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RIVER RODING FLUVIAL/TIDAL FLOOD DEFENCE PROJECT

PHASE II: ANALYSIS OF HISTORICAL DATA

Report to the National Rivers Authority, Thames Region

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Executive summary

1. As part of review of flood defences in the Thames estuary, the National Rivers Authority required an assessment of level of flood protection along the River Roding, from Redbridge to its confluence with the Thames.

2. 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.

3. 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.

4. 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.

5. 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.

6. A hydraulic model was constructed to calculate water levels along the study reach given the fluvial flow, barrier status, effluent discharge and water levels at the Roding mouth. The model allowed water to spill overbank into a flood-plain of infinite storage capacity. The modelling exercise indicated that if high flows in the Roding occur when the barrier is closed, water levels upstream of the barrier may exceed those in the Thames, preventing barrier opening and thus leading to flooding along the lower reaches of the Roding.

7. For the period 1950-1991 a series of annual maximum water levels at nine critical cross-sections was derived and a statistical distribution was fitted to each to estimate return period 2 to 1000 years. This required a large extrapolation beyond the available data. The combination of barrier closure and high Roding flows did not occur in this historical period, thus its return period could not be assessed.

8. For most sections the flood defences give a level of protection greater than 1000 years return period. However, at one section the level of protection appears to be around 270 years, whilst for one others, it is 25 years.

9. For the sections with a 1000 year protection level flood risk is not affected by potential closure of the barriers at a lower water level of 4.67 m. At the section with a 270 return period, the protection level is increased to 450 years.

10. Additional analysis showed that increases in sea level of 6 mm yr^{1} predicted for the year 2030 are likely to result in increased flood levels in the Roding of up to 130 mm for return periods less than 10 years, but the historical data do not allow precise definition of increases for higher return periods. Nevertheless, this degree of water level rise is unlikely to reduce the level of protection for those section with a current protection level of 1000 years return period.

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1. Background

As part of its review of the level of service provided by flood defences in the Thames estuary, the National Rivers Authority required an assessment of the probability of water levels exceeding critical values along the lower reaches of the River Roding.

Two major factors influence water levels in the lower Roding:

(1) water levels in the Thames at the mouth of the Roding; and
(2) fluvial flows from the Roding catchment;

Thames water levels are themselves controlled by the height of the tide and flows from the Thames catchment and are influenced by whether or not the Thames barrier is closed.

The lower Roding can be protected from high water in the Thames by closure of the Barking barrier, which spans the Roding at its mouth. Fluvial flows may be increased by effluent discharge.

A hydraulic model of the lower Roding had been developed to predict water levels given any combination of Thames water level and Roding river flow.

The Institute of Hydrology was contracted to undertake a frequency analysis of the water levels up to the 1000 year return period, based on the joint probability modelling of coincident high Thames estuary levels and fluvial flows.

The study was divided into two phases:

Phase I: feasibility study to ensure that adequate information was available to undertake the study

Phase II: full stage frequency analysis

This document is the final report of Pase II

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

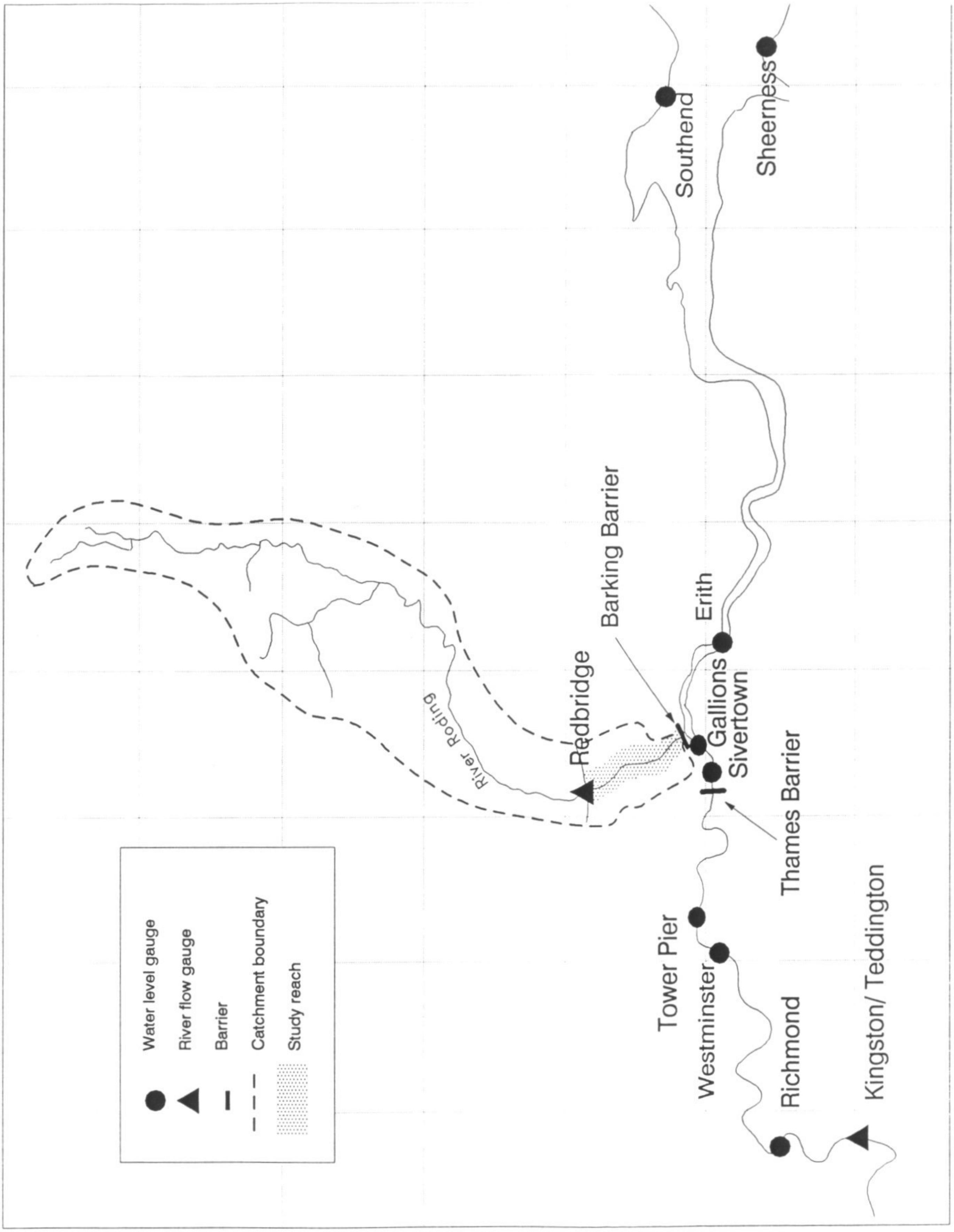

Figure 2.1 Location map

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 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) a hydraulic model to provide water levels along the study reach given (a) and (b); and

(d) a statistical model to analyse frequency of resulting levels.

Each of these elements is discussed in detail in the following sections.

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:

