EASTBOURNE BOROUGH COUNCIL

EASTBOURNE PARK DISTRICT PLAN -

REVIEW OF HYDROLOGY AND HYDRAULICS

MAY 1987

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EASTBOURNE PARK DISTRICT PLAN

REVIEW OF HYDROLOGY AND HYDRAULICS

CONTENTS

APPENDICES

1. TERMS OF REFERENCE

1.1 The following report contains an appraisal of the hydrological assumptions and policies which are summarised in a document entitled 'Eastbourne Park District Plan - Policies and Proposals' dated May 1985, hereafter referred to as the EPDP report (Ref1).

1.2 The original Terms of Reference as contained in your letter to us dated 22 July 1986 stated that:-

> "The following subjects are to be checked by the consultants to form the basis of a report:-
>
> 1. The assumptions used in compiling the hydrological model.
> 2. The estimate of the maximum flood storage required.
> 3. The feasibility of the distribution of the flood storage areas within the Park in the form of lakes and water meadows.
> 4. The suggested maintained normal water level in the lakes and rivers in the four sectors of the Park.
> 5. To confirm that all the hydrological policies incorporated in the Plan are sound and that they will together alleviate and prevent further flooding.
> 6. That the 2m deep lakes can be physically constructed at reasonable expense, that they will retain their water at all times and will not silt up or the banks will not suffer from excessive erosion.
> 7. That the lakes will be suitable for the water recreational uses suggested.
> 8. That the mounding of the excavated material from the lakes and its distribution will have no adverse affect on the lakes and water meadows.
> 9. The suitability of the excavated material for the growing of trees.

10. Suggest ways in which the lakes can be connected to the rivers and the method of control, especially those in the Shinewater Sector.

11. Confirm that the lakes in the Shinewater sector can be constructed in isolation to provide flood storage for the North Langney Area".

1.3 With our letter dated 11 August 1986 we submitted proposals for meeting the Terms of Reference listed above. Our proposal was accepted by the council in early October with the proviso that we were to carry out the hydrological aspects of the study in collaboration with the Institute of Hydrology and would produce an agreed joint report on the hydrology of Willingdon Levels.

1.4 We had discussions with Dr. Reed of the Institute of Hydrology and confirmed in principle that we would work together to provide an agreed joint view of the soundness of the hydrological assumptions and concepts underlying in the Eastbourne Park District Plan. However the Institute of Hydrology felt that to reach agreement on the practicality of the proposed flood alleviation scheme it was necessary not just to check the previous hydrological calculations but to carry out a more rigorous hydrological appraisal.

1.5 In your letter dated 10 October 1986 you agreed to the approach to the combined study by the Institute of Hydrology and ourselves which we set out in our letter to you dated 9 October 1986.

2. SUMMARY AND CONCLUSIONS

Scheme proposed in the EPDP report

2.1 The Eastbourne Park District Plan (EPDP) proposes the development of the marshland core of the town to provide a new area of recreational open space. The development of the marshland core, which is known as Willingdon Levels, is to be achieved by storing that portion of the catchment flood runoff which is in excess of the outlet capacity of the system in a series of amenity lakes.

2.2 The EPDP sets the flood standard that the maximum water level in Park area must not exceed 2.0 m OD in a 100 year flood. In the EPDP report, which provides details of the proposed development it is considered that this standard can be achieved by the provision of 720 Ml of storage. This finding stems from assumptions/calculations that:-

(1) the critical 100 year flood runoff results from a storm of 9 hours duration; and,

(2) the critical 100 year flood runoff to Willingdon Levels exceeds the effective outlet capacity of West Langney Sewer only for one tidal cycle. During this flood the sewer would discharge at a rate of 7.75 m^3/s for a 6 hour low tide period. There would be no discharge during the remainder of the tide cycle.

Conclusions of review study

2.3 In principle, the concept of reducing maximum flood levels on Willingdon Levels by providing additional storage and/or outlet capacity is sound. However the viability of a scheme such as the one proposed in the EPDP report is critically dependent on the interrelationships between flood runoff, outlet capacity and maximum storage requirement during the design event. As the effective outlet capacity decreases, the design storm duration, the flood runoff volume and the maximum storage requirement increase. The hydrology and hydraulic calculations (see Ref. 2), which underlie the development scheme set out in the EPDP report, underestimate the critical 100 year design storm duration and associated flood runoff volume. The net result is that the maximum storage requirement to maintain maximum flood levels below 2.0 m OD has also been significantly underestimated.

2.4 The critical storm duration and hence the shape of the critical 100 year flood hydrograph for the proposed flood alleviation scheme is dependent on the characteristics of the Willingdon Levels catchment, the net outlet capacity and the flood storage volume. The available data show that the critical storm duration, with a flood alleviation scheme as outlined in the EPDP report, is at least 31 hours and quite possibly 53 hours or longer. The storage needed to limit the maximum flood level to 2.0 m OD during the critical 100 year flood would be about:-

(1) 940 Ml if the average effective outlet capacity over a tidal cycle were $7m^3/s$; and,

(2) 1520 Ml if the average effective outlet capacity over a tidal cyle were only 3 m^3/s.

2.5 Runoff to Willingdon Levels drains to the sea via West Langney and Crumbles Sewers. The EPDP report calculations overestimate the effective outlet capacity of the West Langney Sewer primarily because:-

(1) the adopted 1 in 100 year water levels at Fence Bridge, where the sewer discharges into Pevensey Haven, are too low; and,

(2) no account was taken of the impact of inflows to the sewer from Mountney Level.

2.6 The effects of inflows from Mountney Level and the likely higher flood water levels in Pevensey Haven probably serve to reduce the average effective outlet capacity of West Langney Sewer to between 1.0

and 2.0 m^3/s during critical 100 year flood. A preliminary appraisal of the capacity of Crumbles Sewer revealed that it might discharge up to 1.5m^3/s from Willingdon Levels during floods. Together these two sewers can probably provide an average outlet capacity over a tidal cycle of between about 2.0 and 3.5 m^3/s from the Willingdon Levels during the 1 in 100 year design flood.

2.7 With the existing outlet capacity there is insufficient storage available in the lakes shown on the Proposals Map of the EPDP report given the proposed maximum flood and normal retention levels. Therefore to design a workable scheme it will be necessary to consider:-

(1) the provision of additional outlet capacity;

(2) relaxing the design standards and allowing higher maximum water levels on Willingdon Levels or reducing the return period of the design flood;

(3) adopting lower normal retention levels in the storage lakes; and

(4) constructing additional storage lakes.

2.8 Although a combination of measures are likely to be required, the provision of additional outlet capacity from Willingdon Levels is essential. Of the possible alternatives listed in Section 7:-

(1) the construction of a land drainage pumping station and associated main drains; or

(2) the provision of a new gravity outlet to the sea near Langney Point (and possibly via the proposed Crumbles Marina);

would appear to be the most viable alternatives. However, each would need further detailed study to establish their feasibility.

2.9 The hydraulic structures required to convey flows through the Willingdon Levels will need to be relatively large to minimize head losses and maximize the use of the available storage. There would need to be a large channel to convey flood water across the Shinewater and West Langney Sectors. There would be little benefit in transferring flood water from lakes in the West Langney and Shinewater Sectors to the lakes proposed for the Broadwater and Southbourne Sectors in EPDP report. The weirs and culverts linking the main channels and the lakes and also interlinking the different lakes will need to be carefully designed so that the lake system operates in the most effective manner. Section 8 contains an indication of the types of structures required. The design of these structures cannot proceed until the overall parameters defining the design of the lake systems have been satisfactorily established.

2.10 The lakes in the Shinewater Sector could not be constructed in isolation to alleviate 'primary' flooding of the surrounding built up area during major floods. The lakes might possibly be used to alleviate any secondary flooding in minor storms which stems from the existing high water levels in the receiving channels of the Willingdon Levels. However if the lakes were to be used in this way, it is likely that water quality problems would constrain the range of possible recreational uses.

2.11 Siltation within the lake system is not likely to be significant except in the immediate vicinity of the points where water can enter the lakes from the main channels during floods. These areas might require dredging very occasionally. Trash booms might also be required around these areas to contain debris and possibly oil slicks. If an additional outlet was provided into the proposed Crumbles Marina, its effect on siltation and water quality in the Marina would need to be assessed. However with careful design (and reasonable cooperation between the relevant authorities) we do not anticipate that such an outfall would cause any insurmountable problems.

2.12 The limited load bearing capability of the ground will mean that the proposed flood storage lakes will have to be excavated from their rims and haulage roads will need to be built. In spite of these constraints, we see no reason why conventional methods cannot be adopted. Although earth moving costs will be above minimum rates we would not expect the overall excavation costs to be unacceptable.

2.13 The available data suggest that the lakes will retain their water. Should any local discontinuities be revealed during excavation there is ample material available to construct a clay blanket. However, particular care will be needed with the design of the lake edges to prevent:-

(1) erosion and slumping; and

(2) the water in the lakes becoming discoloured by clay particles.

2.14 The available data suggest that the landfill operations have had little adverse effect on the quality of the surface and groundwater within Willingdon Levels. Continued monitoring of the landfill sites is required, but even if traces of leachate are detected, remedial action to protect the proposed lakes would be feasible at reasonable cost.

2.15 The use of the subsoils to construct landscape mounds will not have a detrimental affect on the water quality of the proposed lakes provided the mounds are adequately drained. It will also be possible to establish a wide range of tree species on the mounds.

2.16 Algae and macrophyte growth are likely to be a problem in the proposed lakes because of their shallow depth, long residence times and nutrient loadings. In most years water quality in the lakes will deteriorate during the summer months and it will be necessary to install an appropriate mechanical aeration and recirculation system. Even with such a system it is possible that storm runoff from the urban areas will on occasions cause a sudden deterioration in water quality.

3. DEFINITION OF THE STUDY AREA AND FLOOD PROBLEM

The Park

3.1 The Eastbourne Park District Plan boundary encompasses some 4.9 km^2 of largely agricultural land, much of it acutely low-lying. The typical field height in the Park area - sometimes referred to as the Willingdon Levels - is about 2 mOD. For comparison, the Mean High Water Spring tide is about 3.7 mOD. In this review study, the extent of the Park has been taken from Figure 5 of the EPDP report. The EPDP report divides the Park into four parts: the Shinewater, Broadwater, Southbourne and West Langney Sectors (See Fig.3.1a).

Catchment

3.2 We have assessed the topographic area of the catchment draining to the Park as 27.9km^2.

3.3 Site inspection of the north-west part of the catchment failed to confirm whether the downland immediately south-west of Folkington drains to the Cuckmere or to the Park; the latter has been assumed. Detailed contour information was not available for Central Eastbourne and the effective southern boundary in flood conditions is unclear. However, it is believed that much of the seafront area drains directly to the sea through sands and gravel. While this area has been excluded, we have allowed for a contribution from the neighbouring Bourne Stream catchment - which is piped into the Park drainage system via the Horsey Sewer. This pipe also receives some local storm water in times of intense rainfall. Other storm water imports (eg, from the Old Town and Downside districts) have been ignored.

3.4 The above findings are in close agreement with the catchment boundaries shown in Figure 4 of the EPDP report and the areas quoted in Section 2.12. of the same report. However, the value of 27.9km^2 is significantly less than the value of 33.59km^2 used to calculate flood runoff to Willingdon Levels in the Hydrology and Hydraulics calculations set out in Appendix A of a document we received from you entitled "Eastbourne Park Landscape Studies" (Ref.2).

Drainage outlets

3.5 There are two drainage outlets from the Park:

1) West Langney Sewer which is an arterial drain leading from Langney Bridge on the southeast margin of the Park, to Fence Bridge some 4km to the east. At Fence Bridge the sewer discharges into Pevensey Haven which in turn discharges to the sea at the Pevensey Bay outfalls, which are located approximately 1.4km downstream of Fence Bridge.

2) Crumbles Sewer which is a carrier of smaller dimensions than West Langney Sewer. Crumbles Sewer leaves the Park at Crumbles Sluice (which is again on the south east margin of the Park) and flows to Crumbles Pond in Princes Park, and thence to the sea.

3.6 Flows in both the Crumbles and the West Langney Sewers are subject to tidelock. A detailed discussion of the outlet capacities of the two sewers is contained in Section 7.

The flood problem

3.7 To determine the nature of flooding in the Park we reviewed the available flood data. The review which is detailed in Appendix A revealed that there are two types of flood problem in the area:-

1) A 'primary' flood problem which occurs when high water levels throughout Willingdon Levels directly threaten widespread flooding of property. Such flooding is generally the result of long duration storms (or a sequence of storms), as exemplified by the events of October 1949, November 1960 and January 1986.

2) Local surface water drainage problems in the periphery of the Park which arise from the limited sizes and gradients of various storm water sewers. These

problems, which usually result from short duration storms of high rainfall intensity are linked to, and exacerbated by, concurrent high water levels in the arterial drainage system.

3.8 The flood alleviation proposals outlined in the EPDP report focus on attempting to provide a solution to the 'primary' flood problem.

Risk of flooding

3.9 In the section of EPDP report which presents the evidence of flooding it states that:-

1. "A flood up to a level of 2.0 metres A.O.D. is not an unusual occurrence on the levels";

2. "Floods of up to 2.3 metres A.O.D. appear to have occurred during the last 25 years"; and,

3. "Though the Southern Water Authority has no records of water entering living accommodation ... it is acknowledged that under certain conditions such an event is possible, and properties with floor levels as low as 2.9 metres A.O.D. could be at risk unless steps are taken to provide adequate flood storage capacity".

3.10 Our historical review of flooding in the Park (see Appendix A) confirmed that, although low lying house gardens near the margins of Willingdon Levels have been flooded, to date there has been no widespread primary flooding of property. This is because historically the available natural flood plain storage has been sufficient to store any flood runoff to Willingdon Levels in excess of the combined capacities of Crumbles and West Langney Sewers.

3.11 It should be noted, however, that no event approaching the severity of a 100 year flood has occurred in recent decades during which time much of the catchment urbanisation has occurred. Each new development adds to the risk of primary flooding by increasing the volume of flood runoff to Willingdon Levels and in some cases also reducing the available natural flood plain storage. It is quite conceivable that should a long duration 100 year flood occur, with the existing level of catchment urbanisation and current outlet arrangements, then widespread primary flooding would result. To confirm this statement it would be necessary to:-

(1) Carry out a detailed topographic survey of the whole of Willingdon Levels and its margins to the standard of the recent 1:500 scale survey of the Shinewater Sector. This would allow the natural flood plain storage below selected levels to be computed.

(2) Adopt the best estimate of the effective outlet capacity from Willingdon Levels (see Section 7).

(3) Compute the 100 year flood runoff for a range of different storm durations and to undertake trial routings to define the critical storm duration (see Section 5.10)

(4) Route the 100 year flood hydrograph based on the critical storm duration through the Willingdon Levels to determine the maximum flood level.

