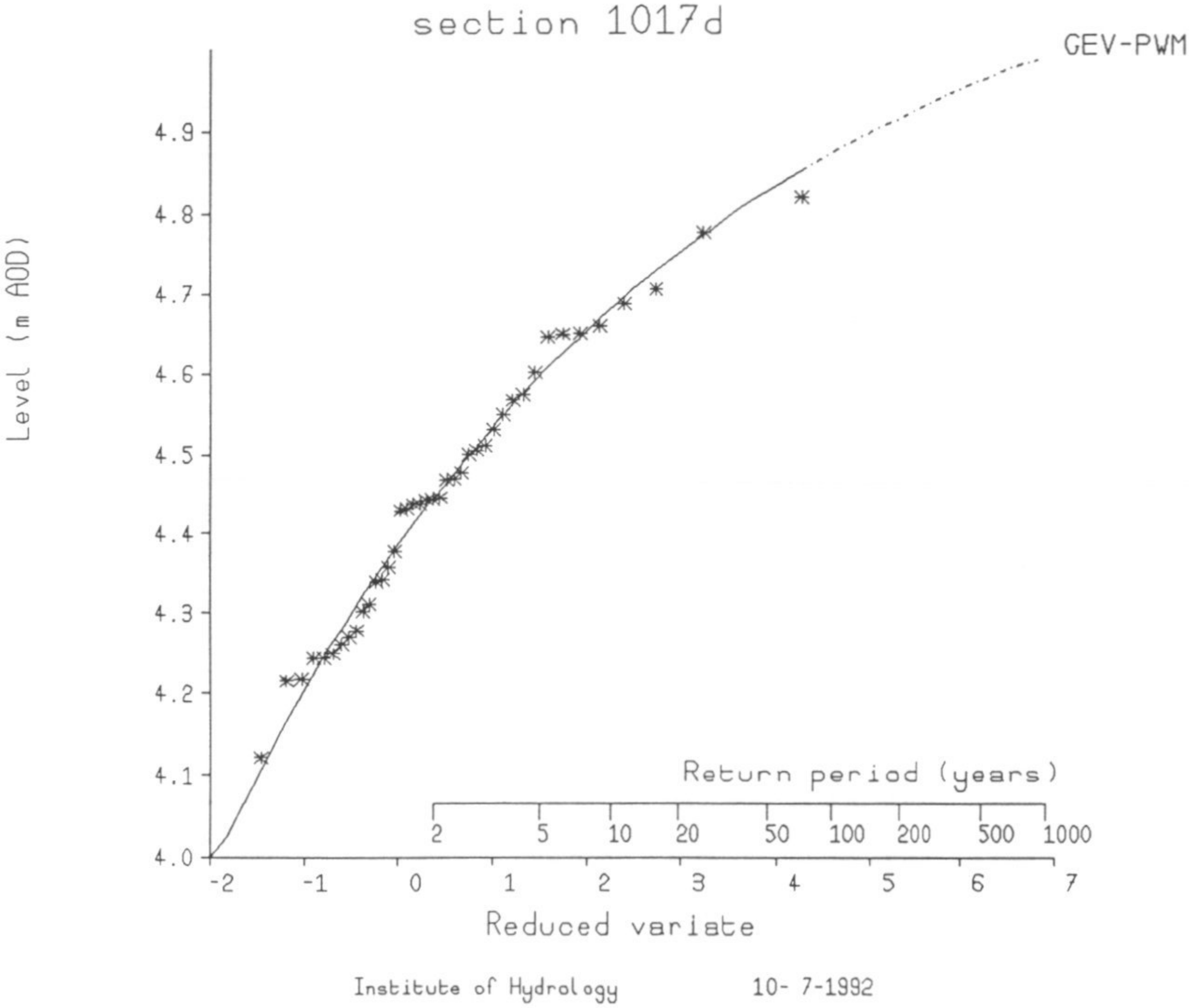

Figure 12.1 *Example level/frequency curve for section 1017d.*

Table 12.1 *Water levels of various return periods for the nine critical cross-sections on the River Roding (standard errors are given in brackets)*

	Return period (years)							
Section	2	5	10	50	100	200	500	1000
1001u	4.41 (0.03)	4.55 (0.03)	4.63 (0.04)	4.77 (0.07)	4.81 (0.08)	4.85 (0.10)	4.89 (0.12)	4.92 (0.14)
1006	4.41 (0.03)	4.56 (0.03)	4.64 (0.04)	4.78 (0.07)	4.83 (0.08)	4.87 (0.10)	4.91 (0.12)	4.94 (0.14)
1012	4.43 (0.03)	4.58 (0.03)	4.66 (0.04)	4.81 (0.07)	4.86 (0.09)	4.90 (0.10)	4.94 (0.13)	4.97 (0.15)
1017d	4.44 (0.03)	4.60 (0.03)	4.68 (0.04)	4.83 (0.07)	4.87 (0.09)	4.92 (0.10)	4.96 (0.11)	4.99 (0.12)
1017u	4.45 (0.03)	4.59 (0.03)	4.67 (0.04)	4.80 (0.06)	4.84 (0.08)	4.88 (0.09)	4.92 (0.11)	4.94 (0.13)
1023b	4.47 (0.03)	4.60 (0.03)	4.68 (0.04)	4.80 (0.06)	4.84 (0.07)	4.88 (0.09)	4.92 (0.11)	4.94 (0.13)
1031u	4.56 (0.03)	4.70 (0.03)	4.78 (0.04)	4.92 (0.07)	4.97 (0.09)	5.01 (0.10)	5.06 (0.13)	5.09 (0.15)
1041u	5.47 (0.06)	5.78 (0.08)	5.98 (0.10)	6.36 (0.19)	6.51 (0.25)	6.64 (0.31)	6.81 (0.41)	6.93 (0.49)
1048	6.37 (0.06)	6.70 (0.08)	6.89 (0.10)	7.26 (0.18)	7.39 (0.23)	7.52 (0.29)	7.66 (0.37)	7.76 (0.43)

13. Model runs with the barrage in place

A second version of the ONDA hydraulic model described in section 8 was constructed by NRA incorporating the proposed barrage. In a repeat set of model runs, the same combinations of fluvial flow, effluent discharge and tidal level were input to the model. This produced a second set of structure functions indicating water level at critical cross-sections along the Roding for specified fluvial flows, effluent discharges and tidal levels with and without the Barking barrier closed.