$$F = (m-0.44)/(n+0.12) \qquad (4.1)$$

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

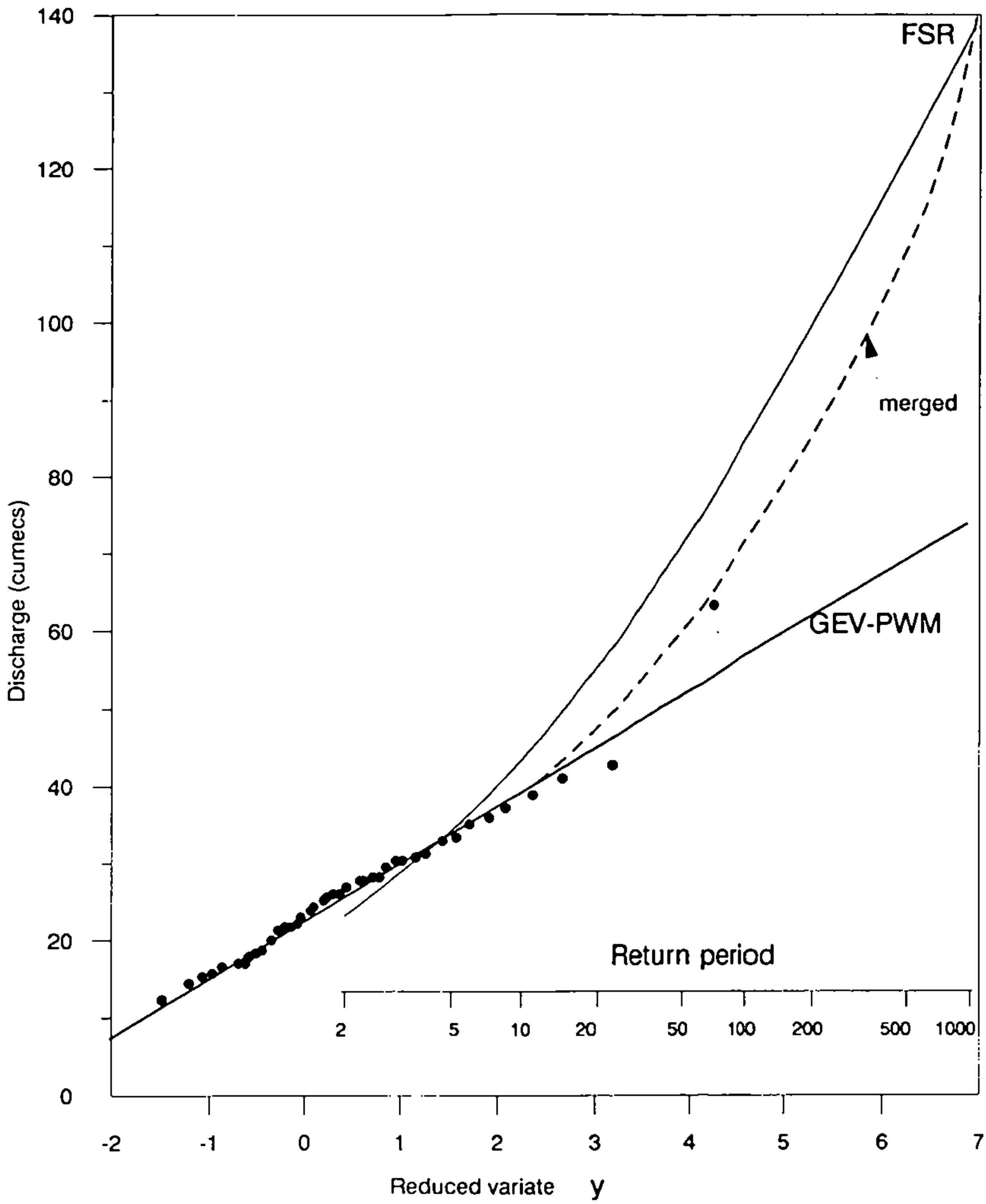

Figure 4.1 *Flood frequency curves for the River Roding at Redbridge*

(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 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