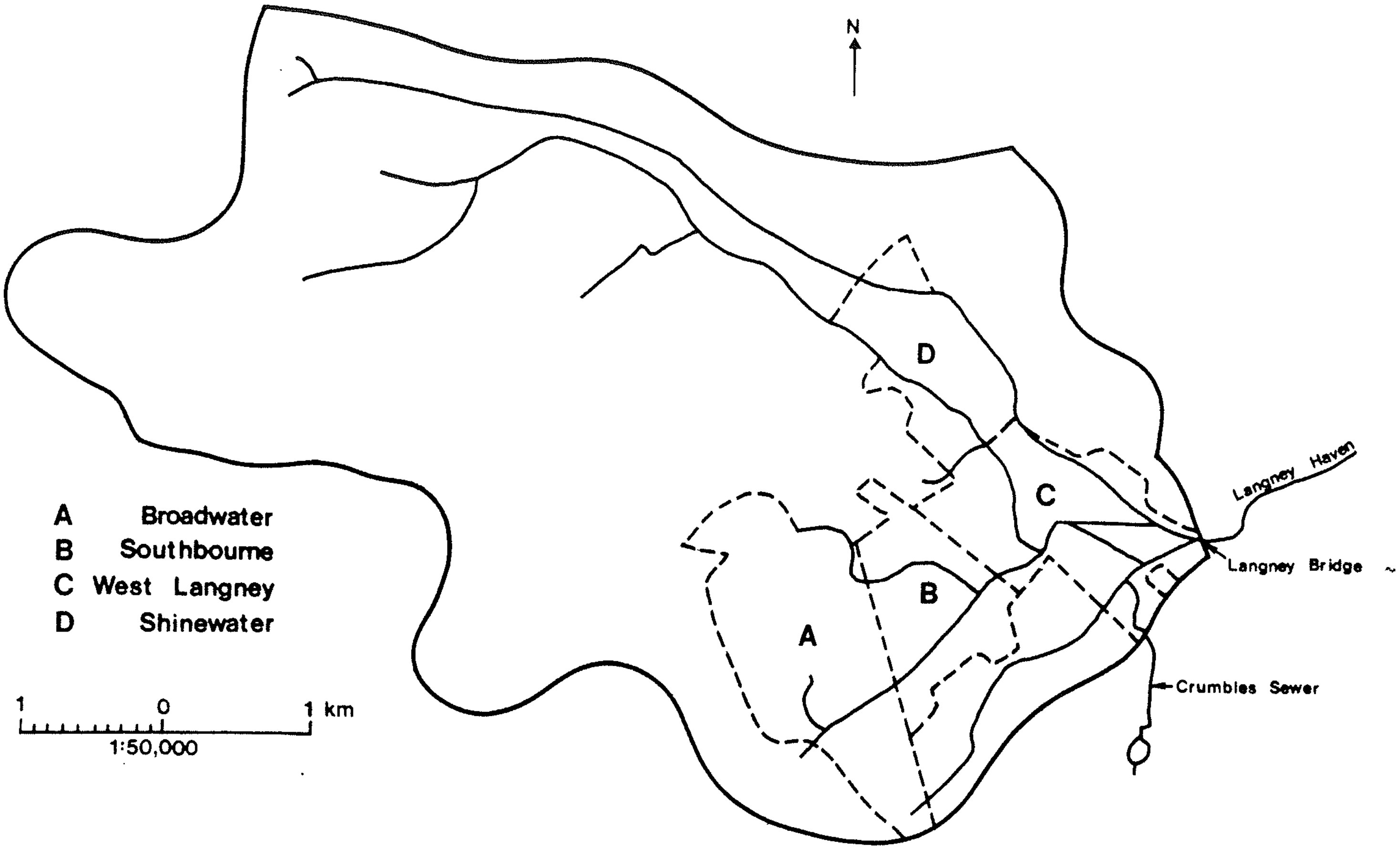


Fig. 3.1a Eastbourne Park catchment - Sectors defined in EPDP report

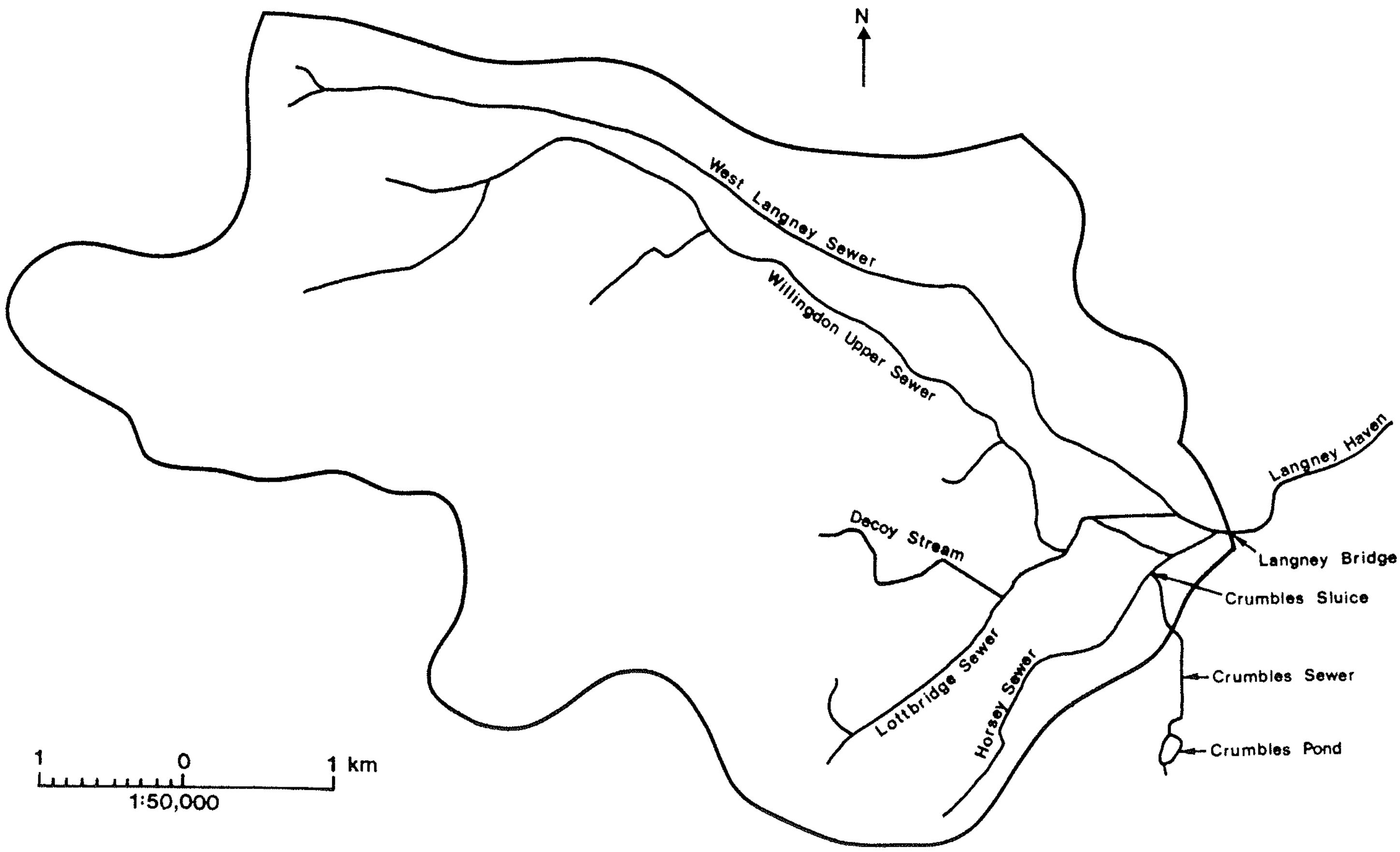


Fig. 3.1b Eastbourne Park catchment - arterial drainage system

4. CATCHMENT INFORMATION

Land use

4.1 The Park itself is a low-lying wetland, partly used for summer grazing, with some slightly higher parts given over to allotments and other uses which can tolerate occasional flooding. The catchment draining to the Park is highly urbanised and includes Willingdon and Hampden Park, most of West Langney, much of Polegate and some northern parts of Central Eastbourne (See Fig.4.1). Urbanisation of the catchment has progressed unceasingly over the last 40 years. Further development is foreseen both in the fringes of the Park and beyond (eg. Polegate). The rural part of the catchment is largely in agricultural use and includes downland and a small amount of woodland. A relatively recent development feature of the Park is the large landfill site off Lottbridge Drove.

Geology/soils

4.2 The underlying geology of the catchment can be summarised in three parts (see Fig.4.2):

(i) permeable areas of the Chalk escarpment of the South Downs and the outcrop of the Lower Greensand (which occurs at the foot of the scarp slope),

(ii) impermeable areas of Gault Clay (outcropping between the Chalk and Lower Greensand) and Weald Clay (in the north-east of the catchment); and,

(iii) low-lying areas of river and marine alluvium. The latter areas are overlain by largely clayey soils which are seasonally waterlogged.

4.3 The urban parts of the catchment are situated primarily on the Gault Clay and Weald Clay but substantial areas between Willingdon and Eastbourne overlay the Chalk, and parts of Polegate are on the Lower Greensand. Development of the low-lying alluvial areas has occurred in several quarters, most notably in North Roselands and West Langney, and around Hampden Park. Hydrogeological mapping of the region indicates that the groundwater catchment is broadly in accord with the topographic boundary.

Topographic surveys of the Park

4.4 A recent high-quality 1:500 topographic survey of the Shinewater Sector was available to the study. From this, 24 fields were identified having land at or below 2.4 mOD. Reference was made to a total of 466 spot heights and area/level and volume/level tables thereby constructed for the Shinewater Sector:

Stratum	Level (mOD)	Area at level (km^2)	Area at or below level (km^2)	Storage (Ml)
1	1.7	0.037	0.037	0
2	1.8	0.137	0.173	10
3	1.9	0.282	0.455	42
4	2.0	0.144	0.599	95
5	2.1	0.155	0.754	162
6	2.2	0.125	0.880	244
7	2.3	0.053	0.934	335
8	2.4	0.014	0.948	429

4.5 Older 1:2500 plans were provided for the Shinewater, Broadwater and Southbourne Sectors but not the West Langney Sector (which is possibly the most low-lying of the four). Comparison of the 1:2500 and 1:500 plans suggests that the former overestimate field levels in the Shinewater Sector by about 0.25 metres. This significant discrepancy casts doubt on the validity of field levels shown on the 1:2500 plans elsewhere in the Park and demands further investigation.

FSR catchment characteristics

4.6 The standard FSR catchment characteristics such as average annual rainfall, mainstream length and slope are well defined on the standard maps and require no specific comment.