The structure functions produced are given in Annex D.

Next the historical reconstruction of daily maximum water levels as detailed in section

10 was repeated using the new structure functions. From this series the annual maximum water levels were extracted for each critical cross section. These are given in Annex E.

As in the 'without barrage' case, with the barrage in the model, the annual maximum levels occurred as a combination of high levels in the Thames and/or high flows in the Roding while the Barking Barrier was open.

A GEV distribution was fitted by PWM separately to the annual maximum water levels at each chosen cross section within the Roding. The resulting water levels of return periods 2 to 1000 years are given in Table 13.1. For comparison the levels for the 'without barrage' case from Table 12.1 are given in brackets.

Table 13.1 *Predicted water levels (m AOD) of various return periods with the amenity barrage in place (equivalent levels for the current, 'without barrage' case are given in brackets).*

	Return period (years)							
Section	2	5	10	50	100	200	500	1000
1001u	4.41 (4.41)	4.56 (4.55)	4.64 (4.63)	4.78 (4.77)	4.82 (4.81)	4.86 (4.85)	4.90 (4.89)	4.93 (4.92)
1006	4.42 (4.41)	4.57 (4.56)	4.65 (4.64)	4.79 (4.78)	4.84 (4.83)	4.88 (4.87)	4.92 (4.91)	4.95 (4.94)
1012	4.43 (4.43)	4.58 (4.58)	4.67 (4.66)	4.81 (4.81)	4.86 (4.86)	4.90 (4.90)	4.95 (4.94)	4.98 (4.97)
1017d	4.45 (4.44)	4.60 (4.60)	4.68 (4.68)	4.83 (4.83)	4.87 (4.87)	4.91 (4.92)	4.96 (4.96)	4.99 (4.99)
1017u	4.45 (4.45)	4.59 (4.59)	4.67 (4.67)	4.80 (4.80)	4.84 (4.84)	4.87 (4.88)	4.91 (4.92)	4.94 (4.94)
1023b	4.49 (4.47)	4.63 (4.60)	4.70 (4.68)	4.82 (4.80)	4.86 (4.84)	4.89 (4.88)	4.93 (4.92)	4.95 (4.94)
1031u	4.57 (4.56)	4.71 (4.70)	4.79 (4.78)	4.94 (4.92)	4.99 (4.97)	5.04 (5.01)	5.09 (5.06)	5.13 (5.09)
1041u	5.47 (5.47)	5.79 (5.78)	5.98 (5.98)	6.37 (6.36)	6.52 (6.51)	6.66 (6.64)	6.84 (6.81)	6.96 (6.93)
1048	6.37 (6.37)	6.70 (6.70)	6.90 (6.89)	7.27 (7.26)	7.40 (7.39)	7.52 (7.52)	7.67 (7.66)	7.77 (7.76)

Figure 13.2 gives the differences in water level predicted to result from building the barrage. The maximum being 0.04 m for floods of 1000 year return period at cross section 1031u, which is upstream of the barrage site.

Table 13.2 *Predicted differences in water levels (m AOD) of various return periods with the amenity barrage in place compared with the 'without barrage' case.*

	Return period (years)							
Section	2	5	10	50	100	200	500	1000
1001u	0.00	0.01	0.01	0.01	0.01	0.01	0.01	0.01
1006	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01
1012	0.00	0.00	0.01	0.00	0.00	0.00	0.01	0.01
1017d	0.01	0.00	0.00	0.00	0.00	-0.01	0.00	0.00
1017u	0.00	0.00	0.00	0.00	0.00	-0.01	-0.01	0.00
1023b	0.02	0.03	0.02	0.02	0.02	0.01	0.01	0.01
1031u	0.01	0.01	0.01	0.02	0.02	0.03	0.03	0.04
1041u	0.00	0.01	0.00	0.01	0.01	0.02	0.03	0.03
1048	0.00	0.00	-0.01	0.01	0.01	0.00	0.01	0.01

14. Flood defence protection levels

Table 14.1 gives, for each cross-section, the right and left bank levels which are assumed to correspond to the flood defence levels. Also given is the return period of water reaching this level for both the 'with' and 'without' barrage cases. It can be seen that for sections 1001u to 1031 the flood defences give a level of protection greater than 1000 years return period. Since the 1000 year water levels are below the bank top the capacity of plain storage is not a problem. This means that extrapolation to this return period is not affected by any potential discontinuity in water levels at bank full.

It can be concluded from this that, at cross sections 1001 to 1031, the barrage does not increase the flood risk significantly.

Table 14.1 *Return periods of floods reaching defence levels at critical cross sections, without and with the barrage in place*

Ref No.	West bank Defence level (m AOD)	Return period without barrage (years)	Return period with barrage (years)	East bank Defence level (m AOD)	Return period without barrage (years)	Return period with barrage (years)
1001u	7.18	> 1000	> 1000	5.68	> 1000	> 1000
1006	5.37	> 1000	> 1000	5.79	> 1000	> 1000
1012	9.30	> 1000	> 1000	5.5	> 1000	> 1000
1017d	5.50	> 1000	> 1000	5.11	> 1000	> 1000
1017u	5.81	> 1000	> 1000	5.55	> 1000	> 1000
1023b	5.09	> 1000	> 1000	5.22	> 1000	> 1000
1031	6.40	> 1000	> 1000	5.80	> 1000	> 1000
1041	6.70	270	240	6.84	590	500
1048	7.45	140	130	7.11	25	24

At section 1041 the level of protection on the west bank appears to be reduced from 270 to 240 years and on the east bank from 590 to 500 years. At cross-section 1048 the reduction in protection level is even less: west bank 140 reduced to 130 years and east bank 25-24 years. All these changes are less than the accuracy of the modelling and are therefore not significant.