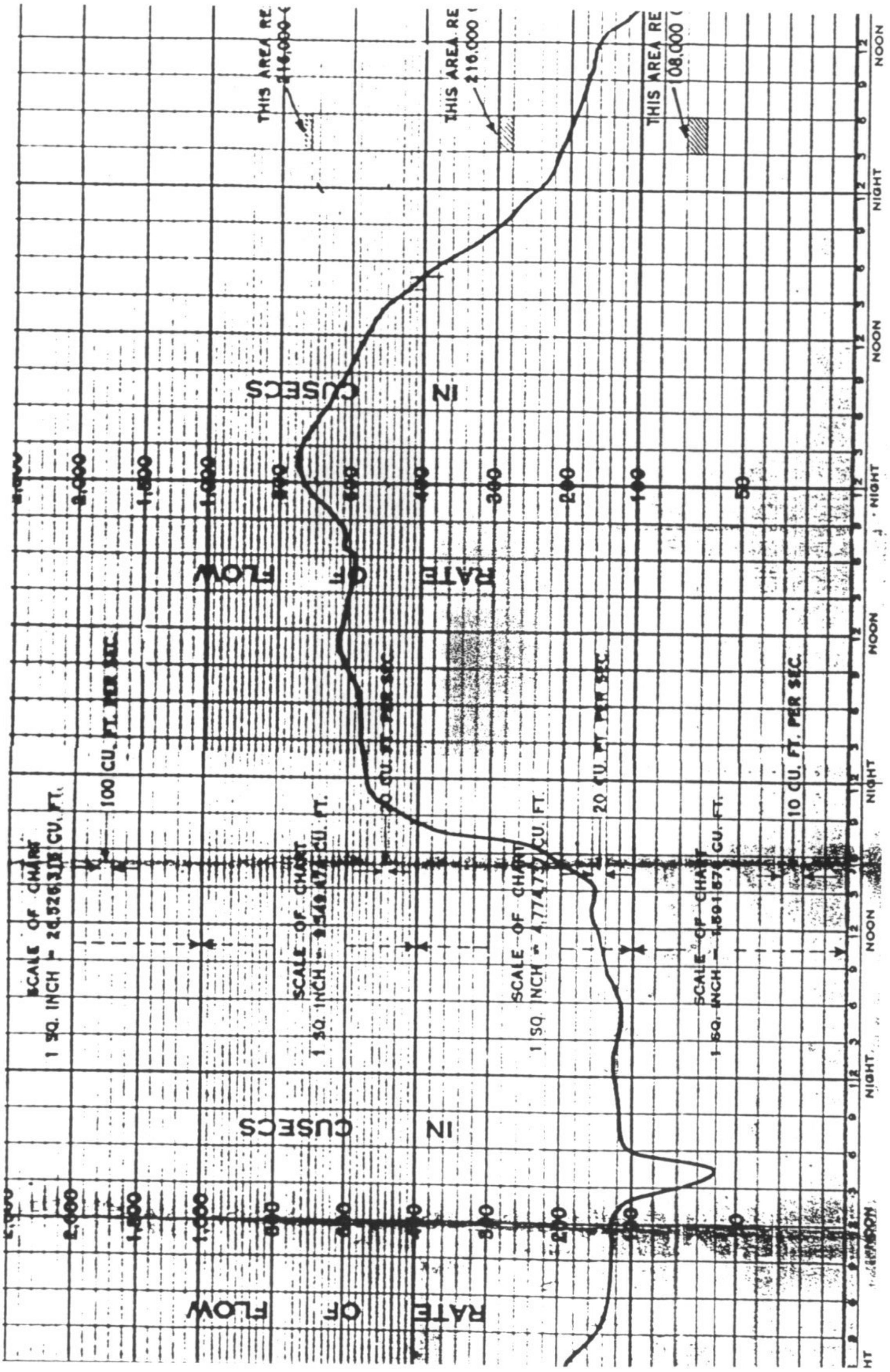

Figure 4.2 *Flow hydrograph for the Roding at Redbridge 2-6/3/54*

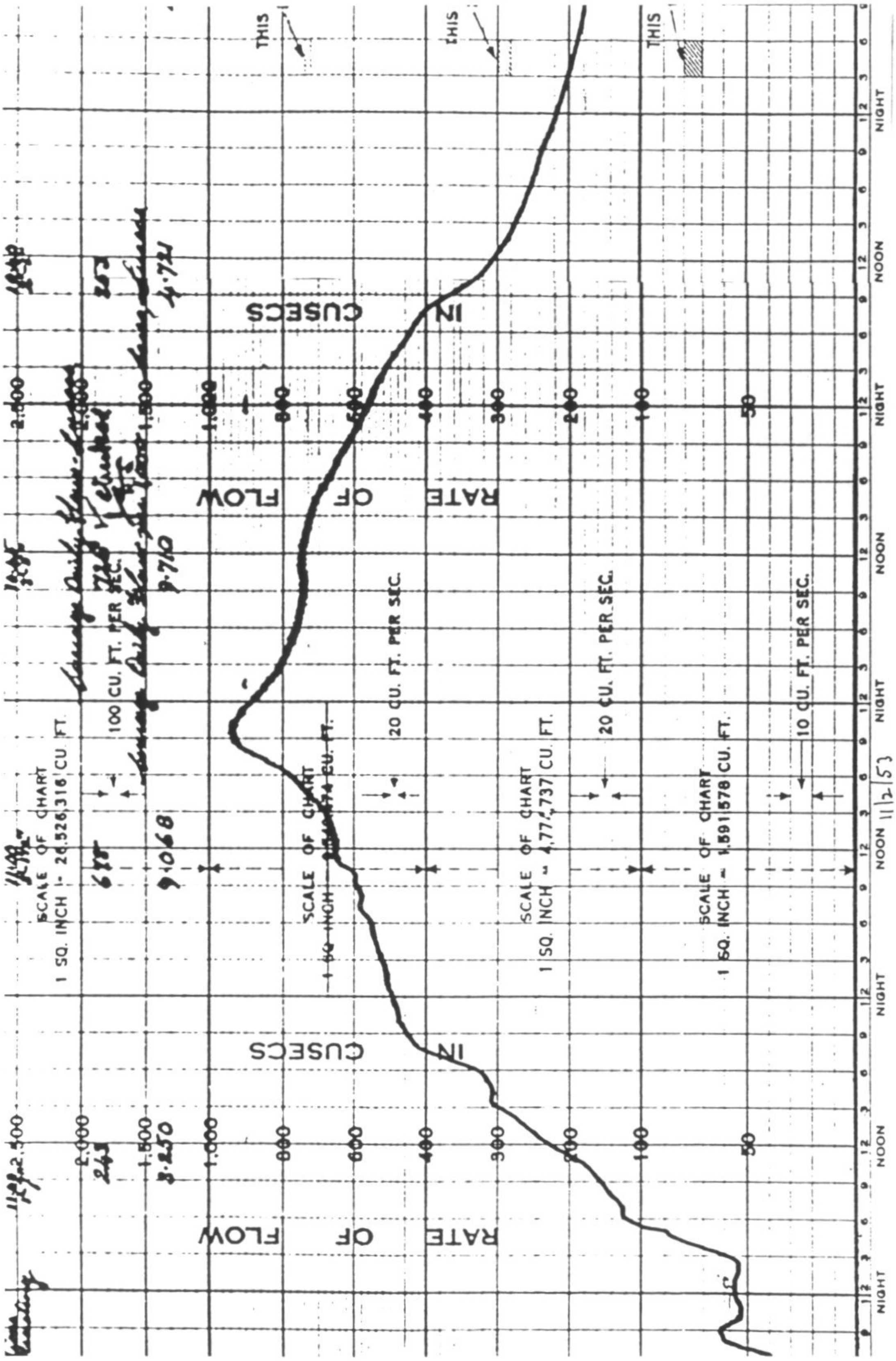

Figure 4.3 *Flow hydrograph for the Roding at Redbridge 9-14/2/53*

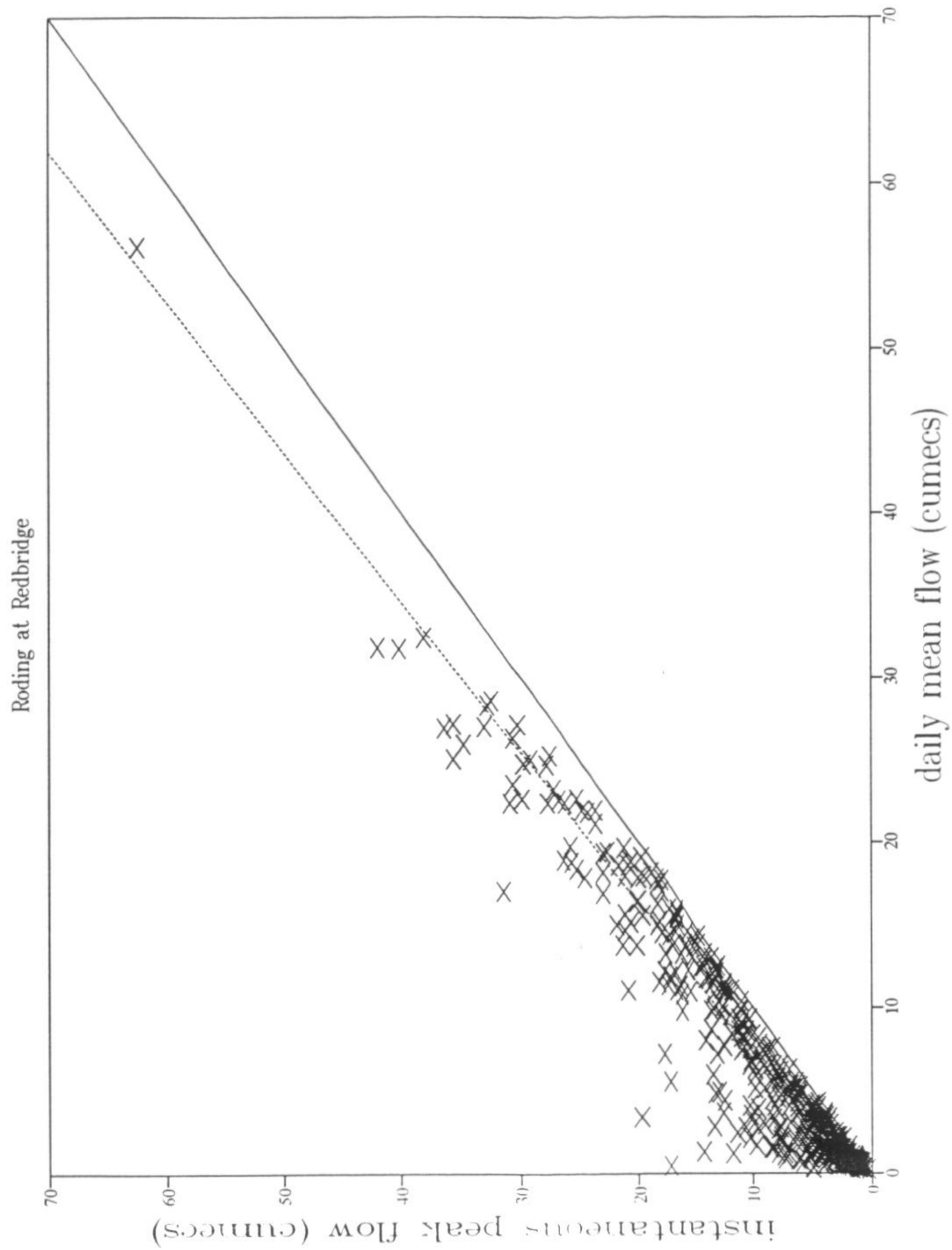

Figure 4.4 *Highest instantaneous flows at Redbridge against mean flow for the day of the peak (m^3s^{-1}).*

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.

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

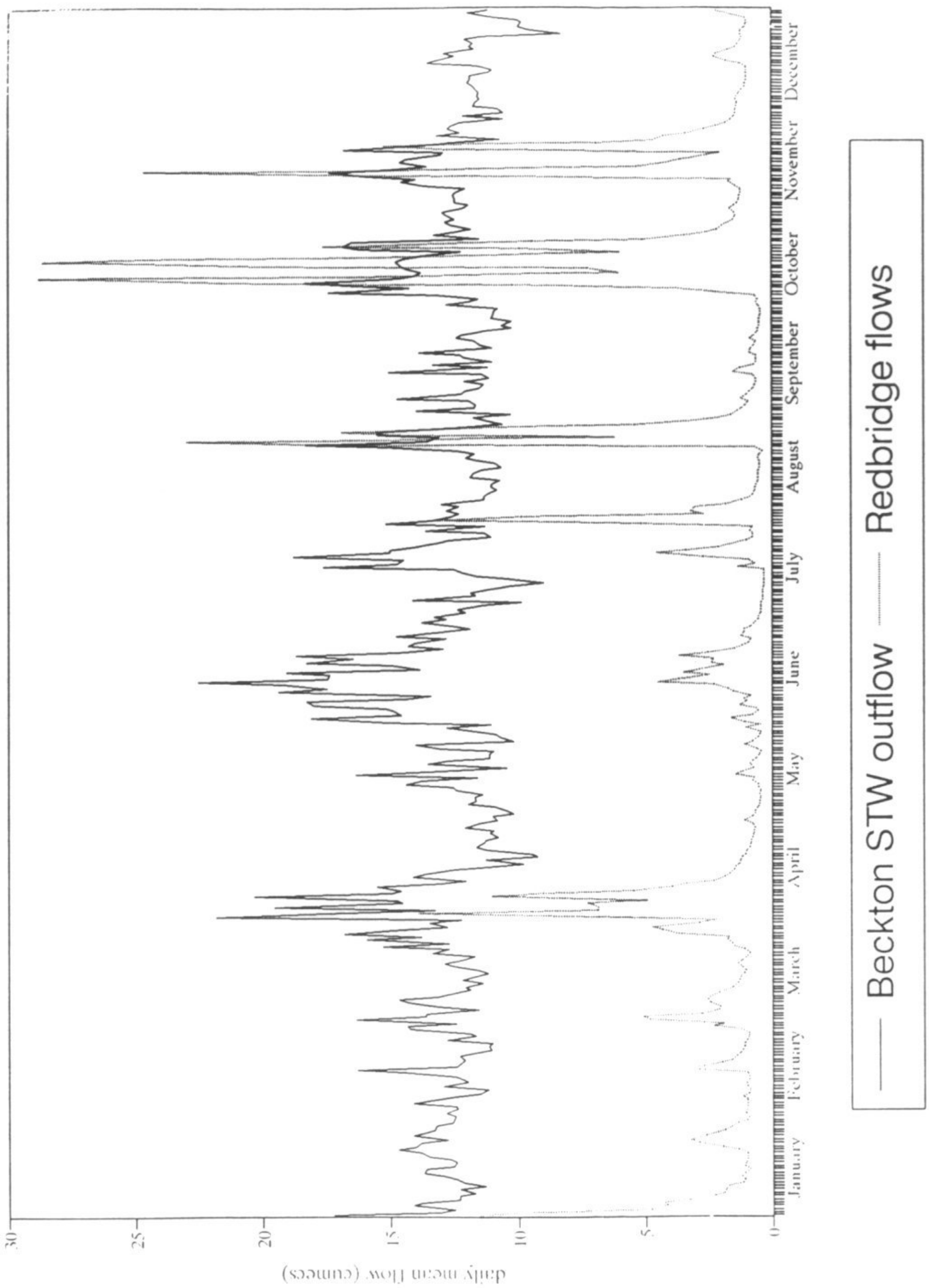

Figure 5.1 *Daily mean discharges from Beckton STW for 1987 together with daily mean flows at Redbridge for the corresponding days (m^3s^{-1}).*

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 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 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.