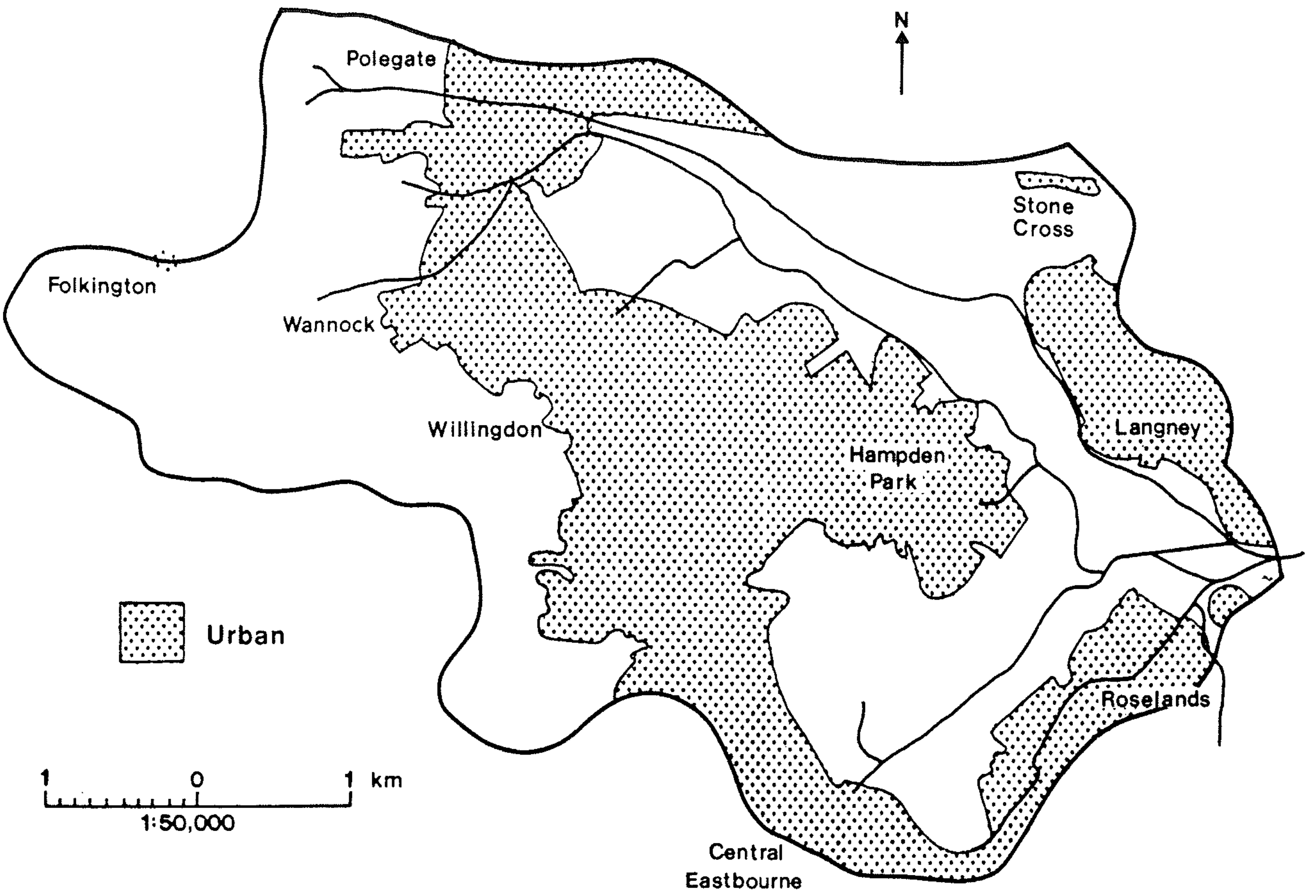


Fig. 4.1 Location of urban areas

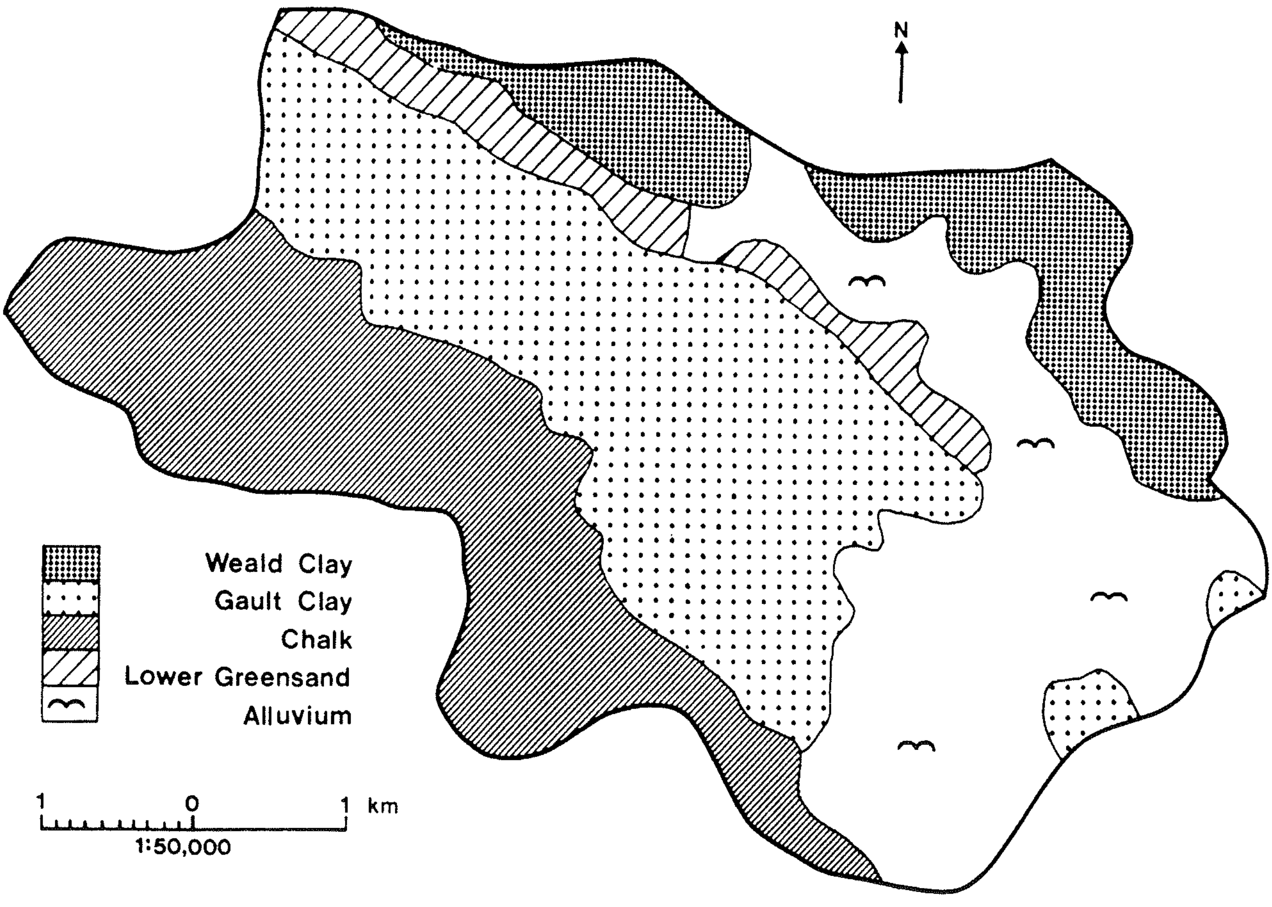


Fig. 4.2 Catchment geology

5. FLOOD ESTIMATES

Summary

5.1 To check the validity of the 100 year flood calculations carried out for the EPDP report we have made an estiamte of the critical 100 year flood runoff to Willingdon Levels using the Flood Studies Report (FSR) rainfall/runoff method, as updated by Flood Studies Supplementary Report No 16 (See Refs 3 and 4). The differences in design storm construction recommended in Flood Studies Supplementary Report No 5 (Ref 5) FSSR 5 for heavily urbanised catchments have not been applied. This is because the Park catchment is sensitive to long duration storms for which the assumption of a relatively 'peaky' summer profile would be inappropriate.

5.2 The critical storm duration, which is dependent both on the available flood storage and the effective outlet capacity, is uncertain but probably lies in the range between 31 and 53 hours. The calculation of 100 year flood hydrographs based on storm durations of 31 and 53 hours are shown in Appendix B.

5.3 The 100 year flood hydrograph based on a 31 hour storm is shown in Figure 5.1. It is of a longer duration and has a significantly larger volume than the event adopted for use in the EPDP report. There are several reasons for this: some methodological and some interpretative. The following paragraphs consider in detail the more important aspects of the flood estimation.

Catchment area

5.4 As outlined in paragraphs 3.2 and 3.3 we consider that an area of 27.9km^2 drains directly to the two outlets from Willingdon Levels (i.e Langney Sewer at Langney Bridge and Crumbles Sewer at Crumbles Sluice). A small additional area drains to the levels via the Bourne Stream and Horsey Sewer (see Fig 4 of the EPDP report). In the hydrology computations for the EPDP report the catchment area draining to Willingdon levels is taken to be 33.59km^2 .

Percentage runoff

5.5 It is often the case that the assessment of percentage runoff to be expected from given soils and land use is the most crucial aspect of flood estimation. Particular care has therefore been taken in assessing the soils and underlying geology of the Park catchment and their interrelationship with urbanisation.

5.6 The basic FSR map of Winter Rainfall Acceptance Potential (WRAP) broadly distinguishes the three parts of the catchment referred to in Section 4.2 and assigns them to WRAP classes 1 (Chalk etc,), 3 (Alluvium) and 4 (Weald Clay) respectively. However, it is known from the analysis of runoff data from catchments located on the Weald Clay that the response is more characteristic of a WRAP class 5 soil. Moreover it is known that the Gault Clay is, if anything, more impermeable than the Weald Clay and therefore also warrants assignment to WRAP class 5.

5.7 The most contentious aspect concerns the ability of the Alluvial Clays of the Park to accept winter rainfall. There are few catchments consisting of such young soils on which runoff response has been assessed scientifically. Moreover where such experiments have been carried out - for example at Newborough Fen in the North Level Internal Drainage Board - it is obvious that the ability of the soils to absorb winter rainfall is only maintained by a rigorous management system incorporating deep drains and pumping stations. This is manifestly not the case in the Park area, where large parts are seasonally waterlogged and local ponding occurs in winter. It is therefore concluded that this area should also be interpreted as WRAP class 5.

Allowance for storage effects

5.8 The hydrology studies carried out for the EPDP report assume that the Willingdon Levels catchment is sensitive to the 100 year flood derived from a 9 hour design storm. Our studies of the available data reveal that in major events, because of the attenuation and delay on runoff caused by the storage of water of the flood plain, the Park catchment is most sensitive to relatively long duration storms (or sequences of storms).

5.9 In the FSR procedure for an unreservoired catchment, the design storm duration (D) is calculated from:

$$D = (1 + SAAR/1000)\ Tp \qquad (1)$$

where SAAR is average annual rainfall (mm) and Tp is a characteristic response time (hr) of the catchment to heavy rainfall. Because the Park catchment is reasonably compact (in stream structure) and heavily urbanised the characteristic response time is only about 5 hours. Application of Equation 1 yields a design storm duration of 9 hours.

5.10 For a reservoired catchment it is necessary to calculate the design storm duration by reference to a characteristic response time which includes the delay imposed by the storage effect. Following the ICE Guide to Floods and Reservoir Safety (Ref 6), the design storm duration is calculated from:

$$D = (1 + SAAR/1000)\ (Tp + RLAG) \qquad (2)$$

where RLAG denotes reservoir lag, the time delay (hr) between the peak inflow to, and outflow from, the reservoir.

5.11 For an impounding reservoir, RLAG is generally estimated from the storage and discharge characteristics of the reservoir. For the Willingdon Levels catchment both the size of the storage, which is provided by the extensive flood plain area upstream of Langney Bridge, and the discharge capacities of the two outlets are uncertain (see Section 7). Thus it has been necessary to estimate RLAG (and hence an appropriate design storm duration from Equation 2) by other means. These are discussed in Section 6.6.

Rainfall on the lakes

5.12 When considering reservoirs with a large surface area, it is appropriate to assume 100% runoff from rain falling directly on the water surface. Because the flood plain is currently only an informal flood storage area - with an extensive lake area only in major floods - rainfall on the lake surfaces has been ignored for simplicity. However at the detailed design stage, rainfall on the reservoir surface will need to be taken into account as 100 ha of water surface will increase the 100 year flood runoff volume by at least 40 to 50 Ml.

Inflows from the Bourne Stream catchment

5.13 The Bourne Stream catchment drains to Willingdon Levels via Horsey Sewer. From the dimensions of the limiting section, we estimate that the carrying capacity of the pipe linking the two areas is about 0.75 m^3/s. We have allowed for the contribution of the Bourne Stream catchment by adding 0.75 m^3/s to the calculated baseflow for the 27.9 km^2 direct catchment to Willingdon Levels.

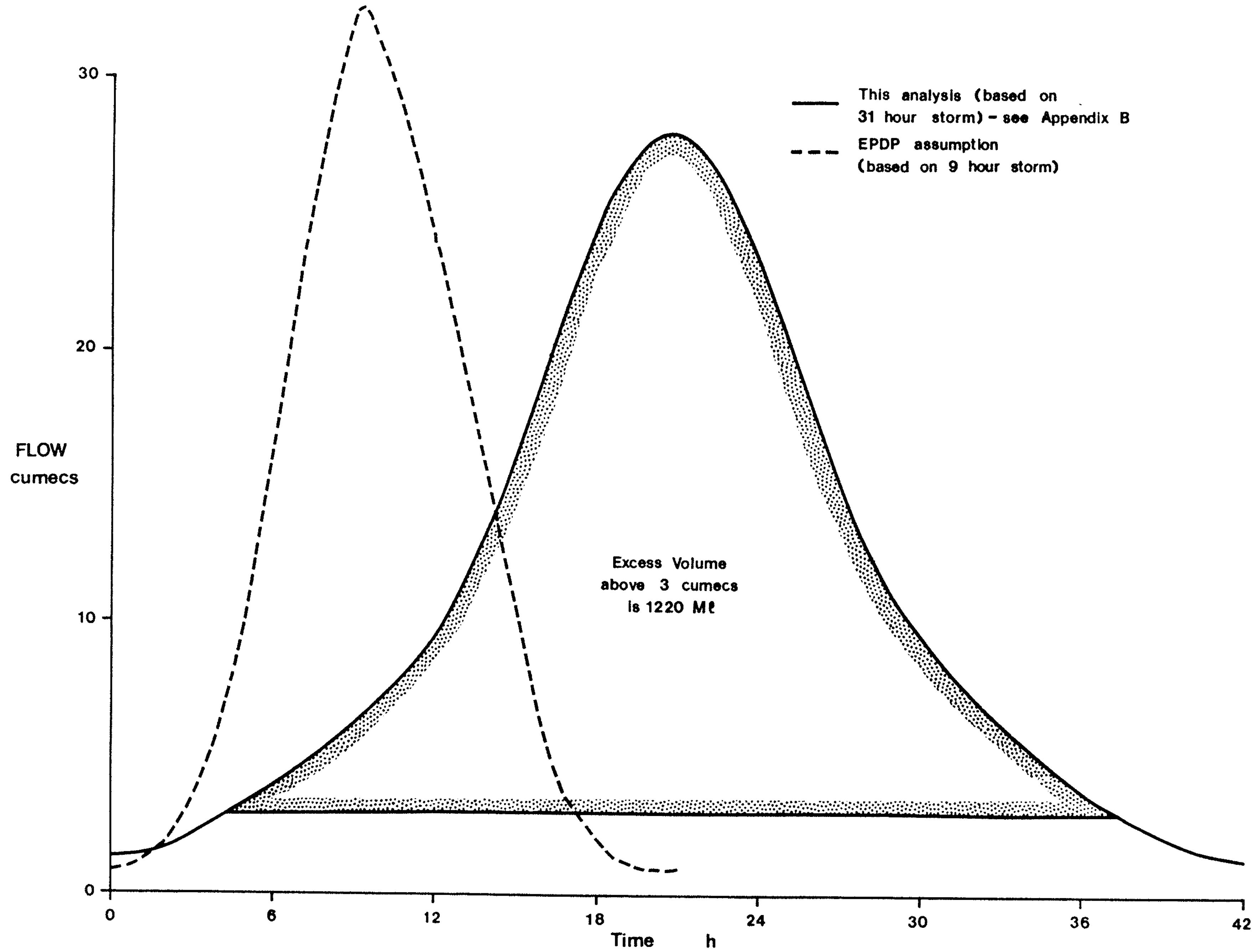


Fig. 5.1 Estimates of 100-year flood hydrograph

Available water level data

6.1 It was thought at the start of the study that the water level data available for the Park area were limited to the daily records for Lottbridge Pumping Station plus spot water level readings at a number of the major sluices within the Levels. However, we were informed by Southern Water that continuous water level readings are available for Langney Sewer at Langney Bridge for the period January 1980 to date. No mention is made of these data in the hydrology or hydraulic studies for the EPDP report.

Langney Bridge water levels

6.2 Figure 6.1 shows Langney Bridge water levels and catchment rainfalls for the period 1-6 January 1986. Catchment rainfall was assessed by reference to four daily raingauges (each located within 7km of the catchment centroid) and a recording raingauge at Wish Valley, Eastbourne.

6.3 Two features are of particular note in Figure 6.1:-

(1) the tidal influence is marked throughout the period of flood runoff; and,

(2) the effective response time of the catchment and flood plain storage to the heavy rainfall, far exceeds the duration of a single high tide. Therefore the degree of synchronization of the runoff peak with the tidal cycle would seem to be relatively unimportant.

Assessment of reservoir lag time

6.4 Based on a hydraulic analysis using the available data it was not possible to define precisely the performance of West Langney Sewer during the flood of 2 to 6 January 1986 (See Section 7). Both the magnitude and timing of outflows from Willingdon Levels could only be assessed within fairly wide limits. However, simply by smoothing over the tidal fluctuation in the Langney Bridge water levels, as shown by the broken line in Figure 6.1, it is possible to get a reasonable estimate of the time of the effective peak discharge from the Park.

6.5 Comparison of the peak discharge time with the centroid of the corresponding period of flood-producing rainfall (see Fig.6) yields a total characteristic response time of about 17 hours, of which 5 hours can be attributed to the catchment response and 12 hours to the delay imposed by the flood plain storage.

6.6 The calculation of the critical design flood hydrograph is strongly influenced by the assumption made about RLAG. It is possible that the 'observed' lag time of 12 hours is typical only of modest flood events and that during an extreme event the delay effect would be rather greater. In this respect, the historical evidence - that the Park catchment is generally sensitive to sequences of heavy storms over a number of days - gives cause for concern that for a 100 year event the critical design storm duration may be longer than 31 hours. To guard against the possibility that 31 hours is too short a storm duration for the critical design flood we have calculated also the 100 year flood runoff to Willingdon Levels from a 53 hour storm (see Appendix B). Such an event would be critical if the lag due to flood plain storage were 24 hours.

Estimate of effective outlet capacity

6.7 A simple water balance approach can be used to obtain a first estimate of the average effective discharge capacity of the two outlets from Willingdon Levels during the flood of 2 to 6 January 1986. It can be seen from Figure 6.1 that most of the flood runoff generated by rainfall periods A and B was discharged within 5 days. As these storms were preceded by heavy rainfall at the end of December, a relatively high percentage runoff would be expected for the event. Taking a conservatively high value of 60% leads to an estimate of 45mm of runoff in 5 days or 9mm/day. For a 27.9km^2 catchment this corresponds to a mean discharge of 2.9 m^3/s.