15. Summary and conclusions

The London Borough of Barking and Dagenham proposed the construction of an amenity barrage across the River Roding at Four Gates to maintain water levels at low tide. The Institute of Hydrology was commissioned to assess the impact on flood risk in the lower Roding of this barrage.

The Institute had just completed an analysis of flood levels along the lower Roding for the National Rivers Authority as part of its review of flood defences in the Thames estuary. The models developed for that study were modified to determine the impact of the barrage.

Water levels in the Roding are controlled by the interaction of Thames tidal levels and flows from the catchment upstream. The Barking barrier spans the Roding at its mouth and when closed precludes all water from the Thames, thus removing the tidal influence. Beckton Sewage Treatment Works normally discharges its effluent directly into the Thames. However when the Thames level reaches 4.4 m (at which level the Barking barrier will be closed) the effluent is discharged to the Roding.

Water levels in the lower River Roding are therefore determined by:

(1) fluvial flows from the catchment upstream of Redbridge;
(2) whether or not the Barking barrier is closed.
(3) water levels in the River Thames at the mouth of the Roding; and
(4) effluent discharges from Beckton STW;

Two versions of a hydraulic model were constructed to provide structure functions relating water levels along the study reach to given fluvial flow, effluent discharges and water levels at the mouth. One version modelled the current configuration of the channel. The second model included the proposed barrage.

Daily mean flow data (1950-1991) from Redbridge provide an adequate time series for the analysis when adjusted to estimate the peak flow on the day. Insufficient data on effluent discharges from Beckton STW were available, so data for the period 1950-1991 was generated synthetically. Records of water level at various locations in the Thames estuary are available for the period 1950-1991. These were used to derive a series of water levels at the mouth of the Roding.

For the historical period 1950-1991, the structure functions were used in association with recorded Roding flows, the barrier status, the Thames/Roding mouth levels and synthetic Beckton effluent discharges to produce two series of annual maximum water levels at nine critical cross-sections, one for the without barrage case and one with the barrage in place.

For each cross section statistical distributions were fitted to the annual maximum water levels to estimate return periods of 2 to 1000. This required a large extrapolation of the available data. The difference between the results indicated the impact on water levels of the barrage.

The models predicted an increase in flood levels at most cross-sections with the barrage in place, particularly at high (> 50 years) return periods. However, the maximum increase was only 40 mm and for sections 1001u to 1031 the flood defences still gave a level of protection greater than 1000 years return period. For the other two sections (1041 and 1048) the reduction in protection level was small and within the error bounds of the analysis. It was concluded that the barrage had no significant influence on flood risk in the lower Roding.

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Annex A Structure functions for the Thames estuary

Table A1.1 Tower Pier structure function (without Barrier)

Southend level (m AOD)	2.00	2.50	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)						
50	2.76	3.39	4.00	4.53	5.06	5.57
100	2.79	3.42	4.03	4.56	5.09	5.59
200	2.84	3.48	4.09	4.63	5.15	5.64
300	2.87	3.52	4.14	4.67	5.18	5.66
400	2.89	3.56	4.19	4.71	5.22	5.70
600	2.90	3.62	4.27	4.78	5.30	5.76
800	2.91	3.64	4.33	4.83	5.30	5.76
1000	2.91	3.65	4.36	4.84	5.31	5.77

Table A1.2 Structure function difference Tower Pier - Westminster (without barrier)

Southend level (m AOD)	2.00	2.50	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)						
50	0.04	0.03	0.01	0.03	0.03	0.03
100	0.05	0.05	0.04	0.04	0.04	0.04
200	0.07	0.07	0.06	0.06	0.05	0.06
300	0.06	0.08	0.07	0.07	0.07	0.05
400	0.06	0.08	0.08	0.09	0.06	0.05
600	0.09	0.09	0.12	0.09	0.06	0.04
800	0.07	0.08	0.10	0.08	0.06	0.06
1000	2.91	0.08	0.10	0.08	0.06	0.06

Table A2.1 Roding mouth structure function (without barrier)

Southend level (m AOD)	2.00	2.50	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)						
50	2.57	3.17	3.76	4.29	4.84	5.33
100	2.58	3.18	3.77	4.30	4.82	5.33
200	2.60	3.20	3.79	4.32	4.84	5.35
300	2.61	3.22	3.81	4.34	4.86	5.37
400	2.63	3.24	3.83	4.36	4.88	5.39
600	2.65	3.27	3.86	4.40	4.92	5.43
800	2.64	3.28	3.89	4.43	4.95	5.46
1000	2.65	3.30	3.91	4.41	4.97	5.48

Table A2.2 Roding mouth structure function (with barrier)

Southend level (m AOD)	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)				
50	4.01	4.42	4.84	5.29
100	4.01	4.42	4.83	5.29
200	4.01	4.42	4.83	5.28
300	4.01	4.43	4.82	5.27
400	4.00	4.43	4.81	5.27
600	4.00	4.42	4.80	5.26
800	3.99	4.40	4.80	5.25
1000	3.98	4.40	4.80	5.25

Table A2.3 Structure function difference Roding mouth - Gallions (without barrier)

Southend level (m AOD)	2.00	2.50	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)						
50	-0.01	-0.02	-0.02	-0.02	-0.01	-0.01
100	-0.01	-0.02	-0.02	-0.02	-0.02	-0.02
200	-0.01	-0.02	-0.02	-0.02	-0.02	-0.02
300	-0.02	-0.02	-0.02	-0.02	-0.02	-0.02
400	-0.02	-0.02	-0.02	-0.02	-0.03	-0.02
600	-0.01	-0.02	-0.02	-0.03	-0.02	-0.02
800	-0.02	-0.02	-0.02	-0.02	-0.02	-0.02
1000	-0.01	-0.02	-0.02	-0.03	-0.03	-0.02