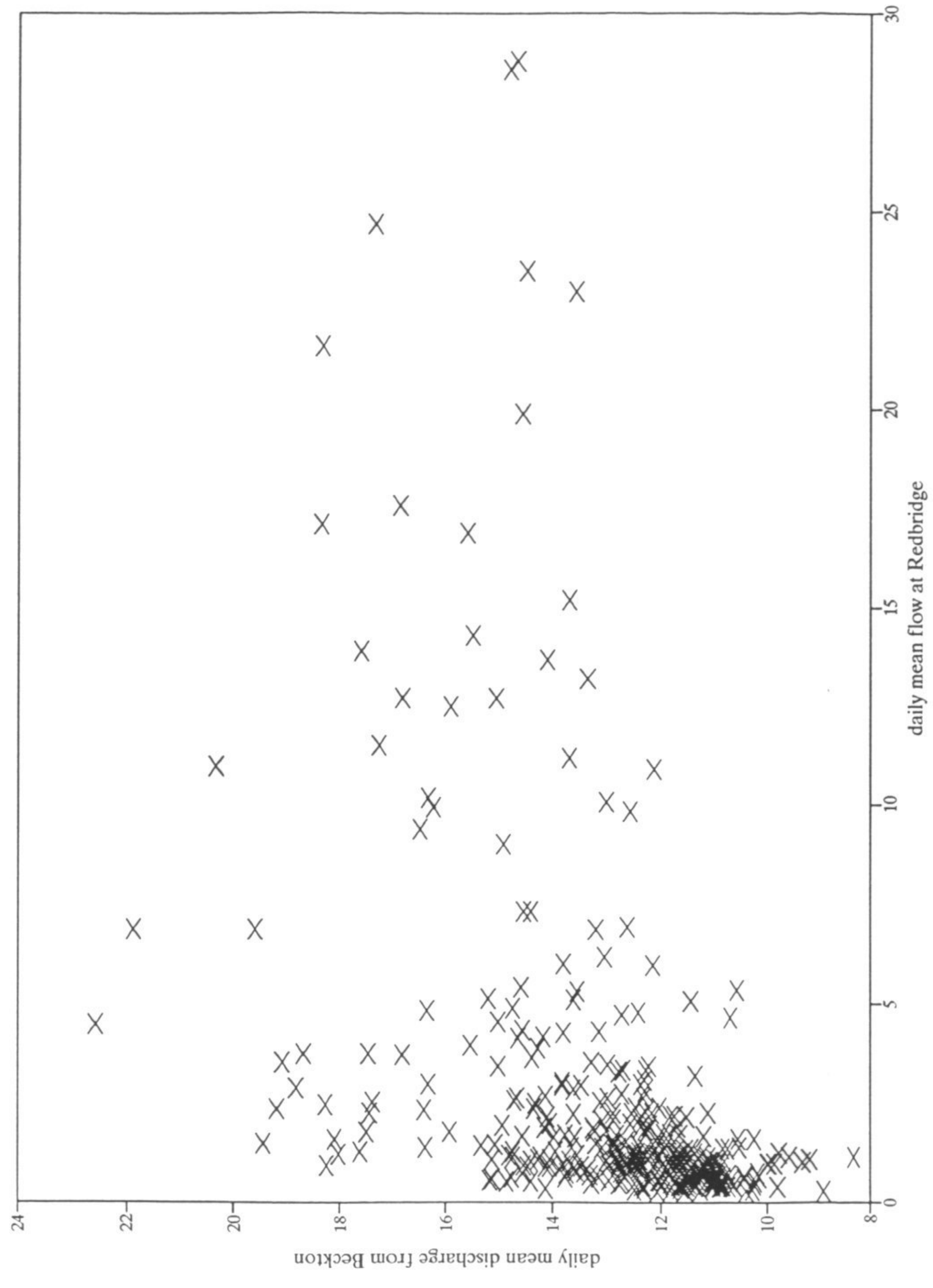

Figure 5.2 Daily mean discharges from Beckton STW for 1987 against daily mean flows at Redbridge for the corresponding days (m^3s^{-1}).

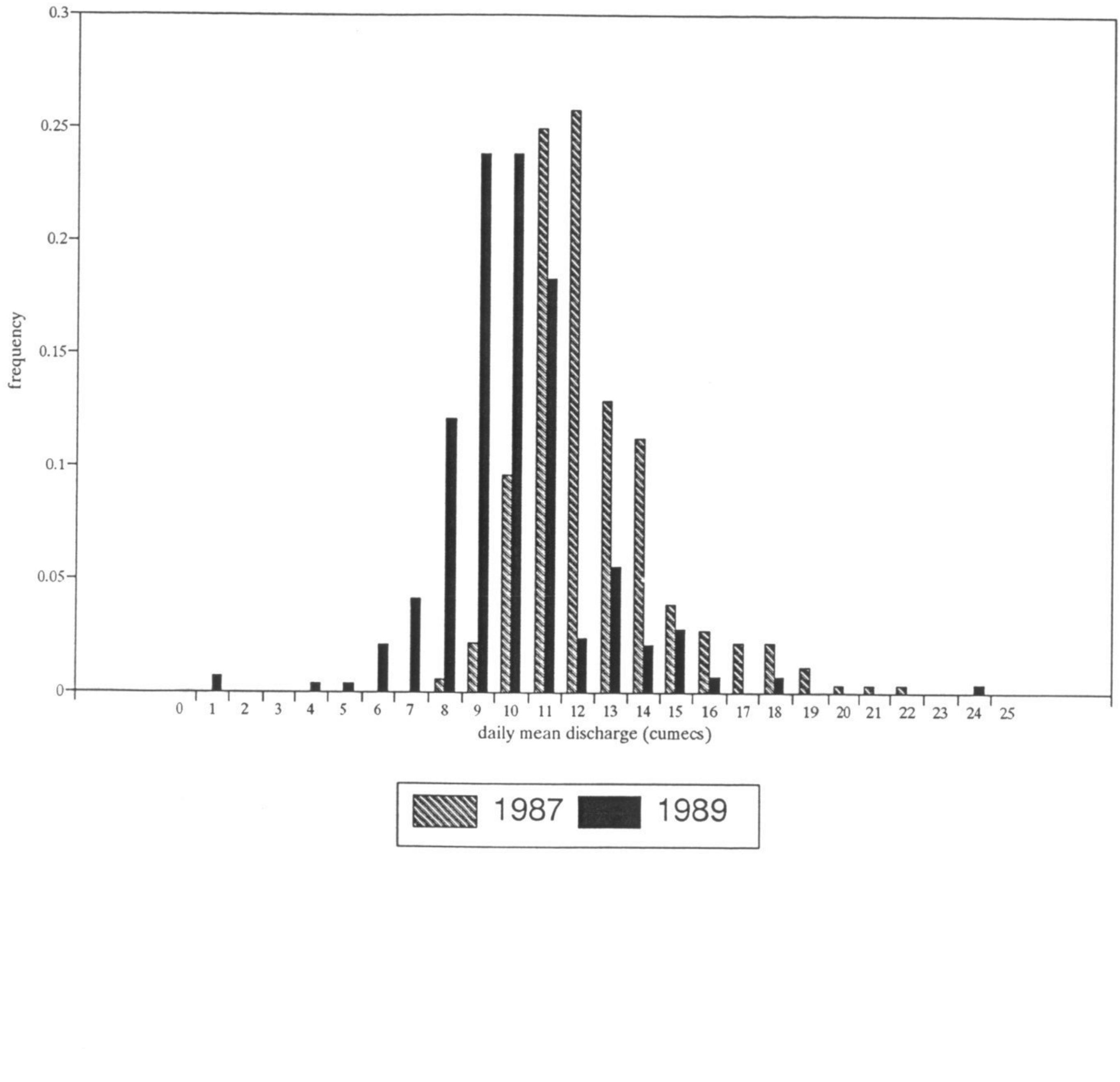

Figure 5.3 *Histograms of daily mean discharges from Beckton STW for 1987 and for 1989 (m^3s^{-1}).*