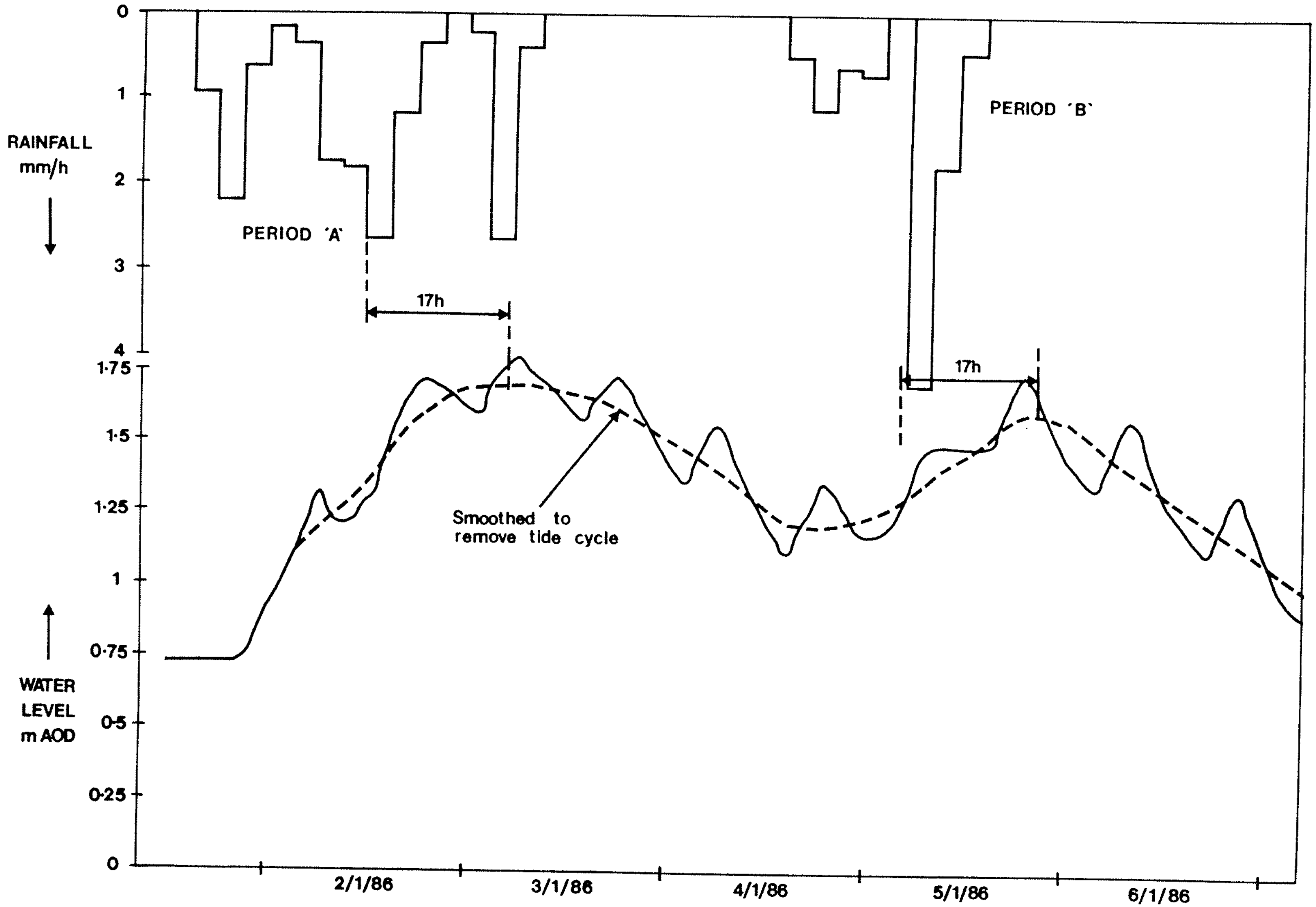


Fig. 6.1 Catchment rainfall and Langney Bridge water level data for January 1986 event

7. OUTLET CAPACITY

Introduction

7.1 Outflow from Willingdon Levels is via Crumbles and/or West Langney Sewers. The work carried out for the EPDP report suggests that during a 100 year flood:-

(1) the average outflow via West Langney Sewer at low tide will be 7.75 m^3/s; and,

(2) there will be no outflow via Crumbles Sewer.

In the following paragraphs we review the calculations and comment on the assumptions that they are based upon.

West Langney Sewer hydraulics

7.2 In the study (Ref 2) carried out for the EPDP report the capacity of West Langney Sewer between Langney Bridge and Fence Bridge (ie across Mountney Level) was estimated using Mannings equation. In this equation:-

(1) the channel parameters were based on the average of nine surveyed cross sections;

(2) a friction factor (n) of 0.035 was adopted; and,

(3) the water surface slope was derived on the basis of a level of 1.9 m OD at Langney Bridge and an everage low water level of 1.4 m OD at Fence Bridge.

7.3 For the most part, the basic approach adopted is reasonable even though it incorporates several simplifying assumptions. The choice of friction factor is realistic and, although we have not been able to obtain a drawing showing details of the surveyed cross sections, we have no reason to doubt the calculation of the channel properties. One small modification that is required is to allow for the loss of kinetic energy as the flow passes into Pevensey Haven via the constricted channel section at Fence Bridge Sluice. This would reduce the sewer discharge, when the water level is at 1.4 mOD, from 7.75 to 7.3 m^3/s.

7.4 Two other factors have a much greater bearing on the effective capacity of West Langney Sewer as a carrier of outflows from Willingdon Levels:-

(1) the actual water surface slope between Langney Bridge and Fence Bridge; and,

(2) the effects of runoff from Mountney Level, which was not mentioned in the EPDP studies. The effect of inflows from this 7.2 km^2 catchment is important because the runoff from Mountney Level will enter West Langney Sewer in preference to the outflow Willingdon Levels.

7.5 In a tide-locked reach, the average discharge over a tidal cycle is very sensitive to variations in the water level at the downstream end of the reach. To improve the estimate of the effective discharge capacity of West Langney Sewer at Langney Bridge and to assess its sensitivity to the water surface slope we have made simplified calculations to allow for the temporal variations in water levels at either end of the reach and for inflows from Mountney Level. In these calculations we have assumed:-

(1) that the channel parameters are as calculated for the EPDP and remain constant over the tidal cycle; and

(2) all the inflow from Mountney Level enters the Sewer halfway between Langney Bridge and Fence Bridge.

7.6 The results of our calculations are summarised on Figure 7.1. The figure shows how sensitive the discharge is both to the level difference between Langney Bridge and Fence Bridge and to the inflow from Mountney Level. For example, if the level difference increases from 0.3 to 0.7m, the discharge would rise from 5.7 to 8.6 m^3/s in the absence of any inflows from Mountney Level. Alternatively, with a head difference of 0.5m, the presence of a 5 m^3/s inflow from Mountney Level would reduce the discharge capacity at Langney Bridge from 7.3 to 4.2 m^3/s.

7.7 During a 100 year flood based on a 31 hour storm we estimate that the average discharge from Mountney Level would be 6.25 m^3/s during the peak 6 hour period and 4.15 m^3/s over the peak 24 hour period. Assuming that the water level difference was 0.5 m, all through the low tide period, the average low tide outflow at Langney Bridge at the height of the design flood would be about 3.2 m^3/s as compared with the value of 7.75 m^3/s assumed in the EPDP report.

7.8 The effect of the water level variations at Fence Bridge throughout the design event are difficult to take account of as there is no information available to us on 1 in 100 year flood conditions. We have attempted to gain some idea of potential conditions, including the period of tidelock, from an analysis of the flood of January 1986.

7.9 The water levels recorded at Langney Bridge and Pevensey Depot on 3 January 1986 are shown on Figure 7.2. This figure also shows the level at Fence Bridge at 9am as recorded by the sluice keeper. The observed value was 0.4m higher than the contemporary level recorded at Pevensey Depot. A possible tide curve for Fence Bridge based on this observed level has been sketched on Figure 7.2. The level records indicate that levels at Pevensey Depot were higher than those at Langney Bridge for a total of 8 hours on 3 January 1986.

7.10 We have assessed the discharge at Langney Bridge for 3 January 1986 using Figure 7.1 on the basis of the observed or inferred water levels and assuming that direct drainage from the catchment downstream of Langney Bridge was half the runoff anticipated in the 100 year design storm. The results shown on Table 7.1 suggest an average discharge at Langney Bridge of between 2.7 and 3.9 m^3/s depending on which water level curve is assumed to apply at Fence Bridge. This estimate allows for the storage of flows draining directly to the Sewer during the high tide period but makes no allowance for any reversal of flow that may have occurred at Fence Bridge. The estimated average daily discharge at Langney Bridge has to be compared with the independent estimate of 2.9 m^3/s for the combined capacities of West Langney and Crumbles Sewers based on flood volume and the time taken to evacuate the flood (See paragraph 6.8). The results suggest that the low water level at Fence Bridge was probably significantly higher than that recorded at Pevensey Depot. This difference would arise from headlosses in Pevensey Haven downstream of Fence Bridge whilst the outfall to the sea at Pevensey was discharging freely at low tide.

7.11 Since the levels at Fence Bridge are not known during a 100 year design flood we have assumed that the levels at both Fence Bridge and Langney Bridge on 3 January 1986 would be raised 0.25m to give a peak level of 2.0 m OD at Langney Bridge. Using these levels, and the 100 year inflows from Mountney Level suggests that the average flow at Langney Bridge during the peak 24 hours would be between 1.1 and 2.2 m^3/s depending on which of the two tide curves assumed for Fence Bridge is the more realistic. This analysis is outlined on Table 7.2. By contrast in the unlikely event that the levels at Fence Bridge are the same as on 3 January 1986, the average daily discharge would rise to between 4.7 and 5.5 m^3/s. Table 7.2 indicates the number of hours of tidelock for each case and the maximum water level difference at low tide.

7.12 The results shown on Table 7.2 indicate the importance of water level differences between Fence Bridge and Langney Bridge. Increasing the difference at low tide will increase the discharge. At high tide, the most important factor is the duration of any tide locked periods when discharge at Fence Bridge is not possible. In summary these results suggest that, if during the critical 100 year flood water levels at Fence Bridge are approximately 0.25 m above those inferred for the 3 January 1986 from the sluice gauge reading, the discharge past Langney Bridge will be severely restricted to a 24 hour average of around 1.1 m^3/s. This value is less than one third the amount (3.9 m^3/s) implied in the EPDP report.

7.13 We have not made any allowances for the transient effects which arise because water levels at both ends of the Sewer are continually changing in response to the level fluctuations in Pevensey Haven nor have we made any allowance for flooding of land downstream of Langney Bridge. These aspects could be analysed using a transient backwater model but we do not think the refinemment is warranted at this stage.

7.14 The height of the sea tides at Pevensey seem likely to have only a small effect on the high water level in Pevensey Haven as the duration of the period when the tidal doors are tide locked will probably be fairly similar whatever the tide range. This aspect could be considered further by examining the tide curves and the level of the tidal doors at Pevensey

Crumbles Sewer hydraulics

7.15 We have examined the available data for the Crumbles Sewer to see if it would be able to increase significantly the outflow from the Willingdon Levels during floods. No outflow was assumed in the EPDP report.

7.16 The sewer discharges to the sea through a 2.3 m square box culvert which is protected by a tidal flap. The culvert invert is set at - 0.03m OD, which will allow free discharge from the Crumbles Sewer whenever sea level is below about + 0.5 m OD, perhaps 7 hours each tide.

7.17 The water level in the sewer channel upstream of the tidal doors during low tide periods will be determined by the head required to pass the discharge through the tide flaps. The flow through the outfall sluice at Princes Park boating lake will also be affected by this level. Overall we estimate that flows of up to about 2 m^3/s can be passed through the boating lake and out to sea at low tide without overtopping either the inlet or the outlet weirs at the boating lake. At flows above about 2.4 m^3/s, the level in the boating lake will exceed the inlet and outlet weir level of 0.86 m OD.

7.18 If discharges through the boating lake are higher, the lake level will rise to allow sufficient flow over the weir. The 11-12m length of this weir limits the rise, so that at a flow of 4 m^3/s the lake level is likely to be 1.1 m OD, just over 0.2 m above the weir level. At higher flows, the level in the lake will be controlled by the backwater from the tidal flap, so that at a flow of 8 m^3/s, the lake level will rise to about 1.6 m OD, about 0.75 m above the weir level.

7.19 The flow entering the Princes Park boating lake is controlled by the Crumbles Sluice on the Willingdon Levels and by the overall capacity of the Crumbles Sewer channel, including the effects of culverts and other structures. From a preliminary assessment of the channel properties we consider the low water discharge capacity of the sewer may be about 3 m^3/s with a level of around + 1.9 m OD in the Willingdon Levels and a level of around 1.1m OD in Princes Park lake. Since discharge can only take place for just over half the tidal period, the sewer may be able to discharge an average of around 1.5 m^3/s with flood levels of + 1.9m OD on the Willingdon Levels. In this assessment we have not made any allowance for runoff which drains directly into the sewer downstream of Willingdon Levels.

7.20 One advantage of discharging through the Crumbles Sewer is that the tailwater level outside the tide flaps is directly related to sea level, which is fairly predictable, and not dependent on flood runoff from another catchment.

7.21 The backing up of water in the sewer at high tide would necessitate flood banks in this sewer that were high enough to contain the peak levels throughout its length. These flood banks may already exist.

Improvements to outlet capacity

7.22 The foregoing analysis of the West Langney and Crumbles Sewers suggests that during a 100 year design flood, the average outflows when the water level on Willingdon Levels is 2.0 m OD will be between 1.0 and 2.0 m^3/s through the West Langney Sewer and upto 1.5

m^3/s through the Crumbles Sewer. The combined average outlet capacity of these two sewers will certainly be less than the average of 3.9 m^3/s assumed for West Langney Sewer in the EPDP report.

7.23 We have attempted to estimate the interrelationships between storage capacity, outlet capacity, flood inflows and peak water levels by routing the design flood through storages of various sizes with a range of outlet capacities. The initial level of the storage was set at 1.36 m OD, the average level of all lakes proposed in the EPDP report.

7.24 The analyses initially used a 100 year flood based on a 31 hour storm, but a check was made using a 100 year flood based on a 53 hour storm as this was found to give higher water levels for outlet capacities of less than 7 m^3/s at a level of 2.0m OD.

7.25 The outlet ratings were based on a pipe running full with a crown at + 1.0m OD which was able to discharge freely. The rating was defined as:

$$Q = q\sqrt{H - 1.0} \quad \text{for values of H above 1.36 m OD}$$

where

(1) q is the 24 hour average discharge in m^3/s with a water level of 2.0 m OD in the Willingdon Levels; and

(2) H is the actual water level in the Willingdon Levels in m OD.

A similar rating could be defined for a free surface discharge through tidal doors. The precise rating for an actual structure would depend on its size, its design and its level.

7.26 The effects of routing the updated 100 year design flood through the 112 ha lake area proposed in the EPDP report is illustrated in Figure 7.3. This demonstrates that the existing 24 hour average combined outlet capacity of around 3.0 m^3/s through West Langney and Crumbles Sewers at 2.0 m OD would cause peak levels in the Willingdon Levels lake system of about 2.6 m OD. In practice somewhat lower levels would be expected because of the flooding of low ground around the lake system. This figure also suggests that the existing outlet capacity would need to be trebled to prevent water rising above 2.0 m OD in Willingdon levels in the design storm.