Table A2.4 Structure function difference Roding mouth - Silvertown (without barrier)

Southend level (m AOD)	2.00	2.50	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)						
50	-0.04	-0.06	-0.06	-0.06	-0.04	-0.04
100	-0.04	-0.06	-0.06	-0.06	-0.06	-0.06
200	-0.04	-0.06	-0.06	-0.06	-0.06	-0.06
300	-0.06	-0.06	-0.06	-0.06	-0.07	-0.07
400	-0.06	-0.06	-0.07	-0.08	-0.07	-0.07
600	-0.06	-0.06	-0.09	-0.07	-0.06	-0.06
800	-0.05	-0.07	-0.07	-0.07	-0.07	-0.06
1000	-0.04	-0.07	-0.10	-0.09	-0.07	-0.07

Table A2.5 Structure function difference Roding mouth - Erith (without barrier)

Southend level (m AOD)	2.00	2.50	3.00	3.50	4.00	4.50
Kingston flow ($m^3.s^{-1}$)						
50	0.11	0.14	0.14	0.15	0.14	0.13
100	0.12	0.14	0.17	0.15	0.16	0.17
200	0.12	0.15	0.17	0.16	0.15	0.14
300	0.12	0.15	0.18	0.17	0.15	0.14
400	0.12	0.15	0.18	0.17	0.16	0.15
600	0.13	0.16	0.19	0.18	0.17	0.15
800	0.12	0.16	0.20	0.18	0.17	0.14
1000	0.12	0.17	0.20	0.15	0.17	0.14

Annex B Structure functions for the study reach on the Roding without the barrage

Table B.1 Roding section 1001 U structure functions

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	1.997	1.997	1.994	1.996	1.994	1.995
3.00	2.992	2.995	2.990	2.989	2.989	2.990
4.00	3.994	3.997	3.994	3.991	3.995	3.994
5.00	5.005	5.005	5.001	5.003	5.005	5.005
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-1.375	-0.008	2.214	3.607	4.488	5.254
5.00	-1.141	0.113	2.336	3.717	4.736	5.290
5.50	-0.965	0.217	2.399	3.845	4.863	5.457
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-1.199	0.086	2.270	3.629	4.511	5.262
5.00	-0.805	0.298	2.462	3.826	4.779	5.335
5.50	-0.521	0.493	2.586	4.006	4.989	5.486
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.889	0.247	2.382	3.726	4.520	5.279
5.00	-0.279	0.660	2.661	3.980	4.884	5.424
5.50	0.154	0.991	2.901	4.243	5.110	5.517
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.627	0.401	2.491	3.822	4.514	5.296
5.00	0.151	0.990	2.856	4.179	4.912	5.511
5.50	0.718	1.437	3.202	4.472	5.402	5.600

Table B.2 Roding section 1006 structure functions

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	1.993	1.994	1.986	1.992	1.988	1.991
3.00	2.983	2.988	2.979	2.982	2.983	2.984
4.00	3.987	3.992	3.985	3.981	3.990	3.986
5.00	5.026	5.026	5.018	5.022	5.025	5.027
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-1.369	-0.008	2.246	3.641	4.488	5.252
5.00	-1.141	0.113	2.335	3.764	4.736	5.290
5.50	-0.965	0.227	2.434	3.845	4.887	5.478
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-1.194	0.086	2.270	3.682	4.522	5.260
5.00	-0.805	0.310	2.462	3.826	4.813	5.335
5.50	-0.521	0.500	2.586	4.006	4.989	5.500
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.889	0.247	2.382	3.726	4.542	5.275
5.00	-0.279	0.660	2.661	4.022	4.884	5.424
5.50	0.155	0.998	2.901	4.248	5.110	5.517
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.627	0.401	2.491	3.822	4.514	5.296
5.00	0.151	0.990	2.856	4.179	4.912	5.511
5.50	0.718	1.437	3.202	4.472	5.402	5.600

Table B.3 Roding section 1012 structure functions

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	1.998	1.994	1.995	1.990	2.008	2.017
3.00	3.008	3.003	3.005	3.011	3.013	3.014
4.00	3.999	3.997	3.991	4.000	3.983	3.980
5.00	5.057	5.057	5.050	5.050	5.056	5.058
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-1.234	0.009	2.261	3.651	4.529	5.250
5.00	-1.127	0.130	2.335	3.784	4.764	5.289
5.50	-0.956	0.236	2.445	3.879	4.936	5.503
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-1.171	0.086	2.206	3.698	4.525	5.256
5.00	-0.799	0.317	2.462	3.863	4.825	5.334
5.50	-0.516	0.510	2.586	4.004	4.988	5.504
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.884	0.261	2.381	3.742	4.570	5.267
5.00	-0.279	0.670	2.661	4.035	4.884	5.423
5.50	0.156	1.005	2.901	4.258	5.161	5.535
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.626	0.419	2.491	3.821	4.561	5.279
5.00	0.151	0.990	2.883	4.180	5.014	5.509
5.50	0.720	1.437	3.215	4.480	5.301	5.596

Table B.4 *Roding section 1017D structure functions*

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.004	2.008	2.012	2.031	2.069	2.119
3.00	3.019	3.011	3.013	3.023	3.032	3.044
4.00	4.015	4.012	4.005	4.018	3.996	4.001
5.00	5.074	5.076	5.060	5.065	5.072	5.075
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-0.428	0.085	2.264	3.651	4.559	5.243
5.00	-0.428	0.177	2.371	3.786	4.797	5.284
5.50	-0.428	0.273	2.447	3.902	5.949	5.512
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-0.428	0.152	2.317	3.700	4.551	5.247
5.00	-0.428	0.346	2.462	3.884	4.823	5.328
5.50	-0.419	0.523	2.615	4.000	5.037	5.503
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.428	0.294	2.413	3.773	4.581	5.255
5.00	-0.256	0.684	2.687	4.036	4.915	5.415
5.50	0.158	1.009	2.925	4.262	5.191	5.556
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.428	0.436	2.491	3.819	4.587	5.263
5.00	0.179	1.004	2.895	4.178	5.028	5.499
5.50	0.722	1.444	3.235	4.531	5.314	5.589