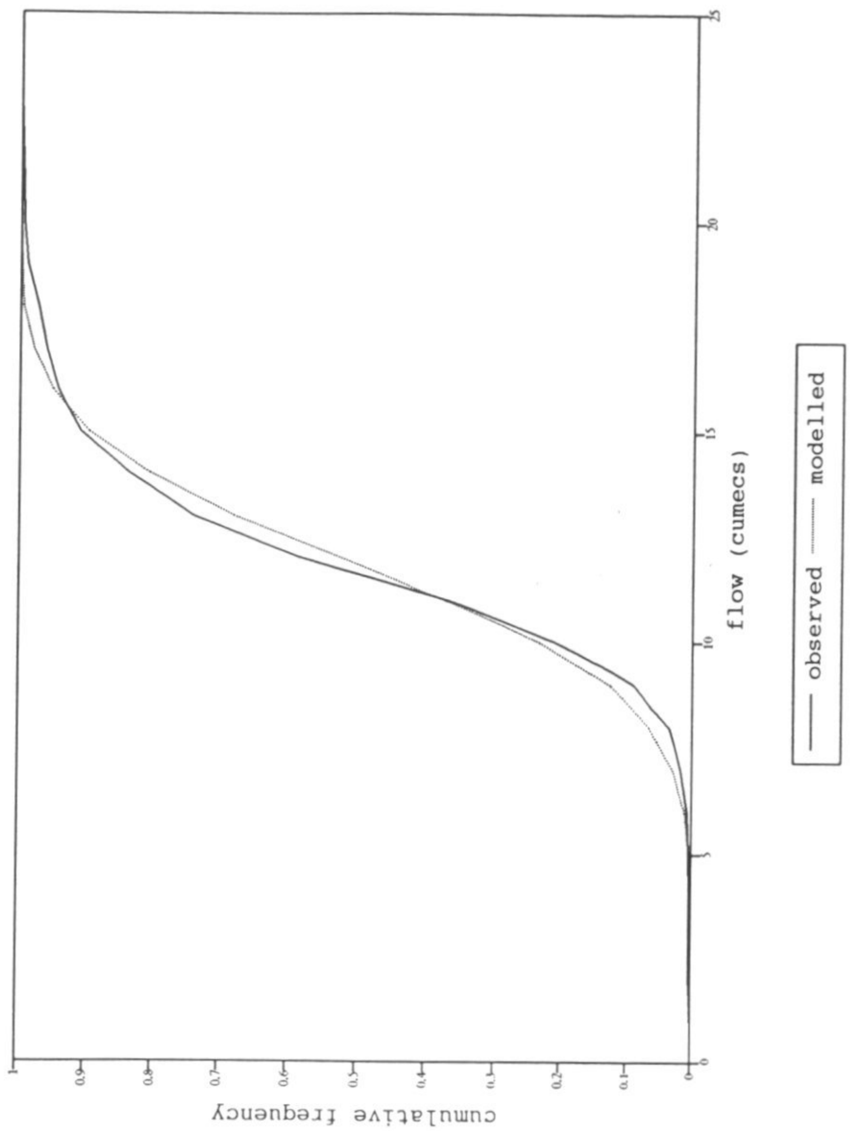

Figure 5.4 *Cumulative frequency of daily mean discharges from Beckton STW for 1987 and 1989 combined together with a normal distribution*

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:

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

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.

Recommendation: 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

Recommendation: 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

most 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

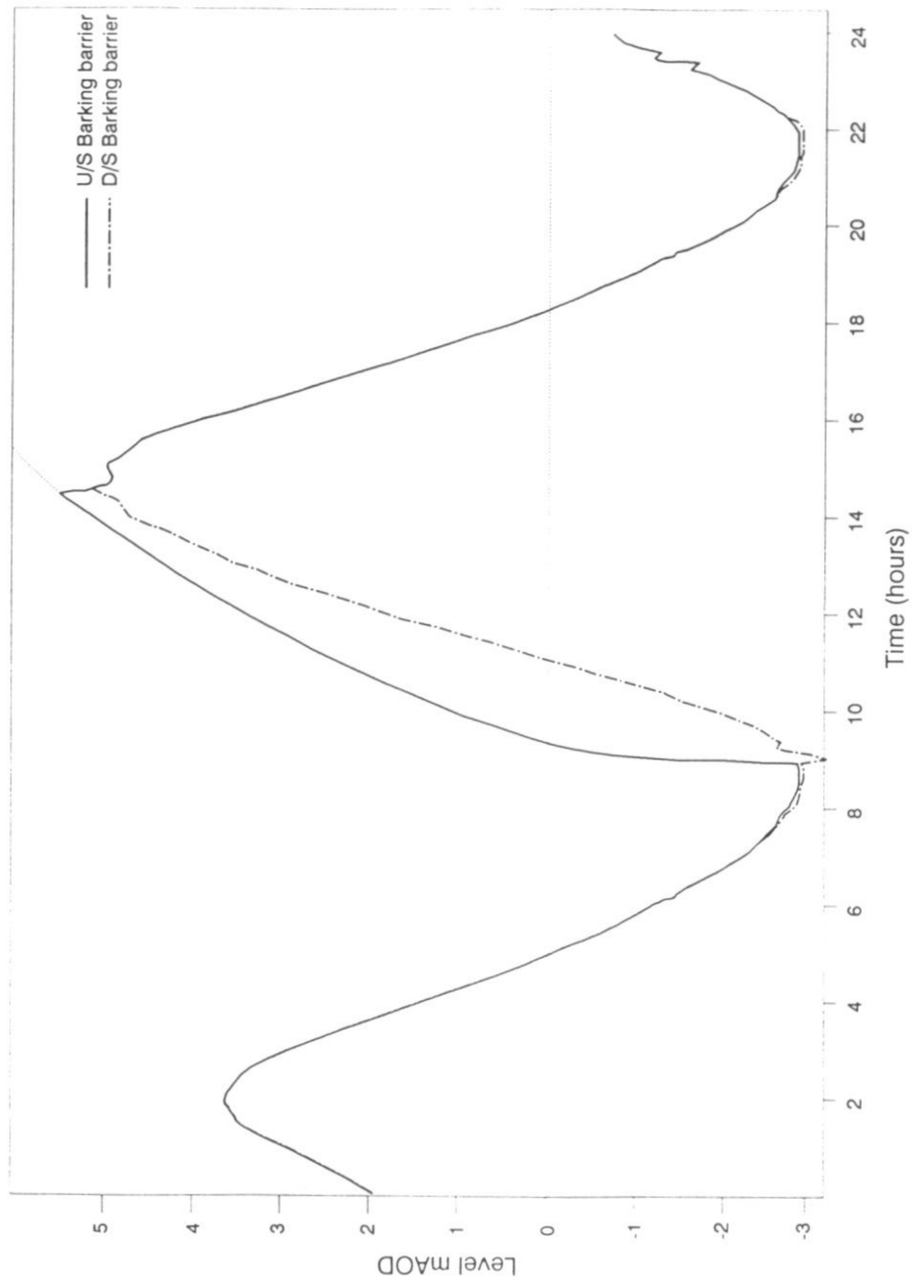

Figure 9.1 *Nightmare scenario example*

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.

Recommendation: 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 D 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.

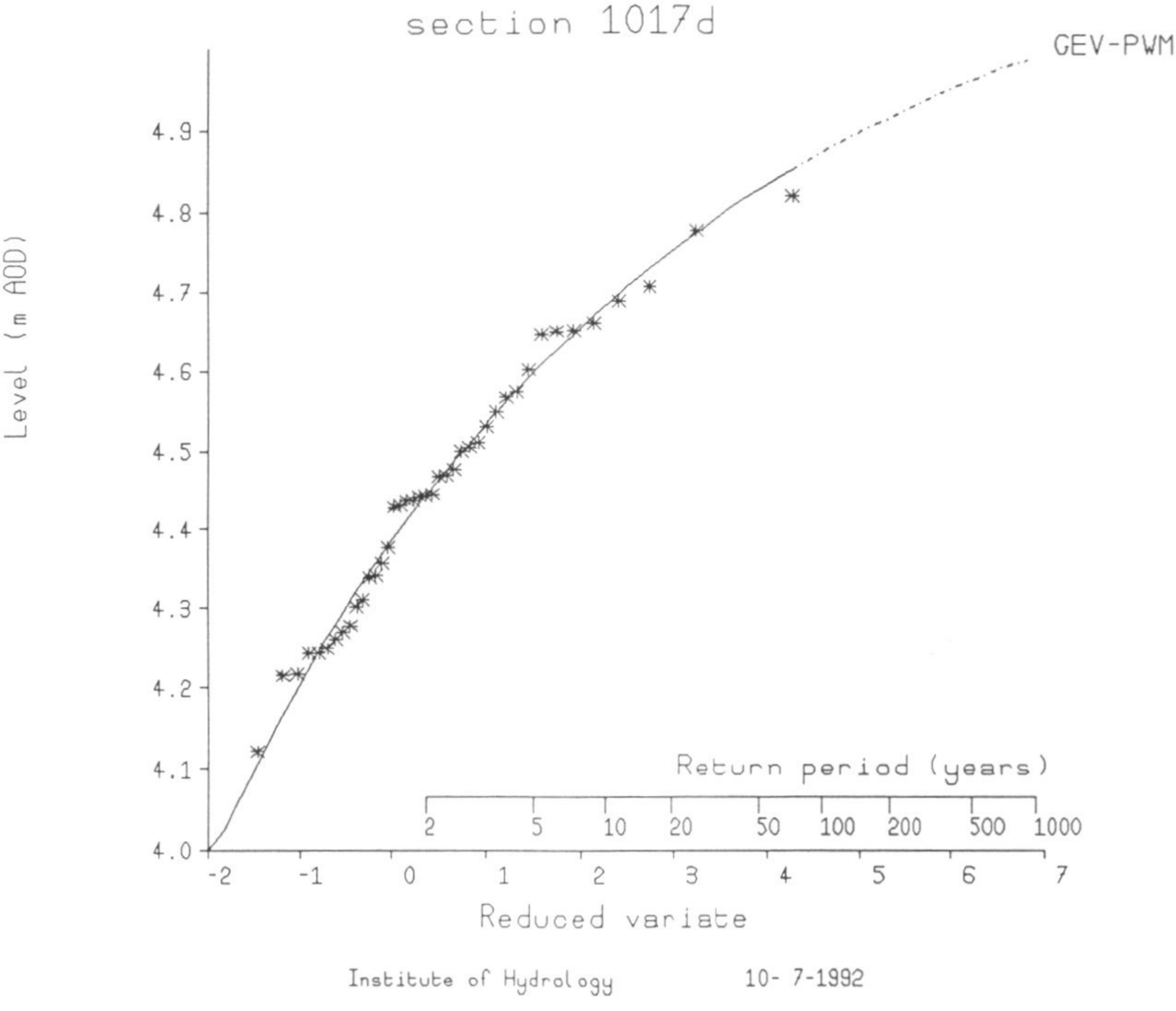

Figure 12.1 *Example level/frequency curve for section 1017d.*

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. Sensitivity of results to barrier operating rules

As indicated in section 7, closure of the barriers relies on the judgement of the operations manager based on forecasts of water levels. A sensitivity analysis was undertaken to investigate the implications of closing the barrier at a lower level than 4.87 (ie equivalent to an over estimate in the forecast). The water level/frequency analysis on the River Roding was repeated assuming that the barriers close when the level at London Bridge reached 4.76 m AOD. The resulting annual maximum levels are in most years the same as or slightly lower than with the barrier closure at 4.87 m (Annex D), although the barriers would close three times as often (see Annex F).