7.27 The extra outlet capacity required in this situation could not easily be provided by modifications to either the West Langney or Crumbles Sewer channels. An alternative would be large tidal doors or a pumping station discharging into the proposed Crumbles Marina. A pumping station would need to be capable of discharging upto 8 m^3/s during flood periods. Tidal doors would need to be capable of discharging upto 16 m^3/s if they were tide locked for half the time. The size of the doors would depend critically on their design, particularly the period of tidelock, but we anticipate that an effective area of at least $5m^2$ below + 1.0 m OD would be required.

7.28 The provision of additional storage or the relaxation of the design top water level in Willingdon Levels would permit a reduction in the capacity of the outlet structures. The effect is illustrated on Figure 7.4. For example the provision of an additional 230 Ml of storage below 2.0m OD, would reduce the required nominal average outlet capacity from 11 to 7 m^3/s. Allowing for the existing average outlet capacity in the two sewers, this would halve the amount of additional capacity needed from 8 to 4 m^3/s.

7.29 The adoption of lower normal retention levels would also increase the available flood storage and so reduce the required outlet capacity. In addition a lower normal retention level would increase local gradients in the storm water sewers around the margins of the Willingdon Levels and so tend to improve surface water drainage from the urban areas (See Section 3.7).

7.30 Irrespective of whether the outlet capacity of the Willingdon Levels is improved by the provision of a new tidal outlet, or by a land drainage pumping station, appropriately sized main channels will need to be constructed to link Willingdon Levels to the sea. The proposed Crumbles Marina could be utilized for the seaward portion of the channel. Since a pumping station could discharge all through the tide cycle, its required capacity would be about half that necessary with gravity flow tidal doors. The importance of minimizing head losses in a gravity system would necessitate connecting channels that were three or four times the size of those required for a pumping station. These advantages of a pumping station would be offset to some extent by the running costs of the station and the costs necessary to ensure reliable operation of a station that would only be required on rare occasions.

	CASE A	CASE B
HWL at Langney Bridge (mOD)	1.75	1.75
HWL at Fence Bridge (mOD)	1.87	1.87
Period of tidelock (hrs)	8.0	8.0
LWL at Langney Bridge (mOD)	1.45	1.45
LWL at Fence Bridge (mOD)	0.65	1.05
Maximum level difference (m) (Langney Bridge - Fence Bridge)	0.86	0.50
Adopted average inflow from Mountney Level (m^3/s)	2.0	2.0
Adopted peak inflow from Mountney Level (m^3/s)	3.3	3.3
24 hr average discharge at Langney Bridge (m^3/s)	3.9	2.7

Notes

(1) In Case A the Fence Bridge levels are assumed to be the same as the levels at Pevensey Depot.

(2) In Case B the Fence Bridge levels have been inferred from Pevensey Depot levels taking into account an observed spot level at Fence Bridge (see Figure 7.2).

Alternative estimates of average discharge capacity of West Langney Sewer at Langney Bridge on 3 January 1986.

TABLE 7.1

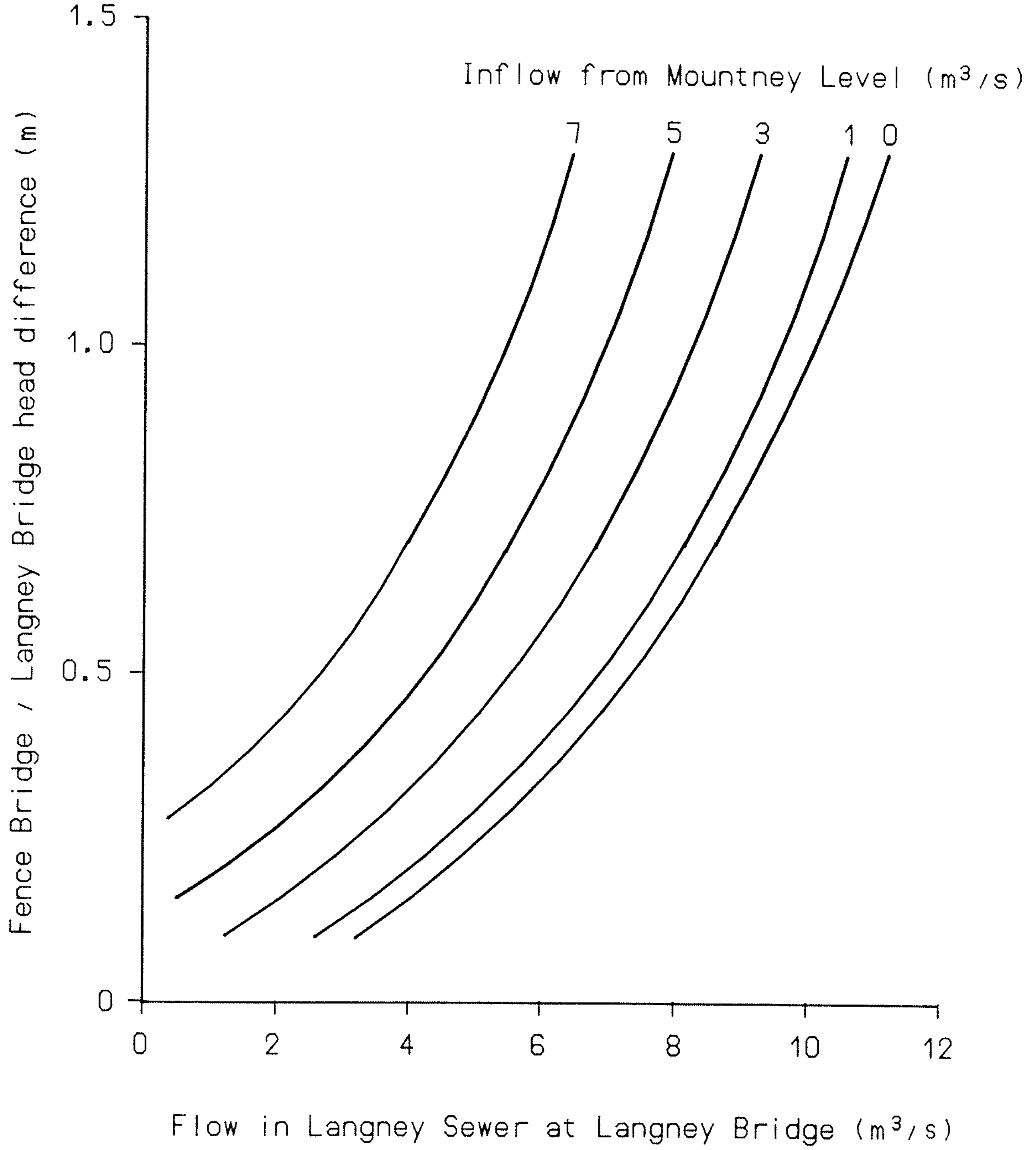
	CASE C	CASE D	CASE E	CASE F
HWL at Langney Bridge (mOD)	2.00	2.00	2.00	2.00
HWL at Fence Bridge (mOD)	2.12	2.12	1.87	1.87
Period of Tidelock (hrs)	8.00	8.00	0.00	0.00
LWL at Langney Bridge (mOD)	1.70	1.70	1.80	1.80
LWL at Fence Bridge (mOD)	0.90	1.30	0.65	1.05
Maximum level difference (m) (Langney Bridge - Fence Bridge)	0.86	0.50	1.20	0.83
Average inflow from Mountney Level (m^3/s)	4.1	4.1	4.1	4.1
Peak inflow from Mountney Level (m^3/s)	6.5	6.5	6.5	6.5
Average discharge at Langney Bridge over 24 hr at height of flood (m^3/s)	2.2	1.1	5.5	4.7

NOTES

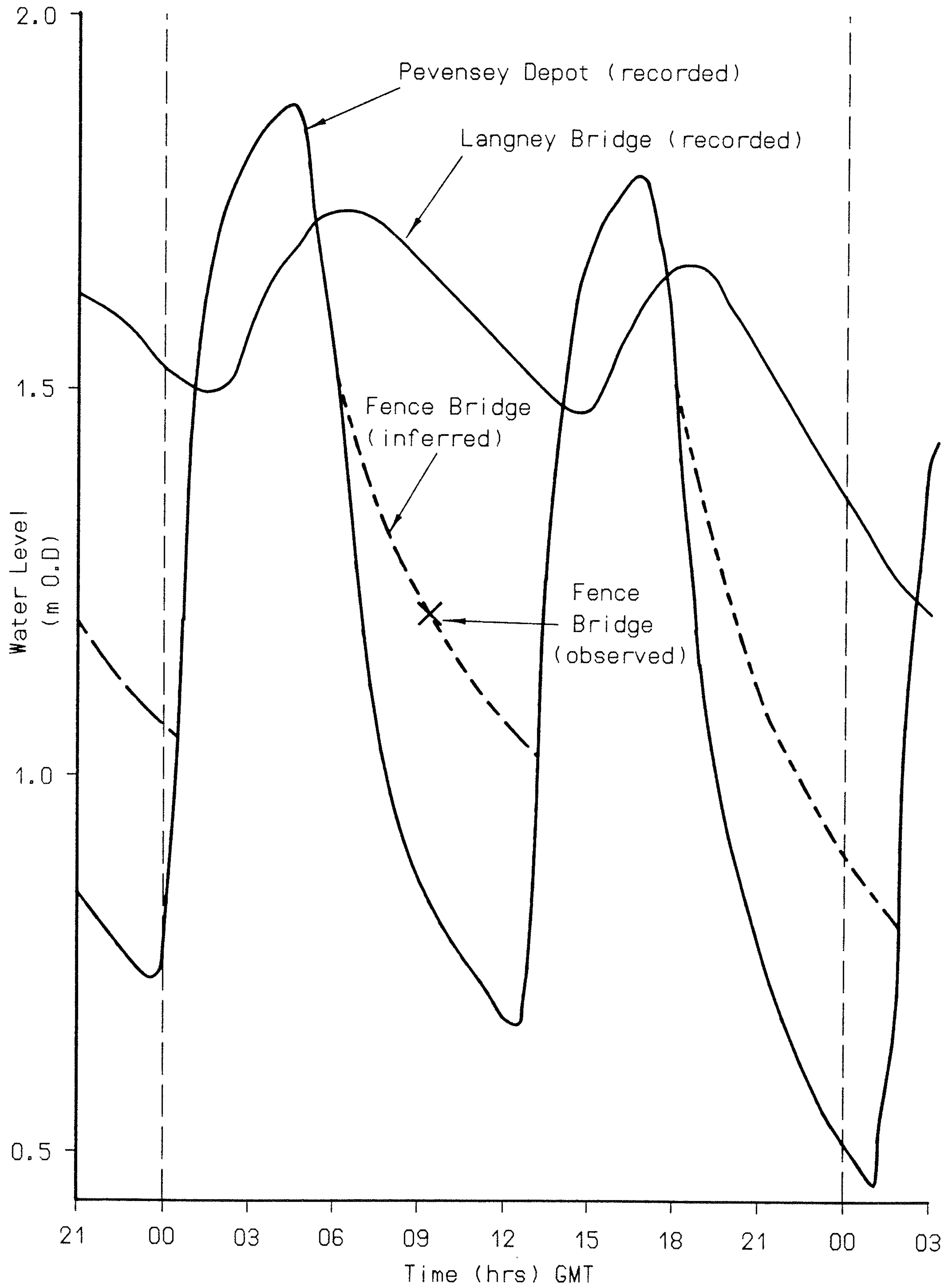
Case C:	Fence Bridge levels:	0.25 m above levels recorded at Pevensey Depot on 3 Jan 1986
	Langney Bridge levels:	0.25 m above levels recorded on 3 Jan 1986
Case D:	Fence Bridge levels:	0.25 m above levels inferred for Fence Bridge on 3 Jan 1986
	Langney Bridge levels:	0.25 m above levels recorded on 3 Jan 1986
Case E:	Fence Bridge levels:	As recorded at Pevensey Depot on 3 Jan 1986
	Langney Bridge levels:	As in EPDP report
Case F:	Fence Bridge levels:	As inferred for Fence Bridge on 3 Jan 1986
	Langney Bridge levels:	As in EPDP report

Estimates of West Langney Sewer discharge at Langney Bridge during a 100 year flood.