Table B.5 Roding section 1017 u structure functions

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.003	2.021	2.282	2.648	3.116	3.571
3.00	3.014	3.017	3.111	3.327	3.566	3.813
4.00	4.016	4.016	4.047	4.139	4.245	4.412
5.00	5.035	5.061	5.065	5.097	5.157	5.239
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	0.684	1.056	2.462	3.798	4.695	5.378
5.00	0.684	1.056	2.554	3.914	4.913	5.410
5.50	0.684	1.056	2.612	4.018	5.045	5.587
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	0.684	1.056	2.506	3.839	4.698	5.381
5.00	0.684	1.056	2.636	4.002	4.920	5.444
5.50	0.684	1.056	2.761	4.101	5.132	5.577
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	0.684	1.056	2.590	3.909	4.694	5.386
5.00	0.684	1.056	2.825	4.134	5.018	5.511
5.50	0.684	1.143	3.035	4.350	5.261	5.609
	1.056 n outflow = 30 $m^3.s^{-1}$					
4.00	0.684	1.056	2.658	3.957	4.707	5.391
5.00	0.684	1.141	3.003	4.275	5.093	5.574
5.50	0.754	1.498	3.311	4.579	5.362	5.631

Table B.6 Roding section 1023 BU structure functions

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.020	2.134	2.774	3.314	3.825	4.300
3.00	3.028	3.030	3.261	3.655	4.050	4.415
4.00	4.020	4.031	4.094	4.276	4.522	4.796
5.00	5.040	5.063	5.076	5.157	5.279	5.441
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	1.555	1.952	2.844	4.007	4.854	5.538
5.00	1.555	1.952	2.881	4.092	5.036	5.578
5.50	1.555	1.952	2.919	4.152	5.160	5.651
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	1.555	1.952	2.862	4.035	4.859	5.542
5.00	1.555	1.952	2.928	4.140	5.086	5.594
5.50	1.555	1.952	2.996	4.264	5.241	5.659
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	1.555	1.952	2.898	4.066	4.877	5.549
5.00	1.555	1.952	3.037	4.280	5.141	5.619
5.50	1.555	1.952	3.190	4.3464	5.342	5.666
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	1.555	1.952	2.941	4.128	4.877	5.556
5.00	1.555	1.952	3.171	4.406	5.206	5.638
5.50	1.555	1.952	3.410	4.653	5.452	5.674

Table B.7 Roding section 1031 U structure functions

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.200	2.577	3.448	4.128	4.646	5.092
3.00	3.063	3.092	3.646	4.230	4.725	5.136
4.00	4.025	4.074	4.230	4.575	4.941	5.318
5.00	5.078	5.120	5.132	5.376	5.510	5.802
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	2.191	2.567	3.470	4.397	5.160	5.834
5.00	2.191	2.567	3.485	4.446	5.306	5.845
5.50	2.191	2.567	3.501	4.496	5.405	5.880
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	2.191	2.567	3.477	4.412	5.167	5.835
5.00	2.191	2.567	3.505	4.488	5.327	5.849
5.50	2.191	2.567	3.536	4.557	5.474	5.882
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	2.191	2.567	3.493	4.439	5.172	5.836
5.00	2.191	2.567	3.550	4.564	5.385	5.861
5.50	2.191	2.567	3.611	4.696	5.575	5.883
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	2.191	2.567	3.511	4.474	5.172	5.837
5.00	2.191	2.567	3.601	4.654	5.430	5.869
5.50	2.191	2.567	3.732	4.846	5.654	5.886

Table B.8 Roding section 1041 U structure functions

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	4.306	4.359	5.156	5.884	6.401	6.910
3.00	4.310	4.359	5.215	5.888	6.409	6.915
4.00	4.313	4.359	5.219	5.905	6.434	6.946
5.00	5.119	5.156	5.529	6.035	6.560	7.067
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	4.313	4.359	5.156	5.894	6.468	7.067
5.00	4.313	4.359	5.156	5.896	6.496	7.078
5.50	4.313	4.359	5.156	5.899	6.519	7.094
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	4.313	4.359	5.156	5.894	6.469	7.067
5.00	4.313	4.359	5.156	5.898	6.501	7.078
5.50	4.313	4.359	5.163	5.902	6.535	7.095
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	4.313	4.359	5.156	5.896	6.470	7.068
5.00	4.313	4.359	5.182	5.903	6.513	7.079
5.50	4.313	4.359	5.222	5.913	6.565	7.096
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	4.313	4.359	5.156	5.897	6.471	7.068
5.00	4.313	4.359	5.222	5.910	6.525	7.082
5.50	4.313	4.359	5.222	5.930	6.592	7.096

Table B.9 Roding section 1048 structure functions

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	4.967	5.252	6.110	6.822	7.372	7.779
3.00	4.980	5.252	6.119	6.824	7.373	7.781
4.00	4.980	5.252	6.120	6.828	7.379	7.790
5.00	5.173	5.373	6.196	6.872	7.409	7.832
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	4.967	5.252	6.110	6.825	7.386	7.831
5.00	4.967	5.252	6.110	6.826	7.393	7.836
5.50	4.967	5.252	6.110	6.827	7.398	7.843
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	4.967	5.252	6.110	6.826	7.386	7.831
5.00	4.967	5.252	6.110	6.826	7.394	7.836
5.50	4.967	5.252	6.111	6.827	7.402	7.843
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	4.313	4.359	6.110	6.826	7.387	7.831
5.00	4.313	4.359	6.113	6.828	7.397	7.836
5.50	4.313	4.359	6.120	6.830	7.410	7.844
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	4.313	4.359	6.110	6.826	7.387	7.831
5.00	4.313	4.359	6.120	6.829	7.400	7.836
5.50	4.313	4.359	6.120	6.834	7.417	7.844