Table 13.1 shows the output from the frequency analysis providing water levels for various return periods for barrier closure at 4.67 compared with that for closure at 4.87 (given in brackets). At low return periods (2-5 years) water levels are reduced by around 8 cm. This increases to around 15 cm for high return periods. The exception is at Redbridge (section 1048) where changes in the operating rule of the barrier has no effect, since the section is upstream of any tidal influence.

Table 13.1 *Water levels of various return periods with barrier closure at 4.67 m AOD (equivalent values for closure at 4.87 are given in brackets).*

	Return period (years)							
Section	2	5	10	50	100	200	500	1000
1001u	4.33 (4.41)	4.43 (4.55)	4.49 (4.63)	4.61 (4.77)	4.65 (4.81)	4.69 (4.85)	4.74 (4.89)	4.78 (4.92)
1006	4.33 (4.41)	4.43 (4.56)	4.49 (4.64)	4.62 (4.78)	4.66 (4.83)	4.71 (4.87)	4.76 (4.91)	4.80 (4.94)
1012	4.34 (4.43)	4.45 (4.58)	4.51 (4.66)	4.64 (4.81)	4.69 (4.86)	4.73 (4.90)	4.78 (4.94)	4.82 (4.97)
1017d	4.36 (4.44)	4.47 (4.60)	4.53 (4.68)	4.66 (4.83)	4.71 (4.87)	4.75 (4.92)	4.80 (4.96)	4.84 (4.99)
1017u	4.36 (4.45)	4.47 (4.59)	4.53 (4.67)	4.65 (4.80)	4.69 (4.84)	4.73 (4.88)	4.77 (4.92)	4.81 (4.94)
1023b	4.39 (4.47)	4.49 (4.60)	4.54 (4.68)	4.66 (4.80)	4.70 (4.84)	4.74 (4.88)	4.78 (4.92)	4.81 (4.94)
1031u	4.48 (4.56)	4.62 (4.70)	4.70 (4.78)	4.87 (4.92)	4.94 (4.97)	5.01 (5.01)	5.10 (5.06)	5.16 (5.09)
1041u	5.47 (5.47)	5.78 (5.78)	5.98 (5.98)	6.33 (6.36)	6.46 (6.51)	6.58 (6.64)	6.71 (6.81)	6.81 (6.93)
1048	6.37 (6.37)	6.70 (6.70)	6.89 (6.89)	7.26 (7.26)	7.39 (7.39)	7.52 (7.52)	7.66 (7.66)	7.76 (7.76)

14. Sensitivity of results to sea level rise

The Inter-governmental Panel on Climate Change (*Houghton et al*, 1990) predicted that sea levels would rise as a result of global warming. NRA (1991) combined these predictions with the expected tectonic and isostatic changes to indicate sea level changes likely to occur by 2030, 2070 and 2100. For the period 1990-2030 a rise of 6 mm per year at Southend is expected (based on the IPCC "business as usual, best estimate").

The NRA paper does not recommend any retrospective calculation to standardised recorded sea-levels prior to 1990, thus it was assumed during this study that the records used for the period 1950-1991 were samples from a stationary series.

The 6 mm rise is clearly an average figure and its application to individual high water levels is unclear. It is possible, for example, that the astronomical element of the tide will increase by 6 mm but that the surge element will remain the same. However, for this study the high water levels were not separated into their astronomical and surge components. A further complication is that the structure functions relating tide level at Southend and flow at Teddington/Kingston might not be entirely appropriate for generally higher sea-levels.

To simplify the modelling exercise 234 mm was added to each historical high water level at Southend (6 mm yr^{-1} for each year 1991 to 2030). The frequency analysis was then repeated using these new Southend data to assess the expected frequency of given water levels in the Roding in 2030. The only change in the procedure was that water levels in the Thames at the Roding mouth were derived exclusively using the Southend levels along with the Teddington/Kingston flows and the appropriate structure function. High water levels at measurement stations within the Thames estuary (such as Gallions) were not used as their data could not be easily adjusted to predict the 2030 levels since they are themselves a result of the interaction of tides and river flows. This means that the two Roding water level series generated are not absolutely compatible.

The resulting annual maximum levels are given in Annex E. Once again maximum levels occurred when the barrier was open, even though it would be closed more often.

Table 14.1 shows the water levels of various return periods at each critical cross section along the Roding. It can be seen that for low return periods (up to 10 years) water levels will be higher in 2030 by up to 130 mm.

In contrast, for return periods, above 10 years the historical data show that water levels will be lower. In theory it would expected that flood levels for all return periods would be increased (or at least not significantly different). This is because an increase in tide levels (exhibited as a translation to the right of the density function of tide levels) would mean an increase in the probability of experiencing a level just below that which would trigger the barrier to close (see Figure 14.1a).

Table 14.1 *Water levels of various return periods predicted for 2030 (equivalent values for 1991 are given in brackets).*

	Return period (years)							
Section	2	5	10	50	100	200	500	1000
1001u	4.52 (4.41)	4.59 (4.55)	4.63 (4.63)	4.68 (4.77)	4.69 (4.81)	4.71 (4.85)	4.72 (4.89)	4.72 (4.92)
1006	4.53 (4.41)	4.46 (4.56)	4.64 (4.64)	4.69 (4.78)	4.71 (4.83)	4.72 (4.87)	4.73 (4.91)	4.74 (4.94)
1012	4.55 (4.43)	4.62 (4.58)	4.66 (4.66)	4.72 (4.81)	4.73 (4.86)	4.74 (4.90)	4.76 (4.94)	4.77 (4.97)
1017d	4.57 (4.44)	4.64 (4.60)	4.68 (4.68)	4.73 (4.83)	4.75 (4.87)	4.76 (4.92)	4.78 (4.96)	4.78 (4.99)
1017u	4.56 (4.45)	4.67 (4.59)	4.66 (4.67)	4.71 (4.80)	4.72 (4.84)	4.73 (4.88)	4.74 (4.92)	4.74 (4.94)
1023b	4.58 (4.47)	4.64 (4.60)	4.67 (4.68)	4.71 (4.80)	4.72 (4.84)	4.72 (4.88)	4.73 (4.92)	4.73 (4.94)
1031u	4.65 (4.56)	4.74 (4.70)	4.79 (4.78)	4.86 (4.92)	4.88 (4.97)	4.90 (5.01)	4.92 (5.06)	4.93 (5.09)
1041u	5.48 (5.47)	5.80 (5.78)	5.99 (5.98)	6.35 (6.36)	6.49 (6.51)	6.61 (6.64)	6.76 (6.81)	6.87 (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 (7.7)

An apparent decrease in levels for return periods above 10 years can occur however if the historical data analysed do not have a smooth density function (see Figure 14.1b). This can be explained as follows. A number of the annual maximum water levels in the Roding will result from peak levels in the Thames just below that which will trigger the barrier to close. Any slight increase in Thames level due to climate change will cause the barrier to close and hence, for that tide, water levels in the Roding upstream of the barrier will be lower. It will therefore be another tide in the year which will generate the annual maximum level in the Roding. If the second highest Thames level after climate change is lower than the previous highest before climate change the annual maximum water level in the Roding will be lower; assuming of course that the Roding flow is the same during both tides. If the second highest Thames level in the year coincides with a lower Roding flow, the resulting annual maximum level in the Roding may be lower even when the second highest Thames level is elevated to the same height by climate change as the previous highest.