TABLE 7.2

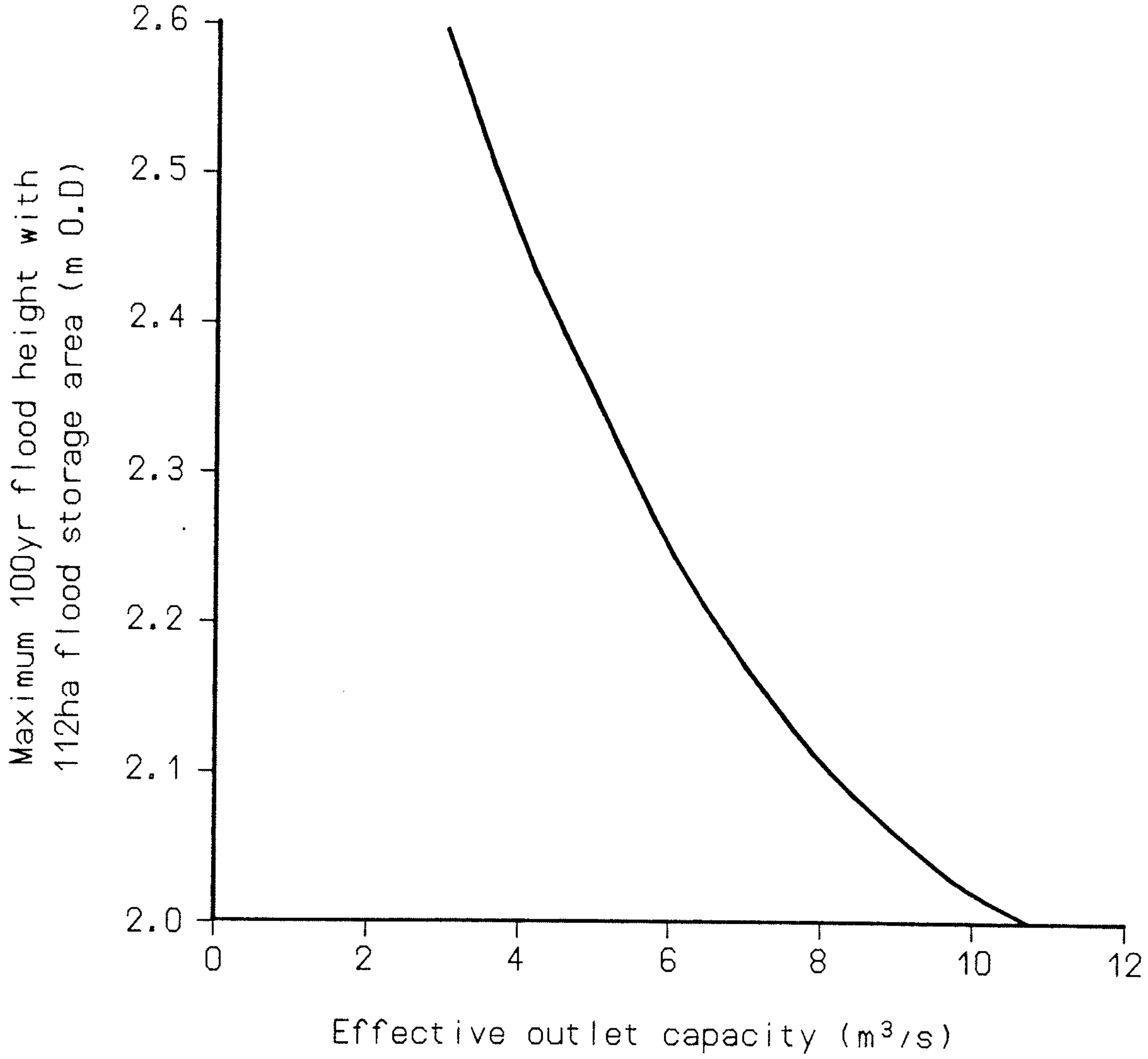


Graph showing calculation of flow in Langney Sewer at Langney Bridge
Figure 7.1



Recorded Levels on 3 January 1986

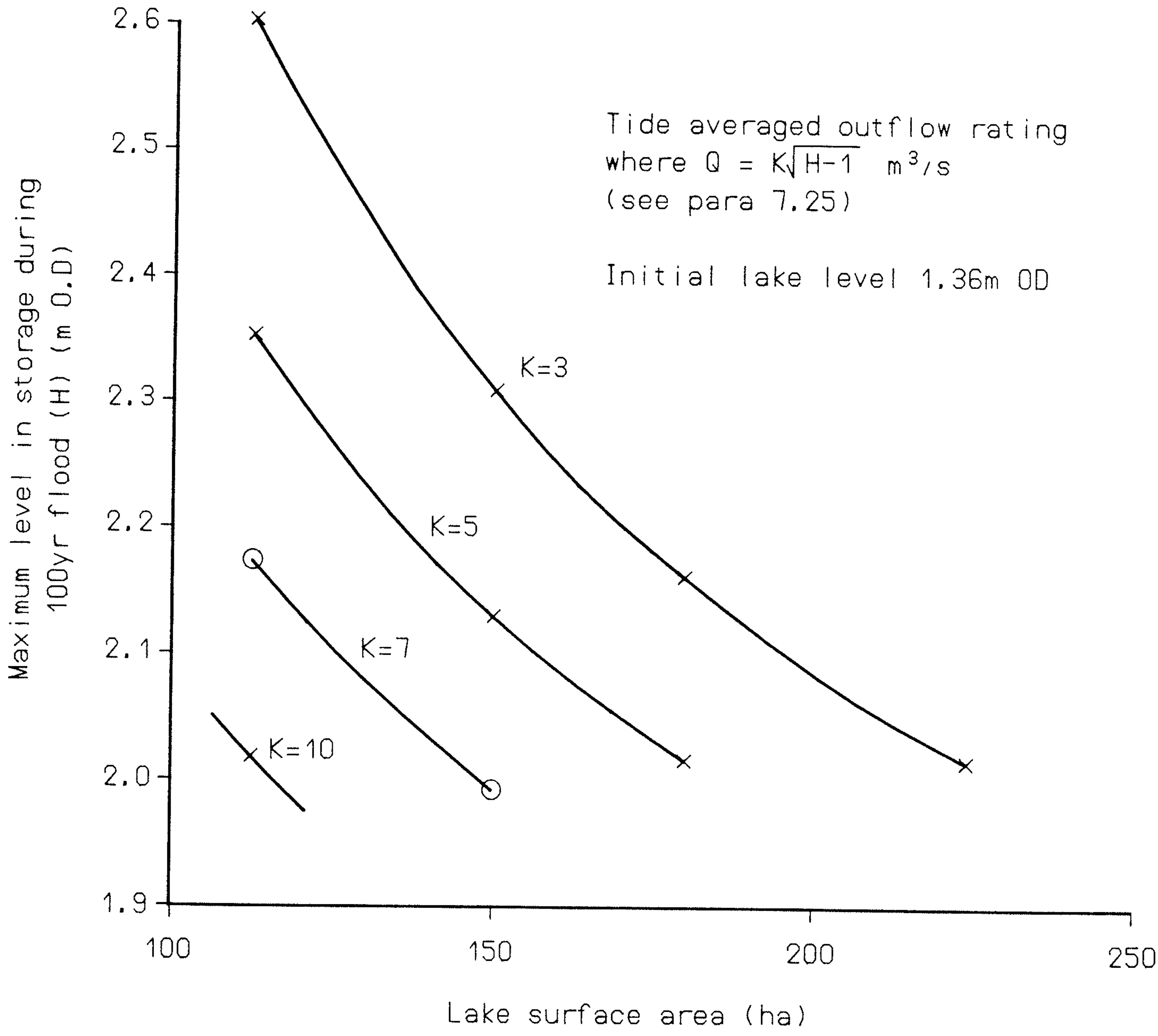
Figure 7.2



Note:
In above graph effective outlet capacity is the average outflow over a tidal cycle with a water level of 2.0m O.D on Willingdon Levels

Relationship between effective outlet capacity and maximum 100yr flood height on Willindon Levels

Figure 7.3



Relationships between effective outlet capacity, lake surface area and maximum 100yr flood level

Figure 7.4

enhanced if the retention level was reduced during the winter months when prolonged large volume floods are most likely.

Operation of the Willingdon Levels during floods

8.4 At times of flood the constraints on the operation of the lakes and channels in the Willingdon Levels will be very different from those that dominate normal operation. During floods a major priority is to direct as much water as possible through the main channels rather than through the lakes.

8.5 The majority of inflow to the Willingdon Levels enters the Shinewater Sector from Wealdon District and then flows on into the West Langney Sector. A smaller proportion enters the Broadwater Sector directly and passes through the Southbourne Sector en route to the West Langney Sector. In our analysis of the behaviour of the Willingdon Levels lake system during floods we have considered the Shinewater and West Langney Sectors separately from the Broadwater and Southbourne Sectors.

Shinewater and West Langney Sectors

8.6 In our flood routing calculations, we have assumed that the levels in all the lakes of the West Langney and Shinewater Sectors, and also in the flooded area upstream, will be the same. In practice a water surface slope will be required to allow water to flow under gravity through the system. The difference in water level across the Willingdon Levels will depend partly on the way the water is transferred across the Levels but mainly on the size of the channels, culverts and weirs that link the proposed lake system.

8.7 Within the Willingdon Levels it will be important to allow as much water as possible to pass straight through the Levels in the main channels, to ensure that the maximum possible volume is discharged early in the flood without filling the available storage. When the storages need to be utilized it would be beneficial to use as far as is practical the storages in the West Langney Sector ahead of those in the Shinewater

Sector. This will require channels that can pass floodwater around the Shinewater lakes, and allow water levels in the West Langney lakes to rise relatively early in the flood. This will provide a large water surface to enable full use to be made of the available low tide outflow capacity at Langney Bridge with a minimum of drawdown.

8.8 Other benefits of large bypass channels would be:

(1) a reduction in the size of culverts and weirs linking the lakes together; and,

(2) preventing 'first flush' pollutants and bed load sediments from entering the lake system.

8.9 In our assessments of the approximate sizes of the drainage structures required we have assumed that the main channel across the Willingdon Levels from Shinewater Bridge to Langney Bridge will require a cross section of $20m^2$ during flood conditions. Assuming the length of this channel to be about 3500m, it would be able to discharge about 5 m^3/s with a head difference of 0.2m from end to end.

8.10 The weirs allowing flood overflow from this channel into the West Langney lakes might be set at a cill level of + 1.55m OD with a length of 50 to 70m. The weirs controlling overflow from the main channel into the Shinewater Lakes would need to be set higher at say + 1.85m OD and be between 90 and 120m long to delay inflow into this sector. The weirs controlling inflows to both the West Langney and Shinewater lakes would need to be able to discharge around $10m^3/s$.

8.11 The levels and lengths of the weirs will need to be carefully designed to ensure that the lakes fill in the most effective manner. If the weirs connecting the channels to the lakes are grassed, care will have to be taken to ensure that the water velocities over the weirs and floodways will be low enough to avoid damage to the grass. The length of the weirs controlling overflow will also depend on the distance between the channel and the lake it supplies, to ensure that the headlosses in the floodway between the channel and the lake are acceptable. We have assumed the head losses in these floodways will have to be less than 0.05m.

8.12 Within each sector, weirs will need to be provided to allow the easy transfer of water from one lake to the next. These weirs would probably be set about 0.15m above the summer retention level of the lakes and would be as long as possible to minimize head losses as water passed from one lake to the next. These weirs would be in addition to the culverts required to circulate water through the lakes during low flow periods.

8.13 We estimate that the culvert required to transfer water from the Shinewater lakes to the West Langney lakes would need to have a capacity of about $5m^3/s$ with a headloss of not more than 0.1m. This culvert might require a cross sectional area of around 6 to $7m^2$ and would be additional to the main 20 m^2 channel under Willingdon Drove.

8.14 The calculations of the size of the drainage structures is dependent on the scheme that is finally chosen. The sizes given above are indicative only and must not be used for design. A more detailed study of routing and outflow arrangements will be required to ensure that structures are of an appropriate size to allow the lake system to operate in the most suitable manner. One major problem will be that the small headlosses which must of necessity occur as flood waters move through the lake system will prevent the whole of the storage area being utilized to the permitted top water level. This will require the provision of extra storage volume or the local relaxation of constraints on top water level. A major objective of the design of the cross drainage structures will be the achievement of an economically and environmentally acceptable balance between the size and cost of the structures and the volume and cost of the storage provided.

Broadwater and Southbourne Sectors

8.15 The lakes in the Broadwater and Southbourne Sectors are fed directly by the streams draining into this part of the catchment. We have assumed 25% of the flood inflows would enter these sectors directly. Routing the design flood through the lakes in these sectors indicates that the storage available above 1.1m OD is sufficient to store the whole inflow to these sectors without exceeding 2.0m OD providing the baseflow can be passed into the West Langney Sector. The present storage available in the Broadwater and Southbourne Sectors is insufficient to provide relief for the storages in the Shinewater and West Langney Sectors.

Siltation and debris problems

8.16 The transfer of as much water as possible through the main channels crossing the Willingdon Levels is necessary for the most effective operation of the flood storage. This mode of operation will encourage sediment and floating debris that is washed downstream in the early stages of a flood to bypass the main lake areas. This should significantly reduce the amount of sediment and debris that would otherwise collect in the lakes.

8.17 The provision of overflow weirs as the main method of water transfer into the lake system during floods will encourage most of the debris travelling downstream at the peak of the flood to enter the lake system. We recommend that appropriate trash screens or booms are placed around the overflow weirs to contain the debris within a small section of the lake from where it can be removed after the flood has subsided. Ideally these booms should be placed in 2m deep water and have a length of 50 to 100m. The low average velocities passing the screen would ensure minimal headlosses even if the screens became partially blocked. The design of the screens or booms will depend on whether there is a need to contain surface oil slicks.

8.18 The use of overflow weirs will ensure that the larger sediments which normally travel close to the stream bed and may be moved during a flood will remain in the main stream. Most of the finer sediments flushed through in the early stages of a flood before the levels rise enough to overtop the weirs will also pass down the main channels.

8.19 During floods, fine sediments suspended in the upper section of the main channels will be diverted into the lakes over the weirs. Any fine sands in the suspended sediments will settle quickly in the quiescent conditions in the lake, mostly within about 100m of the inlet weir. If the accumulation of sand sized material caused a problem it could be periodically removed from the area adjacent to the inlet weirs. The debris screens should be positioned around the area where dredging might be required. The silts and clays that enter the lakes during floods will settle in a very thin layer over a large portion of the lake bed.

8.20 During low flow periods the water circulating through the culverts linking the lakes would contain some silt and clay sized sediments. Most of these sediments would also settle in the lake system and would be added to those which settle during floods.

8.21 The average rate of accumulation of silts and clay sized sediments in the lake system is likely to be very slow. For example if all the runoff from the catchment passed through the lake system and contained 100 mg/l of silt and clay, the average rate of siltation in the lake system would be around 1mm/yr. In practice, since both assumptions are conservative the average rate of accumulation will be much lower, though locally could reach 1 or 2 mm/year. The accumulation of sediment on the lake bed over a 50 year period would be unlikely to affect the proposed uses of the lake.

9 CONSTRUCTION OF THE AMENITY LAKES

Lake dimensions and drift deposits

9.1 In the EPDP report is is envisaged that the proposed amenity lakes will comprise excavations approximately 3m deep and normally filled with water to a depth of 2m, so that the water surface lies about 1m below original ground level.

9.2 Although the thickness of the different drift deposits varies across the sites of the proposed lakes the typical sequence is:-

(1) topsoil;
(2) a 1m thickness of firm to soft, brown to grey silty clay;
(3) a 1m thickness of peat; and
(4) several metres of soft silty clay.

Thus the part of the excavation normally above water level will consist of firm clay whereas the submerged sides and floor the lake will be in the soft clay. The peat will typically outcrop in the vinicity of the shoreline.

Method of excavation

9.3 The soft clay floor of the excavations for the lakes will have insufficient bearing capacity to support the movement of excavating or haulage plant. Excavations will therefore need to be carried out from the rim of the excavation using dragline or back hoe excavators standing on the stronger surface clay and topsoil, and loading into rubber tyred dump trucks. Even so it will probably be necessary to use moveable mats to provide a stance for the machines while they are travelling or working.

9.4 Haulage of the excavated material to the landfill sites or tipping points for the landscaped mounds mentioned in the EPDP report will require the construction of a system of temporary roads because the ground surface is generally too weak to support heavy wheel loads without excessive rutting. Such roads would typically consist of between 0.5 and 1.0m of firm fill laid over a layer of woven geotextile to resist punching.

9.5 It will be sensible to leave the existing grass and topsoil in place when constructing such roads so that advantage is taken of the firmer surface crust they provide.

Stability and protection of the lake margins

9.6 Along the rim of the excavation the presence of the soft clay beneath the peat and surface layer of clay may be expected to give rise to slipping and slumping of the face. This has been demonstrated in the trial ponds excavated as part of the site investigation. Considerable care will be needed in the positioning of the excavators and the permanent edge of the excavation will need to be battered back or stepped to an average slope no steeper than, say, 1 in 4 if serious slumping is to be avoided.

9.7 The EPDP report envisages that there will be landscape mounds at some points along the lake margins. Considerations of stability of the excavated margin will dictate that such mounds should be of limited height (say 3m maximum) and set well back from the shoreline.

9.8 Where the peat layers outcrop in the lake shore, close to normal top water level, they will erode more quickly under the action of waves than the overlying clay. The resulting overhangs will be unstable and will collapse when the clay layer cracks. This process will tend to lead to a soft clay shoreline with peat redeposited on the lake bed below the lowest level disturbed by wave action. Other factors which will come into play include the growth of reeds and shoreline vegetation, which will reduce wave action and act to stabilise this soft margin. Erosion of the clay margin of the lakes by wave action will tend to colour the water and the fine nature of the particles in suspension will cause the colour to persist even when wind action is no longer present.