Annex C Annual maximum water levels without the barrage

	1001u	1006u	1012	1017d	1017u	1023bu	1031u	1041u	1048
1950	4.281	4.283	4.295	4.311	4.317	4.334	4.397	5.165	6.068
1951	4.656	4.667	4.690	4.707	4.693	4.699	4.743	5.520	6.434
1952	4.428	4.434	4.451	4.468	4.460	4.468	4.508	5.160	6.063
1953	4.602	4.612	4.633	4.651	4.641	4.647	4.694	5.415	6.325
1954	4.405	4.410	4.426	4.443	4.438	4.447	4.493	5.242	6.145
1955	4.531	4.539	4.558	4.575	4.572	4.583	4.638	5.157	6.064
1956	4.098	4.095	4.105	4.121	4.121	4.131	4.161	5.090	5.990
1957	4.464	4.470	4.489	4.506	4.494	4.501	4.534	5.108	6.008
1958	4.413	4.416	4.431	4.446	4.461	4.485	4.566	5.543	6.463
1959	4.244	4.242	4.255	4.270	4.318	4.374	4.521	5.678	6.594
1960	4.214	4.214	4.227	4.243	4.240	4.256	4.421	5.754	6.678
1961	4.723	4.736	4.761	4.778	4.758	4.763	4.802	5.224	6.127
1962	4.306	4.309	4.323	4.339	4.345	4.365	4.435	5.319	6.225
1963	4.215	4.215	4.228	4.243	4.241	4.325	4.549	5.691	6.589
1964	4.188	4.187	4.201	4.217	4.213	4.220	4.556	5.996	6.927
1965	4.399	4.404	4.420	4.437	4.429	4.437	4.475	5.530	6.442
1966	4.602	4.613	4.635	4.652	4.634	4.640	4.674	5.489	6.401
1967	4.598	4.608	4.630	4.647	4.631	4.637	4.673	5.454	6.267
1968	4.442	4.446	4.462	4.478	4.490	4.511	4.588	5.754	6.674
1969	4.392	4.397	4.415	4.431	4.420	4.427	4.455	5.556	6.484
1970	4.270	4.271	4.286	4.302	4.296	4.303	4.356	5.379	6.290
1971	4.344	4.347	4.361	4.377	4.381	4.396	4.456	5.325	6.234
1972	4.322	4.324	4.341	4.357	4.349	4.355	4.383	5.190	6.093
1973	4.763	4.778	4.803	4.821	4.799	4.804	4.844	4.942	5.611
1974	4.225	4.224	4.234	4.249	4.272	4.355	5.090	6.871	7.746
1975	4.467	4.471	4.487	4.501	4.524	4.552	4.647	5.549	6.471
1976	4.469	4.476	4.495	4.512	4.500	4.506	4.539	5.365	6.276
1977	4.556	4.565	4.586	4.603	4.589	4.595	4.631	5.683	6.569
1978	4.490	4.498	4.515	4.532	4.548	4.603	4.743	5.821	6.744
1979	4.397	4.398	4.414	4.429	4.468	4.530	4.719	5.721	6.587
1980	4.400	4.405	4.423	4.444	4.429	4.436	4.515	5.346	6.208
1981	4.505	4.513	4.533	4.550	4.536	4.542	4.576	5.530	6.443
1982	4.309	4.312	4.326	4.342	4.339	4.349	4.462	5.977	6.910
1983	4.525	4.533	4.551	4.568	4.566	4.577	4.633	5.440	6.349
1984	4.236	4.234	4.245	4.260	4.288	4.322	4.429	5.186	6.052
1985	4.430	4.436	4.453	4.470	4.462	4.471	4.590	5.589	6.459
1986	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
1987	4.188	4.187	4.199	4.215	4.241	4.340	4.609	5.857	6.769
1988	4.638	4.649	4.672	4.689	4.671	4.677	4.713	5.978	6.920
1989	4.399	4.404	4.422	4.438	4.427	4.433	4.475	5.490	6.409
1990	4.611	4.622	4.644	4.661	4.648	4.654	4.697	5.968	6.899
1991	4.249	4.250	4.262	4.278	4.276	4.287	4.328	4.818	5.720

Annex D Structure functions for the study reach on the Roding with the barrage

Table D.1 Roding section 1001 U structure functions (with barrage)

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	1.992	1.997	1.993	1.995	1.993	1.994
3.00	2.994	2.996	2.992	2.994	2.993	2.993
4.00	3.994	3.997	3.993	3.992	3.998	3.996
5.00	5.015	5.015	5.014	5.013	5.014	5.014
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-1.338	0.005	2.443	3.823	4.496	5.502
5.00	-1.117	0.136	2.532	4.003	4.902	5.531
5.50	-0.937	0.249	2.670	4.055	5.064	5.546
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-1.167	0.107	2.457	3.849	4.493	5.510
5.00	-0.780	0.338	2.688	4.039	4.894	5.560
5.50	-0.495	0.548	2.834	4.209	5.139	5.580
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.862	0.282	2.589	3.943	4.493	5.526
5.00	-0.238	0.729	2.914	4.201	4.989	5.599
5.50	0.227	1.091	3.153	4.441	5.241	5.608
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.602	0.448	2.725	4.035	4.495	5.540
5.00	0.227	1.073	3.107	4.385	4.996	5.617
5.50	0.837	1.588	3.444	4.610	5.326	5.621

Table D.2 Roding section 1006 structure functions (with barrage)