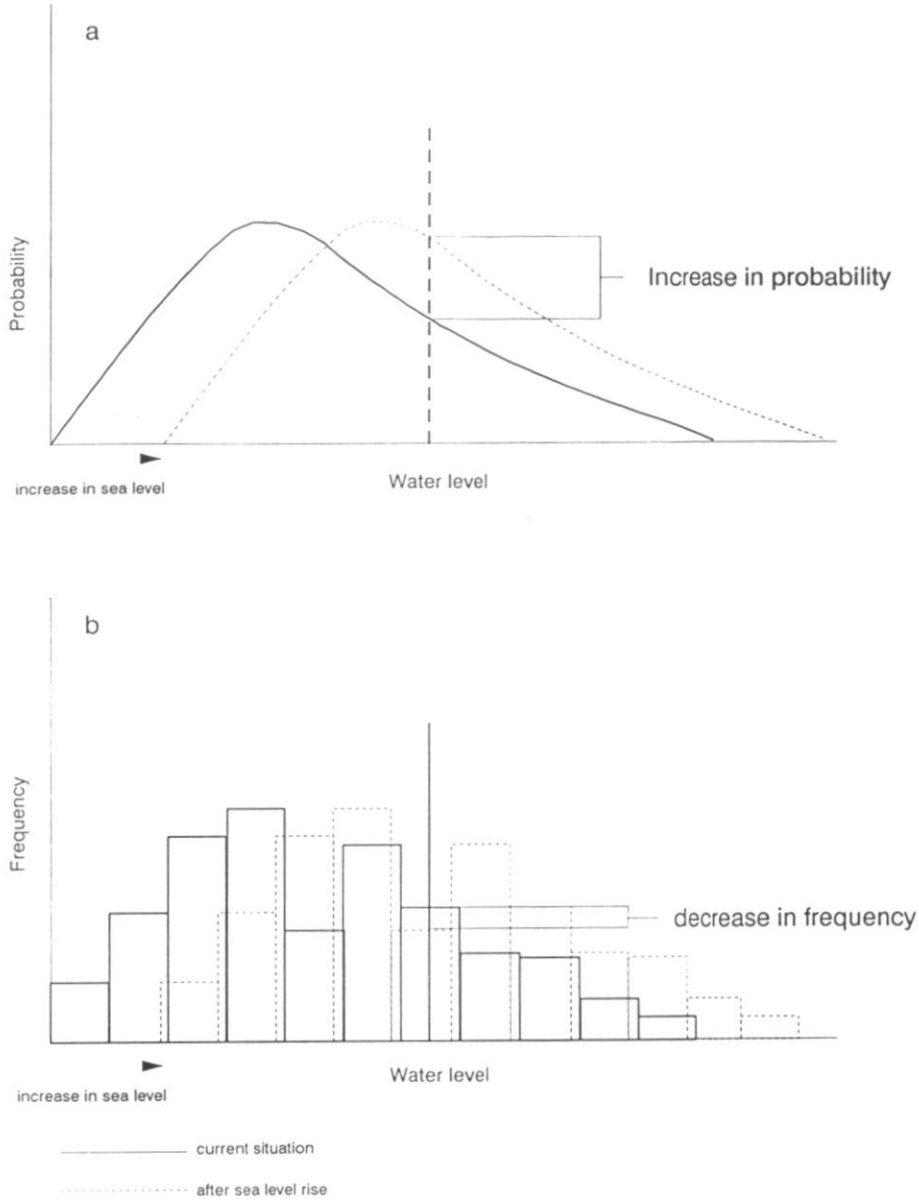

Figure 14.1 *Effects of sea-level rise on the probability of experiencing water levels just below the barrier closure level threshold*

This phenomena is illustrated in Table 14.2 which gives the four highest levels at cross section 1006, immediately upstream of the barrier, for each of the years 1950, 1952 and 1954. Also given are the levels for the same dates after the sea level rise expected by 2030. For 1950, all Roding levels increase and the annual maximum remains so (the 3rd and 4th ranks change due to use of different structure functions). In 1952 elevation of the annual maximum leads to a barrier closure, thus the previous 2nd rank becomes the annual maximum. However, even though its level is increased from 4.120 m to 4.385 m this is still lower than the 4.434 m of the previous maximum. For 1954 sea level rise causes the barrier to close on the days of the highest and second highest values with the result that the 3rd ranked becomes the annual maximum. In this case the new maximum level 4.458 m is greater than the previous highest 4.410 m.

Table 14.2 Effects on annual maximum water levels of sea-level rise

Date			Present situation			Following sea level rise		
			Maximum level	Rank	Barrier closure	Maximum level	Rank	Barrier closure
6	2	1950	4.283	1	no	4.535	1	no
11	11	1950	4.135	2	no	4.392	2	no
11	12	1950	4.095	3	no	4.351	4	no
15	9	1950	4.093	4	no	4.355	3	no
3	12	1952	4.434	1	no	-0.112		yes
5	9	1952	4.120	2	no	4.385	1	no
29	3	1952	4.079	3	no	4.336	2	no
6	9	1952	4.064	4	no	4.325	3	no
4	4	1954	4.410	1	no	0.092		yes
12	11	1954	4.352	2	no	-0.241		yes
14	10	1954	4.189	3	no	4.458	1	no
9	3	1954	4.152	4	no	4.407	2	no

If the synthetic generation approach to the study had been adopted, this problem would not have arisen since smooth density functions would have been fitted to the river flow and Thames level data series. However, as discussed in Section 1, the scale of work required to undertake a full synthetic generation study was not felt to be appropriate.

It is clear that the effect of sea level rise on flood risk is not straight forward. For some years the annual maxima will increase for others it will decrease. The effect on water levels of specific return periods will depend on the combination of these increases and decreases.

At Redbridge (section 1048) where sea level changes have no effect, since the section is upstream of any tidal influence.

In summary it seems likely that increases in sea level by 2030 will result in increased flood risk in the lower Roding by up to around 130 mm, for return periods of less than 10 years, but the historical data do not allow precise definition of this increase for higher return periods.

15. Flood defence protection levels

Table 15.1 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

Table 15.1 *Return periods of defence levels for various scenarios*

Ref. number	West bank Defence level (m AOD)	1992 Barrier closure 4.87	2030 Barrier closure 4.87	1992 Barrier Closure 4.67	East bank Defence level (m AOD)	1992 Barrier closure 4.87	2030 Barrier closure 4.87	1992 Barrier closure 4.67
1001u	7.18	> 1000	> 1000	> 1000	5.68	> 1000	> 1000	> 1000
1006	5.37	> 1000	> 1000	> 1000	5.79	> 1000	> 1000	> 1000
1012	9.30	> 1000	> 1000	> 1000	5.5	> 1000	> 1000	> 1000
1017d	5.50	> 1000	> 1000	> 1000	5.47	> 1000	> 1000	> 1000
1017u	5.55	> 1000	> 1000	> 1000	5.81	> 1000	> 1000	> 1000
1023b	5.09	> 1000	> 1000	> 1000	5.22	> 1000	> 1000	> 1000
1031	6.40	> 1000	> 1000	> 1000	5.80	> 1000	> 1000	> 1000
1041	6.70	270	?	450	6.84	590	?	> 1000
1048	7.45	140	140	140	7.11	25	25	25

of water reaching this level taken from the frequency analysis for the various scenarios used to compile Tables 12.1, 13.1 and 14.1. It can be seen that for all scenarios, at 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.

At section 1041 the level of protection appears to be around 270 years, on the west bank and 590 years on the east bank whilst for section 1041, it is only 140 and 25 years for the west and east banks respectively.

For the flood defences which have a standard of service of greater than 1000 years, this is not sensitive to closure of the barriers at a lower level of 4.67 m. Likewise, for rises in sea level of around 130 mm predicted for the year 2030, the 1000 year level of protection remains.

At section 1041, the protection levels would be increased to 450 and more than 1000 years return period respectively for the west and east banks. This level of protection is not sensitive to changes in the barrier operation. However the available data make it difficult to assess the effect of sea-level rise at this section, but a slight lowering of the protection level (increase in flood risk) is likely to occur.

At section 1048, changes to the barrier operation and rises in sea-level have not effect on the flood defence protection levels, since it is largely unaffected by tides.

16. Summary and conclusions

As part of review of flood defences in the Thames estuary, the National Rivers Authority required an assessment of the probability of water levels up to a return period of 1000 years along the River Roding, from Redbridge to its mouth. Water levels 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. The barrier is closed when the Thames barrier is closed and can only be opened when the water levels either side equalise.

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

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.

A hydraulic model was constructed to provide structure functions relating water levels along the study reach to given fluvial flow, effluent discharges and water levels at the mouth. The model allowed water to spill overbank into a flood-plain of infinite storage capacity. The modelling exercise indicated that if high flows in the Roding occur when the barrier closure is closed, water levels upstream of the barrier may exceed those on in the Thames, preventing barrier opening and thus leading to flooding along the lower reaches 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 a series of annual maximum water levels at nine critical cross-sections.

For each cross section a statistical distribution was 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 combination of barrier closure and high Roding flows did not occur, thus its return period could not be assessed.

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

At section 1041 the level of protection appears to be around 270 years, on the west bank and 590 years on the east bank. At section 1041 the protection levels are 140 and 25 years for the west and east banks respectively.

The modelling exercise was repeated assuming that the barrier would close when the Thames reached 4.67 m at London Bridge rather than 4.87 m. For cross-sections 1001 up to 1031, the level of protection remained greater than 1000 years. At section 1041 the level of protection increased to 450 years for the west bank and greater than 1000 years for the east bank. Protection levels at 1048 were not affected.

Additional analysis was undertaken to consider the effects of increases in sea level of 6 mm yr^{-1} predicted for the year 2030. It was concluded that increases in sea level will result in increased flood levels in the Lower Roding by up to 130 mm for return periods less than 10 years, but historical data do not allow precise definition of increases for higher return periods. For sections 1001 to 1031 the defence levels remain in excess of 1000 years. At section 1041 the available data make it difficult to assess the effect of sea level change, but a slight increase in flood risk is likely. Increases in sea-level are likely to have no influence at section 1048 as this is not affected by tides.