9.9 Measures which can be considered for limiting the effects of wave action along the lake shoreline include:-

(1) placing gravel and pebbles to form artificial beaching;
(2) the use of timber revetments;
(3) the encouragement of reed growth in appropriate locations; and,
(4) the use of geotextile and concrete slab protection systems.

Of the above, beaching is the most natural and immediately effective measure. It is also likely to prove the most cost effective given that suitable materials should be available within an economic haul distance.

Seepages to and from the lakes

9.10 On the evidence of the available borehole information there is little doubt concerning the ability of the lakes to hold water since they are underlain by a continuous layer of clay. Should any local discontinuities be revealed during excavation there is ample material available to construct a clay blanket.

9.11 Any inflows to or outflows from the lakes through the peat layer are likely to be small because of the relatively low permeability and shallow hydraulic gradients. Should it be necessary to exclude any undesireable seepages, there is ample clay available to blanket the peat outcrop. Alternatively a cutoff could be constructed by excavating a trench through the peat , some distance back from the shoreline and refilling the trench with clay.

10. WATER QUALITY

Available data

10.1 Southern Water have taken water samples at several points on the main streams of the catchment at approximately monthly intervals over a 10 year period. These samples were analysed only for sanitary parameters to monitor the affect of discharges from Polegate sewage treatment works (STW). Since the works was take out of service in July 1986, monitoring has been reduced.

10.2 Because Polegate STW no longer discharges to the catchment the above data provide only a general guide to likely future conditions in Willingdon levels. We have therefore restricted our investigation to the information for 1985 and 1986 for the sampling points on:-

(1) Willingdon Upper Sewer immediately upstream of Willingdon Drove; and

(2) West Langney Sewer at Fence Bridge.

These data are summarised in Table 10.1.

10.3 Southern Water have also taken a few spot samples at various points around the Lottbridge Drove landfill site which have been analysed for heavy metals.

10.4 Additional data are available from six trial ponds and from three shallow boreholes, in the Park area. These data were collected as part of a pollution monitoring study carried out by the Countryside Research Unit of Brighton Polytechnic (Ref. 7). The data from the trial ponds are summarised in Table 10.2.

Quality of streamflows

10.5 The samples taken from the Willingdon Upper Sewer show the water to be slightly alkaline, moderately hard to hard (137 to 270 mg/l total hardness as $CaCO_3$) and with total alkalinity ranging from 75 to 202 mg/l. The mineral and saline constituents of the water are moderate, the chloride ranging from 40 to 93 mg/l.

10.6 The median concentrations of ammoniacal nitrogen and nitrite nitrogen in the Willingdon Sewer are 0.47 mg/l and 0.10 mg/l respectively. These concentrations would normally be taken as evidence for gross contamination of a surface water drainage by the effluent and suggest relatively low diluting flows from the upper catchment. The organic content is substantial at times, indicating the current effect of urban drainage or perhaps resuspension of material deposited from Polegate STW.

10.7 The concentrations of total oxidised nitrogen in Willingdon Upper Sewer show considerable variation (from 0.5 to 11.3 mg/l) although the median of 4.6 mg/l is not unusual. The concentrations at the Fence Bridge site are reduced by half over the Willingdon Sewer concentrations, almost certainly indicating plant uptake of the nitrogen.

10.8 The concentrations of orthophosphate, the other major plant nutrient, are high for a surface water and almost certainly stem from sewage discharges. This median concentration falls from 2.8 mg/l at Willingdon Drove to 1.00 mg/l at Fence Bridge again suggesting the uptake of phosphorus by algae or macrophytes.

10.9 The pH variations at the two sites provides further evidence for algal or macrophyte growth in the system. Also the values for total oxidised nitrogen show that nitrogen is at times completely removed from solution. This indicates that nitrogen is the limiting nutrient on algal or macrophyte growth in this ecosystem, a somewhat unusual occurrence in freshwater systems where orthophosphate is commonly the limitation.

10.10 Figure 10.1 shows the recorded nitrogen and phosphorus concentrations since January 1985. The effect of removing the sewage effluent discharge from the Willingdon drainage is clearly shown. It must be expected that these nutrients will, however, continue to be leached from the surrounding drainage for a year or two before falling to more stable and lower concentrations.

Water quality in the trial pits and boreholes

10.11 The study by the Countryside Research Unit (Ref. 7) was concerned with detecting any deleterious effect of the landfill sites on water quality. The study used phenols and bacteria as indicators of contamination of the ground water by industrial trade wastes from the landfill sites.

10.12 The choice of phenol is unusual and we are not aware of any other published literature where phenol has been used as an indicator of pollution by landfill lechates. However, there were only three occurrences of low levels of phenol, traces being found in two of the ponds and one borehole. These could have resulted from a wide variety of sources and we do not feel they are a cause for concern. We would expect any ground water contamination, even at low concentrations, to be present at a permanent background level rather than as pulses of higher concentration.

10.13 The bacteriological data for the surface waters in the trial ponds show that median concentrations of total coliforms fluctuate from 500 to 1700 bacteria per 100 ml and peak at 20,000 per 100 ml, with E.coli giving occasional positives. These concentrations fall well within the expected range for surface waters in an actively cattle grazed drainage area. There is no evidence that these numbers reflect other than normal animal faecal contamination. Salmonella was found to be absent for all tests.

10.14 In addition to the study by the Countryside Research Unit, Southern Water have analysed spot samples taken from around the Lottbridge Drove landfill site for heavy metals. These analyses have not yielded any values above the expected background levels for the area.

10.15 We do not consider phenols and bacteria are particularly sensitive indicators of pollution and would have preferred to have seen more tests using indicators such as ammonia and heavy metals (e.g. copper, chrome, lead and zinc). However, from the limited data available there is no evidence to suggest contamination of the surface or groundwater of the Park as the result of leachate from the landfill site.

Effects of urban drainage

10.16 Approximately 40% of the catchment draining to Willingdon Levels is already urbanised and the limited urban developments detailed in the EPDP report are unlikely to cause a further significant deterioration in the quality of the catchment runoff. However it should be recognised that the runoff from highly urbanised catchments often contains significant loads of particulate organics, some heavy metals and slicks containing oil and tar. There are no data for these parameters for Willingdon Levels and it is difficult to predict the likely loadings because runoff quality is highly site specific. Studies in North America and Australia have shown that stormwater runoff from residential areas can produce loads equivalent to the load from a sewage effluent in terms of nitrogen, phosphorus and suspended material. Much of the suspended material will reach the water courses when mobilised in the first flush following a period of dry weather, typically as the result of a summer thunderstorm.

Water quality requirements for the park

10.17 The development of the Park is primarily for flood storage, but nevertheless it will not be seen as a success if water quality in the planned lakes and water features is not attractive. Table 10.3 lists the water bodies that have been proposed, together with their planned recreational use, and "key" features. These features are primarily the physical requirements or objectives necessary to achieve suitable water quality for the intended uses.

10.18 There are no legal requirements or standards for amenity and recreational lakes in the UK, but we have set out in Table 10.4 the criteria and actual standards (the latter as concentrations) which might well be adopted by a management team as objectives for the three major uses of the water for which water quality has importance. These criteria are taken from the EEC Directives which now apply by law to certain natural water bodies.

10.19 There is a fourth use of the water not described in Table 10.4. The appearance of the water will be of great concern to the public in several of the water bodies. In Southbourne Lake and Winkney Lake parents and children will be major users and the aesthetic quality of the environment will be important for the venue's success. In these lakes, in particular, good clarity will be important, as well as the absence of floating debris.

10.20 The EPDP report envisages that the lakes:

(1) will have a constant depth of 2 m at normal top water level; and

(2) will be offline in the sense that, except during times of flood, the major part of the flow across Willingdon Levels will be via the water courses.

Runoff from the catchment will obviously be required to fill the lakes initially, to make good evaporation losses and to ensure that the lake contents are renewed. We estimate that the long term average runoff to the Park is between 0.3 and 0.4 m^3/s with perhaps 80% of the average annual runoff occurring during the winter months. Even if all summer flow from the catchment upstream of the Shinewater sector were routed through the lakes, their contents would not be replaced in an average summer.

10.21 Based on evidence of the 1985 and 1986 data, the water quality in the drainage channels entering the Park area falls within Class 2 of the National Water Council classification, although occasionally deteriorating towards Class 3 quality. It should be noted that the existing quality (see Table 10.1) exceeds the guideline concentrations for coarse fish for ammonia and phosphorus. Southern Water consider that the occasional deterioration in water quality within Willingdon Levels was due to storm water overflows from Polegate STW. This source of pollution has now been removed and a trend to lower concentrations in nitrogen and phosphorus has been observed. There should also be a gradual improvement in the BOD, ammonia and nitrite levels in the runoff from the catchment upstream at the Shinewater sector. Nevertheless, the risk that flood runoff from the urban areas, probably as the result of a summer thunderstorm, will cause a sudden deterioration of water quality must be recognised.

10.22 The variations in nitrogen and phosphorus concentrations in the present drainage system provide evidence of algal and/or macrophyte growth. In the open waters of the proposed amenity lakes, with their very long retention times, the potential for growth will be much greater and particularly so if measures are taken to ensure the clarity of the waters (see paragraphs 9.8 and 9.9). In spite of a general improvement which may occur in the ambient concentrations of nitrogen and phosphorus in runoff from the catchment it likely that:-

(1) on occasions during summer algae will flourish in the surface waters; and

(2) macrophytes will rapidly colonise the whole lake area where light reaches the bottom sediments.

10.23 To reduce the amount of maintenance required we recommend that consideration is given to:-

(1) deepening parts of the proposed lakes; and

(2) where possible ensuring a constant throughput of water.

If the middle of Broadwater, Shinewater and West Langney lakes were deepened to 3 to 4 metres this would not only help to limit the growth of algae and macrophytes but would also provide a more attractive habitat for fish, allowing them to avoid the hot upper layers in summer and the attentions of diving birds. The average depth of the lakes need not be altered greatly from 2 metres, as part of the shoreline can be made gradually shelving to provide natural marshy banksides. This would be particularly suitable, say, on the western shores of Shinewater Lake and its wildlife reserve island. and on the western shore of West Langney Lake.

10.24 In spite of the above, with the long residence times in the lakes, water quality can be expected to decline markedly during most summers unless remedial action is taken. A study will need to be carried out to establish the most effective method of maintaining appropriate water quality standards in the lakes. However we expect that it will prove necessary to install some form of mechanical aeration and recirculation system. It may also be necessary to use algicides.

Landscaping and tree growth

10.25 We do not envisage that the earth moving operations to form mounds or embankments will result in any longterm water quality problems. Our experience in the construction of comparable development at Strathclyde Park, showed that utilisation of the subsoils caused no water quality problems, provided they were adequately drained. However the establishment of a plant cover on the newly formed embankments was slow unless top soiling was undertaken.

10.26 The use of pre-seeding fertilisers and subsequent fertiliser dressings could provide undesirable enrichment of the lakes in Eastbourne Park. The plant cover should be allowed to establish itself on the newly formed land, for at least a year, before the lakes are stocked with fish.

10.27 Evidence from the Lottbridge Drove experimental planting site (Ref. 8) suggests that it will be possible to establish a wide range of tree species within the Park without undue difficulty.

10.28 We have reviewed the relevant data in an MSc thesis by A J Morey (Ref. 9). No excessive chloride or sulphate concentrations were found in groundwater samples from the area (with the exception of a single sample with sulphate in excess of 1000 ppm), nor were pH values other than around neutrality recorded.

10.29 It seems unlikely therefore that any water quality problem is likely to arise in the Park from use of the indigenous subsoils in the landscaping. Some waterlogging of some mounds may occur due to the mixed nature of the sub soils with pockets of peaty deposits in the clay silts, but we believe that local remedial drainage using standard land drainage techniques will be able to cope with such difficulties. It will probably be useful to employ some of the pioneer species such as alder, <u>Alnus sp.,</u> and the wet ground tolerant species of willow, <u>Salix spp.</u> as initial plantings to help dry out areas where taller cover trees are wanted in the long term design. The initial plantings can be thinned out when the permanent trees are well established.

Determinand		Willingdon Upper Sewer TQ62570 03580				Fence Bridge TQ64900 04740			
		Mean	**Median**	**Minimum**	**Maximum**	**Mean**	**Median**	**Minimum**	**Maximum**
Temperature	°C	11.75	12.5	4.0	21.0	11.7	12.0	6	20
pH Value	Unit	7.55	7.5	7.05	8.01	7.6	7.5	7.09	9.05
Dissolved Oxygen Saturation	%	65	68	40	78	69	70	56	82
Alkalinity pH 4.5 as $CaCO_3$	mg/l	161	180	75	202	177	185	136	208
Total Hardness as $CaCO_3$	mg/l	228	236	137	271	230	225	127	308
Chloride	mg/l	61	63.5	40	93	74	76	45	102
Ammoniacal Nitrogen	mg/l	0.82	0.47	0.05	2.50	0.26	0.12	0.02	1.2
Nitrite Nitrogen	mg/l	0.19	0.10	0.01	0.69	0.06	0.06	0.01	0.19
Total Oxidised Nitrogen	mg/l	4.09	4.6	LT0.5	11.30	2.0	1.4	LT0.5	5.30
Orthophosphate Phosphorus	mg/l	2.50	2.80	0.31	5.71	1.62	1.00	0.22	5.10
Biochemical Oxygen Demand 5 Days	mg/l	3.44	2.65	0.90	13.0	2.48	2.15	1.0	5.5

Notes

LT = Less than

Based on Southern Water data for period July 1985 to November 1986

Summary of water quality data in
Willingdon and West Langney sewers

TABLE 10.1

Trial Ponds

	1		2		3		4		5		6	
	Median	Range	Median	Range	Median	Range	Median	Range	Median	Range	Median	Range
Aerobic bacteria ($\times 10^{-4}$) ml^{-1}	1.3	0.3-18	2.0	0.9-9.8	1.4	0.5-11.0	4.8	0.2-6.6	1.0	0.2-4.8	1.0	0.2-2.9
Anaerobic bacteria ml^{-1}	475	20-1080	495	126-1660	330	30-1013	133	40-280000	140	20-7200	147	67-2700
Yeasts and moulds ml^{-1}	26	6-970	19	10-63	14	10-24	9	10-25	18	10-38	22	10-43
Coliform bacteria ml^{-1}	10	10-38	40	10-43	17	10-200	10	10-16	13	10-700	10	10-250
E.coli ml^{-1}	-	(- -)	+	(- +)	+	(- +)	+	(- +)	+	(- +)	+	(- +)
Salmonella	-	(- -)	-	(- -)	-	(- -)	-	(- -)	-	(- -)	-	(- -)
pH Value	7.8	7.5-8.7	7.8	7.2-8.3	8.1	7.5-8.0	8.1	7.2-8.3	8.1	7.4-8.3	8.0	7.7-8.3
5 day BOD mg/l	5.6	4.2-10.8	3.3	2.5-5.0	2.7	2.2-5.9	3.6	2.4-14.7	1.6	1.0-2.5	4.5	1.0-5.9
Phenols mg/l	0.5	0.5->5.0	0.5	0.5->5.0	0.5	0.5-1.0	0.5	0.5->5.0	0.5	0.5->5.0	0.5	0.5-1.0