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	1.997	2.005	1.998	1.999	2.000	2.002
3.00	3.004	3.007	3.010	3.010	3.009	3.009
4.00	4.000	4.002	3.996	3.996	3.997	3.992
5.00	5.033	5.033	5.031	5.029	5.031	5.031
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-1.337	0.005	2.493	3.858	4.516	5.508
5.00	-1.112	0.136	2.534	4.003	4.902	5.560
5.50	-0.937	0.249	2.670	4.097	4.104	5.546
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-1.163	0.107	2.499	3.897	4.511	5.508
5.00	-0.780	0.348	2.688	4.043	4.930	5.560
5.50	-0.495	0.548	2.835	4.212	5.139	5.579
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.862	0.282	2.589	3.943	4.514	5.521
5.00	-0.238	0.729	2.914	4.233	5.010	5.598
5.50	0.227	1.091	3.152	4.441	5.241	5.608
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.602	0.448	2.725	4.035	4.519	5.534
5.00	0.227	1.083	3.130	4.383	5.013	5.617
5.50	0.837	1.589	3.445	4.649	5.369	5.621

Table D.3 Roding section 1012 structure functions (with barrage)

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.018	2.034	2.018	2.009	2.020	2.025
3.00	3.013	3.012	3.014	3.034	3.033	3.033
4.00	4.007	3.999	3.999	4.000	4.024	4.019
5.00	5.559	5.058	5.046	5.051	5.057	5.059
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	-1.226	0.015	2.443	3.870	4.541	5.497
5.00	-1.100	0.154	2.577	4.086	4.902	5.530
5.50	-0.931	0.266	2.670	4.119	5.118	5.546
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	-1.143	0.108	2.510	3.915	4.540	5.503
5.00	-0.778	0.358	2.688	4.086	4.964	5.561
5.50	-0.491	0.561	2.864	4.217	5.161	5.578
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	-0.859	0.282	2.626	3.953	4.549	5.514
5.00	-0.238	0.729	2.935	4.244	5.018	5.595
5.50	0.231	1.100	3.175	4.487	5.287	5.607
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	-0.602	0.464	2.725	4.034	4.561	5.525
5.00	0.226	1.090	3.147	4.383	5.027	5.615
5.50	0.836	1.602	3.468	4.700	5.413	5.621

Table D.5 Roding section 1017 U structure functions (with barrage)

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.118	2.356	3.004	3.533	3.972	4.364
3.00	2.989	3.027	3.168	3.533	3.972	4.364
4.00	4.018	4.020	4.056	4.160	4.327	4.513
5.00	5.028	5.058	5.067	5.100	5.167	5.262
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	2.133	2.363	3.005	4.008	4.425	5.581
5.00	2.133	2.363	3.005	4.209	5.069	5.616
5.50	2.133	2.363	3.005	4.341	5.286	5.625
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	2.133	2.363	3.005	4.048	4.728	5.586
5.00	2.133	2.363	3.005	4.209	5.126	5.633
5.50	2.133	2.363	3.057	4.341	5.387	5.640
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	2.133	2.363	3.005	4.115	4.728	5.586
5.00	2.133	2.363	3.108	4.334	5.126	5.633
5.50	2.133	2.363	3.297	4.572	5.387	5.640
	1.056 n outflow = 30 $m^3.s^{-1}$					
4.00	2.133	2.363	3.005	4.164	4.731	5.590
5.00	2.133	2.363	3.262	4.492	5.150	5.643
5.50	2.133	2.363	3.551	4.748	5.472	5.648

Table D4 Roding section 1017 D structure functions (with barrage)

Roding mouth levels (m AOD)	Roding flows ($m^3.s^{-1}$) 1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.117	2.339	2.853	3.239	3.571	3.871
3.00	3.009	3.023	3.061	3.239	3.571	3.871
4.00	4.019	4.015	4.017	4.039	4.083	4.122
5.00	5.072	5.075	5.062	5.069	5.082	5.095
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	2.132	2.345	2.854	3.890	4.563	5.495
5.00	2.132	2.345	2.854	4.017	4.918	5.528
5.50	2.132	2.345	2.854	4.134	5.126	5.546
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	2.132	2.345	2.854	3.935	4.571	5.499
5.00	2.132	2.345	2.854	4.114	4.981	5.560
5.50	2.132	2.345	2.936	4.250	5.206	5.578
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	2.132	2.345	2.854	4.005	4.587	5.506
5.00	2.132	2.345	2.999	4.254	5.021	5.593
5.50	2.132	2.345	3.213	4.520	5.324	5.607
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	2.132	2.345	2.854	4.049	4.606	5.513
5.00	2.132	2.345	3.175	4.415	5.065	5.613
5.50	2.132	2.345	3.493	4.716	5.428	5.622

Table D.6 Roding section 1023 BU structure functiona (with barrage)

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	2.122	2.405	3.195	3.812	4.326	4.776
3.00	3.007	3.039	3.313	3.812	4.326	4.776
4.00	4.025	4.027	4.108	4.303	4.577	4.871
5.00	5.040	5.087	5.091	5.163	5.290	5.463
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	2.137	2.412	3.197	4.181	4.907	5.644
5.00	2.137	2.412	3.197	4.283	5.175	5.652
5.50	2.137	2.412	3.197	4.358	5.336	5.669
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	2.137	2.412	3.197	4.210	4.908	5.644
5.00	2.137	2.412	3.197	4.334	5.202	5.660
5.50	2.137	2.412	3.229	4.449	5.377	5.672
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	2.137	2.412	3.197	4.241	4.909	5.645
5.00	2.137	2.412	3.263	4.462	5.254	5.561
5.50	2.137	2.412	3.413	4.636	5.462	5.676
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	2.137	2.412	3.197	4.305	4.910	5.646
5.00	2.137	2.412	3.394	4.584	5.263	5.676
5.50	2.137	2.412	3.629	4.809	5.556	5.681

Table D.9 Roding section 1048 structure functions (with barrage)