17. Recommendations

1: 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.

2: flood plain storage should evaluated to permit more confident extrapolation of water levels beyond bank top height.

3: 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.

18. References

Boorman, D B (1985) *A review of the Flood Studies Report rainfall runoff model parameter estimation equations*. Institute of Hydrology Report No 94.

Houghton, J.T., Jenkins, G.J. and Ephraumas, J.J. (1990) *Climate change - The IPCC Scientific Assessment*. Cambridge University Press.

Institute of Hydrology (1988) *Hydrological data UK: 1987 Yearbook*. Institute of Hydrology.

Institute of Hydrology (1990) *Hydrological data UK: 1989 Yearbook*. Institute of Hydrology.

National Rivers Authority (1991) *Climate change - sea level rise and isostatic variations*. Policy Implementation Guidance Note TE/FD/001. NRA, Bristol

Natural Environment Research Council (1975) *Flood Studies Report* NERC, London.

Thames Water (1988) *Tidal Thames defence levels*. Final Report stages 1 to 3, Thames Water.

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 study reach on the Roding

Table B.1 Roding section 1001 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	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 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	2.020	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 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	2.200	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 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	4.306	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 (barrier closure at 4.87 m)

	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 Annual maximum water levels (barrier closure at 4.67 m)

	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	4.393	4.397	4.415	4.432	4.420	4.427	4.455	5.157	6.064
1956	4.098	4.095	4.105	4.121	4.121	4.131	4.161	5.090	5.990
1957	4.300	4.302	4.315	4.331	4.336	4.352	4.413	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	4.206	4.205	4.219	4.235	4.231	4.239	4.267	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	4.269	4.271	4.283	4.299	4.300	4.314	4.382	5.489	6.401
1967	4.432	4.434	4.451	4.465	4.501	4.544	4.665	5.454	6.267
1968	4.289	4.289	4.302	4.317	4.331	4.355	4.433	5.754	6.674
1969	4.392	4.397	4.415	4.431	4.420	4.427	4.455	5.556	6.486
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	4.286	4.287	4.303	4.319	4.312	4.319	4.344	4.715	5.611
1974	4.225	4.224	4.234	4.249	4.272	4.355	5.090	6.871	7.746
1975	4.323	4.322	4.336	4.350	4.382	4.420	4.534	5.549	6.471
1976	4.372	4.377	4.393	4.410	4.402	4.410	4.446	5.365	6.276
1977	4.382	4.368	4.404	4.420	4.411	4.418	4.599	5.683	6.569
1978	4.307	4.307	4.320	4.335	4.357	4.386	4.480	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	4.336	4.336	4.350	4.365	4.402	4.447	4.572	5.530	6.443
1982	4.309	4.312	4.326	4.342	4.339	4.349	4.462	5.977	6.910
1983	4.308	4.310	4.323	4.339	4.347	4.367	4.435	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	4.430	4.435	4.453	4.470	4.460	4.467	4.602	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 E Annual maximum water levels (predicted for year 2030)

	1001u	1006u	1012	1017d	1017u	1023bu	1031u	1041u	1048
1950	4.528	4.535	4.554	4.570	4.571	4.584	4.644	5.166	6.068
1951	4.374	4.378	4.395	4.411	4.416	4.437	4.511	5.522	6.434
1952	4.381	4.385	4.403	4.419	4.409	4.416	4.446	5.161	6.063
1953	4.454	4.460	4.480	4.496	4.483	4.490	4.520	5.417	6.325
1954	4.452	4.458	4.477	4.494	4.481	4.487	4.516	5.258	6.149
1955	4.663	4.675	4.698	4.715	4.695	4.701	4.736	5.162	6.064
1956	4.351	4.354	4.370	4.387	4.380	4.388	4.424	5.103	5.993
1957	4.546	4.555	4.573	4.590	4.589	4.601	4.660	5.161	6.020
1958	4.531	4.539	4.558	4.574	4.575	4.588	4.648	5.590	6.463
1959	4.491	4.495	4.513	4.530	4.563	4.608	4.731	5.712	6.604
1960	4.461	4.468	4.486	4.503	4.496	4.509	4.566	5.761	6.679
1961	4.454	4.460	4.479	4.495	4.484	4.491	4.524	5.224	6.127
1962	4.552	4.561	4.581	4.598	4.597	4.612	4.677	5.321	6.225
1963	4.462	4.469	4.487	4.504	4.495	4.551	4.740	5.738	6.603
1964	4.452	4.458	4.477	4.493	4.481	4.487	4.638	6.001	6.928
1965	4.486	4.493	4.512	4.529	4.519	4.526	4.565	5.567	6.452
1966	4.519	4.527	4.546	4.563	4.558	4.568	4.627	5.492	6.402
1967	4.620	4.631	4.654	4.671	4.654	4.659	4.696	5.323	6.233
1968	4.535	4.542	4.560	4.576	4.583	4.601	4.672	5.768	6.678
1969	4.663	4.675	4.699	4.716	4.696	4.701	4.737	5.565	6.488
1970	4.540	4.548	4.569	4.586	4.571	4.577	4.612	5.381	6.291
1971	4.520	4.528	4.548	4.565	4.555	4.562	4.605	5.333	6.234
1972	4.593	4.603	4.625	4.642	4.625	4.630	4.664	5.191	6.093
1973	4.557	4.566	4.587	4.604	4.588	4.594	4.626	4.775	5.611
1974	4.473	4.477	4.494	4.509	4.525	4.548	5.111	6.874	7.747
1975	4.573	4.582	4.603	4.620	4.605	4.611	4.648	5.553	6.471
1976	4.655	4.667	4.690	4.707	4.692	4.698	4.740	5.366	6.276
1977	4.513	4.521	4.540	4.557	4.547	4.579	4.758	5.723	6.580
1978	4.552	4.561	4.580	4.597	4.594	4.605	4.662	5.822	6.744
1979	4.538	4.547	4.567	4.584	4.570	4.603	4.722	5.672	6.573
1980	4.536	4.544	4.565	4.582	4.567	4.573	4.607	5.360	6.211
1981	4.571	4.580	4.602	4.619	4.604	4.610	4.682	5.532	6.443
1982	4.530	4.539	4.558	4.575	4.569	4.577	4.626	5.981	6.911
1983	4.597	4.606	4.626	4.643	4.644	4.658	4.721	5.441	6.350
1984	4.425	4.428	4.444	4.459	4.477	4.503	4.592	5.253	6.072
1985	4.525	4.533	4.553	4.570	4.559	4.565	4.701	5.620	6.467
1986	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
1987	4.487	4.491	4.508	4.522	4.544	4.571	4.664	5.858	6.770
1988	4.641	4.652	4.675	4.692	4.676	4.682	4.768	5.981	6.921
1989	4.680	4.692	4.716	4.733	4.712	4.717	4.752	5.493	6.410
1990	4.555	4.564	4.584	4.601	4.594	4.601	4.651	5.964	6.898
1991	4.361	4.365	4.379	4.396	4.395	4.406	4.458	4.816	5.719

Annex F Barrier closures

Table F1. Barrier closure dates including sensitivity analysis

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
Barriers close if Tower Pier ≥ 4.67 (sensitivity analysis)						
28 11 1951	3.785	203.0	4.928	4.654	1.736	-0.639
31 12 1951	3.607	205.0	4.674	4.508	4.284	-0.115
1 2 1953	4.610	72.3	5.410	5.390	2.128	-0.024
11 1 1955	3.719	112.0	4.785	4.600	4.396	-0.008
26 9 1957	3.658	61.7	4.755	4.552	0.752	3.240
21 3 1961	3.901	87.4	4.846	4.751	0.612	2.699
10 12 1965	4.145	222.0	5.243	4.958	22.58	2.706
16 9 1966	3.780	24.9	4.816	4.658	0.386	3.525
1 3 1967	3.551	296.0	4.694	4.469	11.08	0.996
5 10 1967	3.780	56.1	4.785	4.655	0.637	3.263
21 12 1968	3.612	234.0	4.694	4.514	10.12	0.890
23 12 1968	3.536	306.0	4.694	4.458	9.330	0.757
25 11 1973	3.780	8.4	4.740	4.660	0.431	3.676
14 12 1973	3.917	11.0	4.770	4.777	0.594	3.497
29 1 1975	3.620	325.0	4.840	4.523	15.36	1.654
22 1 1976	3.800	19.5	4.720	4.676	0.615	2.196
13 11 1977	3.820	57.2	4.820	4.688	0.833	3.734
12 1 1978	4.210	225.0	5.120	5.016	21.20	2.643
12 1 1978	3.700	225.0	4.730	4.584	21.20	2.454
17 9 1978	3.690	14.9	4.690	4.582	0.38	3.223
31 12 1978	3.500	139.0	4.710	4.420	4.797	0.025
31 12 1978	4.030	139.0	5.210	4.857	4.797	0.187
14 11 1981	3.700	36.4	4.730	4.589	0.615	2.524
2 2 1983	3.700	179.0	4.824	4.584	4.60	0.096
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