	Roding flows ($m^3.s^{-1}$)					
Roding mouth levels (m AOD)	1.0	5.0	20.0	35.0	50.0	65.0
a. barrier open						
2.00	4.968	5.252	6.119	6.825	7.377	7.791
3.00	4.980	5.252	6.119	6.825	7.377	7.791
4.00	4.980	5.252	6.120	6.828	7.380	7.795
5.00	5.167	5.374	6.196	6.894	7.410	7.833
b. barrier closed						
	Beckton outflow = 5 $m^3.s^{-1}$					
4.50	4.968	5.252	6.119	6.827	7.389	7.840
5.00	4.968	5.252	6.119	6.828	7.399	7.841
5.50	4.968	5.252	6.119	6.829	7.409	7.845
	Beckton outflow = 10 $m^3.s^{-1}$					
4.50	4.968	5.252	6.119	6.827	7.389	7.840
5.00	4.968	5.252	6.119	6.828	7.400	7.841
5.50	4.968	5.252	6.119	6.830	7.413	7.845
	Beckton outflow = 20 $m^3.s^{-1}$					
4.50	4.968	5.252	6.119	6.827	7.389	7.841
5.00	4.968	5.252	6.119	6.830	7.403	7.842
5.50	4.968	5.252	6.129	6.834	7.420	7.845
	Beckton outflow = 30 $m^3.s^{-1}$					
4.00	4.968	5.252	6.119	6.828	7.389	7.841
5.00	4.968	5.252	6.119	6.832	7.405	7.842
5.50	4.968	5.252	6.119	6.839	7.427	7.845

Annex E Annual maximum water levels with the barrage

	1001u	1006u	1012	1017d	1017u	1023bu	1031u	1041u	1048
1950	4.284	4.292	4.298	4.314	4.320	4.341	4.403	5.165	6.068
1951	4.662	4.676	4.692	4.708	4.692	4.709	4.744	5.521	6.434
1952	4.433	4.443	4.454	4.469	4.460	4.475	4.513	5.161	6.063
1953	4.608	4.621	4.635	4.651	4.640	4.658	4.697	5.416	6.325
1954	4.409	4.419	4.428	4.444	4.439	4.454	4.498	5.243	6.145
1955	4.536	4.548	4.560	4.576	4.573	4.594	4.642	5.161	6.064
1956	4.099	4.106	4.109	4.124	4.124	4.132	4.171	5.090	5.990
1957	4.468	4.480	4.493	4.507	4.493	4.506	4.538	5.109	6.008
1958	4.417	4.427	4.435	4.452	4.465	4.496	4.572	5.545	6.463
1959	4.247	4.253	4.260	4.281	4.327	4.390	4.529	5.679	6.594
1960	4.216	4.224	4.230	4.245	4.243	4.260	4.446	5.756	6.679
1961	4.730	4.745	4.764	4.778	4.755	4.771	4.801	5.224	6.127
1962	4.309	4.319	4.326	4.341	4.349	4.373	4.441	5.320	6.225
1963	4.217	4.225	4.231	4.246	4.243	4.346	4.561	5.692	6.591
1964	4.190	4.198	4.206	4.220	4.214	4.233	4.593	5.999	6.927
1965	4.403	4.413	4.423	4.438	4.429	4.443	4.480	5.531	6.442
1966	4.608	4.622	4.638	4.652	4.632	4.646	4.675	5.491	6.401
1967	4.604	4.617	4.634	4.648	4.629	4.643	4.674	5.455	6.268
1968	4.447	4.457	4.466	4.482	4.493	4.522	4.593	5.755	6.674
1969	4.396	4.407	4.419	4.433	4.420	4.431	4.460	5.569	6.489
1970	4.272	4.281	4.290	4.304	4.297	4.307	4.363	5.380	6.290
1971	4.348	4.357	4.364	4.380	4.383	4.404	4.462	5.326	6.234
1972	4.325	4.335	4.345	4.359	4.349	4.359	4.388	5.191	6.093
1973	4.771	4.786	4.806	4.820	4.796	4.812	4.842	4.944	5.611
1974	4.228	4.234	4.239	4.257	4.331	4.738	5.171	6.904	7.756
1975	4.473	4.483	4.491	4.508	4.529	4.566	4.652	5.552	6.471
1976	4.474	4.486	4.499	4.513	4.499	4.511	4.542	5.366	6.276
1977	4.562	4.574	4.589	4.604	4.587	4.602	4.633	5.684	6.572
1978	4.495	4.507	4.517	4.534	4.556	4.619	4.750	5.822	6.744
1979	4.401	4.410	4.418	4.438	4.476	4.548	4.729	5.723	6.592
1980	4.404	4.415	4.427	4.441	4.429	4.447	4.521	5.347	6.208
1981	4.510	4.522	4.536	4.551	4.535	4.548	4.578	5.531	6.443
1982	4.312	4.321	4.328	4.344	4.340	4.354	4.515	5.980	6.911
1983	4.530	4.541	4.553	4.569	4.567	4.588	4.637	5.441	6.349
1984	4.239	4.245	4.251	4.268	4.294	4.335	4.435	5.187	6.052
1985	4.434	4.445	4.456	4.471	4.463	4.477	4.599	5.591	6.461
1986	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
1987	4.190	4.197	4.203	4.218	4.253	4.364	4.623	5.858	6.771
1988	4.644	4.657	4.675	4.689	4.669	4.683	4.714	5.984	6.922
1989	4.403	4.414	4.426	4.440	4.426	4.437	4.481	5.494	6.410
1990	4.618	4.631	4.646	4.662	4.647	4.664	4.699	5.971	6.899
1991	4.251	4.260	4.265	4.280	4.278	4.290	4.336	4.820	5.720

Annex F Barrier closures

Table F1. Barrier closure dates

Date 1006	Southend Level	Kingston Flow	Tower Pier Level	Roding Level	Roding Flow	Site Level
Barriers close if Tower Pier ≥ 4.87 (legal level)						
28 11 1951	3.785	203.0	4.928	4.654	1.736	-0.639
1 2 1953	4.610	72.3	5.410	5.390	2.128	-0.024
10 12 1965	4.145	222.0	5.243	4.958	22.58	2.706
12 1 1978	4.210	225.0	5.120	5.016	21.20	2.643
31 12 1978	4.030	139.0	5.210	4.857	4.797	0.187
25 12 1988	2.450	22.8	4.979	n.a.	0.530	n.a.
1 3 1990	3.810	146.0	4.918	4.674	2.22	-0.737