Less than (limit of detection)

\- Absent + Present

Data for period February to July 1984

Analyses by Brighton Polytechnic

Pollution monitoring data

TABLE 10.2

Name of Water Feature	Major Uses	Management Objectives
Shinewater Lake	Angling, Wildlife	Island reserve, WQ for fish
Winkney Lake	Model Yachting	Hard shoreline, shallow, good clarity, WQ suitable for accidental immersion.
Hydneye Lake	Angling, Boating	Island for camping(?), WQ suitable for accidental immersion and fish
Larkspur Lake	Wildlife Reserve	Natural shorelines, WQ not critical but to fishery quality
Willingdon Marshes	Wildlife Reserve	Natural shorelines, WQ not critical but to fishery quality
Lakelands Pond	Angling, Wildlife(?)	Natural shorelines, WQ not critical but to fishery quality
West Langney Lake	Sailing, Rowing, Canoeing, Board Sailing, Angling, west shore as a nature reserve	Island for camping, landing facilities on one shore, west shore natural shoreline, WQ suitable for accidental immersion and fish.
Highfield Pond	Angling, Wildlife	WQ not critical but to fishery quality
St Anthony's Lake	Angling	WQ not critical but to fishery quality
Langley Rise Marsh	Wildlife, Views	Natural vegetation, WQ not critical but to fishery quality
Southbourne Lake	Rowing, Canoeing, Views	Natural shoreline in parts, high clarity, WQ suitable for accidental immersion.
Southbourne Marsh	Wildlife, Views	Natural vegetation, WQ not critical
Decoy Pond	Views	Natural vegetation
Broadway Ponds	Settlement traps, Wildlife, Views	Natural appearance, WQ not critical
Broadwater Lake	Sailing, Rowing, Canoeing, Board Sailing, Angling	Some natural shores, WQ suitable for accidental immersion.
Water courses	Canoeing, Angling	Navigable, WQ not critical but to fishery quality

Planned uses of water features in Eastbourne Park

TABLE 10.3

WATER QUALITY CRITERIA SUITABLE[1] FOR AMENITY LAKES

Concentrations in mg/l except where indicated as annual means

Determinand	Coarse fishery[2]	Other freshwater life[2]	Bathers[3]
Temperature °C	28 maximim	-	-
pH Value Unit	6 to 9	6 to 9	6 to 9
Suspended solids	25	-	-
Secchi disc depth (m)	-	-	2=G, 1=MAC
Dissolved oxygen	GT 8mg/l as median	-	80-120 as % satn
Orthophosphates as P	0.31	-	
Total Ammonia as N	0.78=M, 0.16=G	-	
Unionised Ammonia as N	0.021=M, 0.004=G	-	
Nitrite as N	0.009=G	-	
Arsenic[4]	0.05	0.15	
Cadmium [4,5]	0.005	0.005	
Chromium[4]	0.25	0.05	
Copper[4]	0.04=M[6], 0.01=G	0.01=G	
Lead[4]	0.25	0.06	
Mercury[4,5]	0.001	0.001	
Nickel[4]	0.2	0.1	
Zinc[4,5]	1.0=M[6], 0.25=G	0.10	
Hydrocarbons	Below concentrations causing harm to fish or tainting of flesh		
Phenols			
Surfactants			
Faecal coliforms @44°C[6,7]			2,000=M, 100=G
Total coliforms @ 37°C[6,7]			10,000=M, 500=G
Salmonella[6,8]			0=M
Faecal streptococci[6,7]			100=G
Enteroviruses[6,9]			0=M

Footnotes

1 None of these sets of criteria and standards apply by law to amenity waters (see text)

2 EEC Standards for water supporting freshwater fish life

3 EEC Standards for bathing water

4 EEC Standards for trace substances for protection of freshwater fish and other freshwater life. Values shown are for alkalinity 200-250 mg/l

5 Total metal value (Particulate + dissolved components)

6 Concentration is a 95 percentile

7 Unit is MPN/100 ml

8 Unit is MPN/1 litre

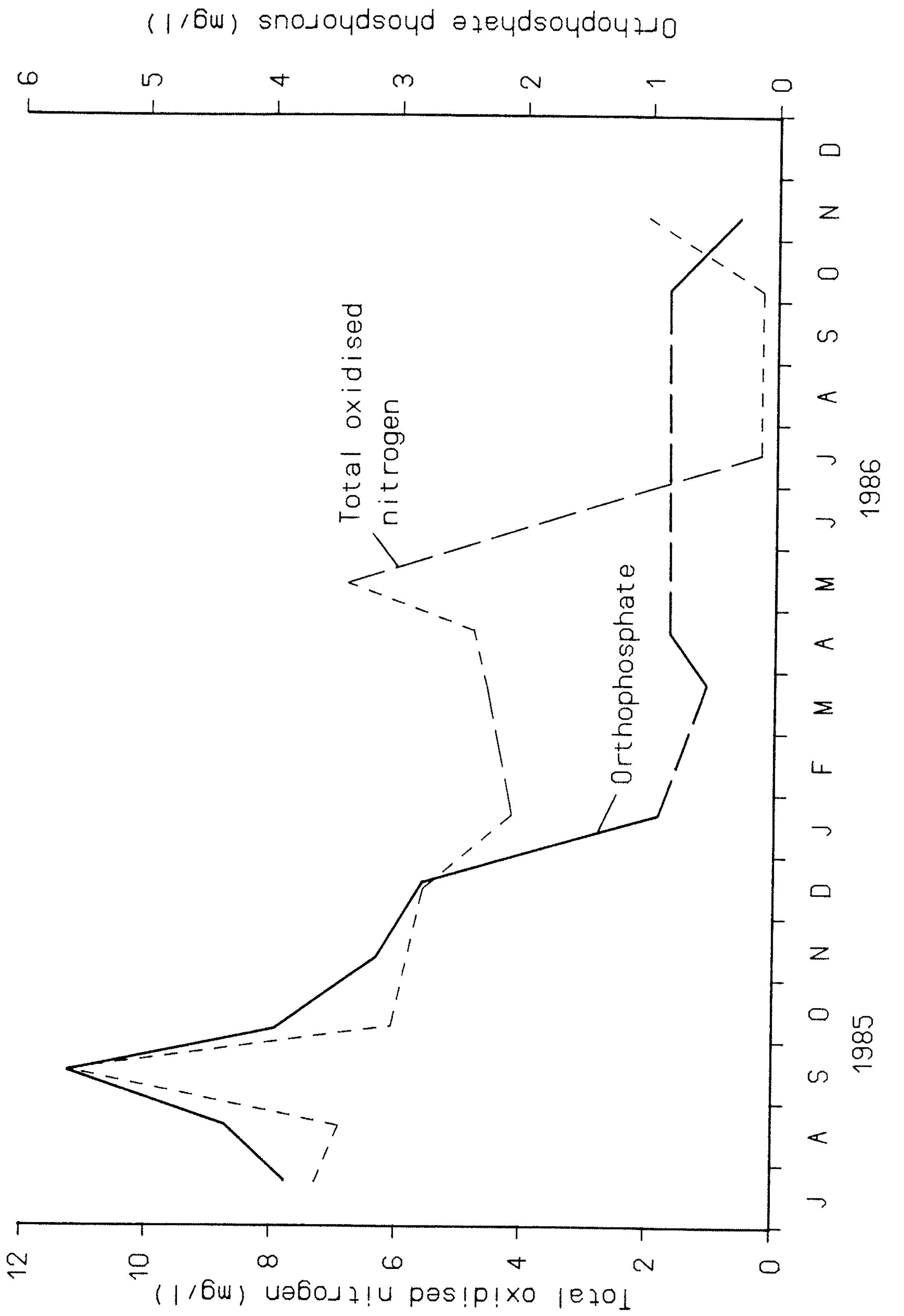
9 Unit is MPN/10 litres

M = Mandatory standard

G = Guideline standard

GT = greater than

TABLE 10.4



Nutrient Levels in Willingdon Upper Sewer

Figure 10.1

APPENDIX A

HISTORICAL REVIEW OF FLOODING

Rainfall search

A.1 At an early stage of the study, a search of daily rainfall records was made for possible historical flood-producing events on the Park catchment. In this respect, the long and generally continuous record for Eastbourne Wilmington Sq. was exceptionally helpful. A search for large 1-day and 2-day rainfalls in the period 1888-1984 provided a preliminary list of events for which rainfall depths over a range of durations were examined further. Subsequently, two events in 1985/1986 were added. The outcome of the rainfall search was the list of notable rainfall events presented here as Table A.1 The list is thought to be reasonably comprehensive but is biased to include some recent events (for which additional data are available) that would otherwise not have qualified as notable.

A.2 The rainfall data from Table A.1 is represented in Fig. A.1 to highlight the relative severity of the events over particular durations (from 1 to 16 days). Comparison of the depth-duration data for particular events with 5-year and 100-year design values provides an indication of the relative severity of particular events. As a first example, it is interesting to note that the still-remembered severe flood of October/November 1960 (EVENT M) had rainfall depths which were exceptional only for durations of 4 days or greater. Moreover, at longer durations the event was still much smaller than those of October 1939 (EVENT G) and October 1949 (EVENT J).

Antecedent condition

A.3 Flood runoff from heavy rainfall events is affected by the antecedent catchment wetness and this is conveniently indexed by antecedent rainfall and time of year. (See second and third columns of Table A.1)

Flood reports

A.4 Various sources of information were used to glean information about the nature of flooding in the park area and its surrounds. These sources included:-

(1) the EPDP report and a companion document entitled 'Eastbourne Park Landscape Studies';

(2) a Section 24 Survey produced by Southern Water;

(3) correspondence files relating to specific incidents brought to the attention of Southern Water; and,

(4) photographs supplied by Eastbourne Borough Council and a local resident.

We also undertook an independent search of local newspaper records, guided by the results of the search of the rainfall records.

A.5 Further details of some of the events listed in Table A.1 are given below. Attention has been biased towards the larger and more recent events.

4/15 OCTOBER 1939 (EVENT G)

No information was found relating to flooding in this event. The antecedent condition was dry and it is possible that the resultant flood was insufficient to warrant mention in wartime newspapers. (From Fig A.1 it is seen that the long-duration rainfall depths were extremely high).

15/26 OCTOBER 1949 (EVENT J)

Newspaper accounts centred on flooding in Central Eastbourne, attributing this to torrential rain coinciding with the high tide such that surface sewers could not cope. A separate reference to "serious flooding at Hampden Park, where the marshes around the Hydney Estate are still deeply flooded right out to Stone Cross" is perhaps the most telling of all the historical information gathered. (Rainfall depths in this event provide the historical maxima for all durations above 2 days - See Fig A.1).

19 OCTOBER / 4 NOVEMBER 1960 (EVENT M)

Newspaper accounts of 2 November 1960 refer to: "heading for the wettest ever year", "stretches of road under water at Wannock", and "Monday evening's rain was too much for the dyke and streams entering marshland between Polegate and Hampden Park, the water overflowing to form large 'lakes', the one in the picture isolating a pylon". The issue of 5 November 1960 has a photograph showing flooding nearly to doorstep level in Hampden Avenue on Thursday afternoon. This would seem to confirm the longevity of innundation of the Park area in this event (assuming that the street flooding was indicative of drowned surface water outfalls). An East Sussex River Board report on the November 1960 flood indicates that it as "not particularly severe especially in the upper reaches, worse conditions having been experienced in the past few years. In the lowland however, flood levels were higher as the levels in the Crumbles and Willingdon Upper Sewers and Langney Haven are affected by conditions at the Pevensey Bay outfalls as the two systems are interconnected. (Elsewhere a water level of 1.93m is quoted for Crumbles Sluice). There is no doubt that works carried out in this area during the last few years had a beneficial effect in lowering levels". This quote raises the point that the existing drainage system has evolved over many years and points to the significance of upgrades to the Pevensey Bay tidal outfalls undertaken in the mid/late 1950's. The EPDP report speaks of this event as being "the last time when conditions arose which gave rise to concern... but the flood did not materialise".

24 NOVEMBER / 9 DECEMBER 1960 (EVENT M)

This event followed soon after Event M and led to reports in the 7 December 1960 issue: "water swirls off Downs", "wettest ever year", "the water was coming off the hills in streams", "worst rainstorm and gale for many years; 12 hours of non-stop wind and torrential rain", "many call-outs to cellars in Hampden Park area", "drainage ditch at Manor Close, Willingdon overflowed" and, accompanying a picture of flooded gardens, "the flooding at Lower Willingdon". These reports indicate that, in some circumstances, moderate to rapid response flows from the chalk areas may be significant and that a flood build-up in the Park area may be accompanied by gales (which presumably might exacerbate flooding through wave set-up).

9/19 JUNE 1971 (EVENT O)

This event led to a newpaper report (23 June 1971) "water, water, everywhere" but little detail other than a well overflowed and flooded land at Wilmington, in an adjacent catchment. A Sussex River Authority report indicated that those flooding problems that did arise were confined to gardens and could be attributed to local features of the drainage system. However, the report acknowledged that "following the extensive development of Eastbourne on the boundaries of the Willingdon development of Eastbourne on the boundaries of the Willingdon Marshes ... the rate of run off will inevitably be increased in future years" - in response to complaints from local residents about the "general tipping on the Marsh and filling in of the flood plains". This quote reminds that the exacerbation of flood frequency by development has a long history in the Park. Well level data for Folkington (in the north-west corner of the catchment) showed a remarkable rise of 5 metres during the period of this event. While this demonstrates the capacity of the chalk to absorb heavy rainfall, the rate at which the well level subsequently fell indicates that the spring flow response on the scarp slope can be relatively rapid.

13/22 NOVEMBER 1974 (EVENT P)

The newspaper of 23 November 1974 reported a big flood after a week of heavy rain and hundreds of houses and gardens inches deep in water. (The reference to inundation of hundreds of houses would seem to be an overstatement; presumably some house flooding occurred due to local surface sewer problems). The Hampden Park area appeared to be particularly affected.

28 NOVEMBER / 1 DECEMBER 1976 (EVENT S)

Newspaper reports were directed more at flooding in the Cuckmere catchment to the north-east.

6/8 JULY 1980 (EVENT T)

The newspaper of 12 July 1980 refers to flooding of gardens and some roads and basements, the latter being attributed to rainfall in excess of the local surface water drainage system. (Reference to Fig A.1 indicates that the short-duration rainfall depths in this event were indeed exceptionally high. Given that the antecedent period was wet, the fact that the event did not lead to widespread flooding would seem to confirm the general sensitivity of the Park catchment to long-duration events).

? 1984

According to Southern Water files, a drainage problem arose at a factory in Birch Road (off Lottbridge Drove). This appears to have been a design miskake - the surface water discharging 0.5m below the summer retention level in the arterial drainage system. A reply from Eastbourne B.C. indicated that the problem would be improved by the balancing reservoirs foreseen in the EPDP which would "enable the water, to be kept at a lower level".

16 DECEMBER 1984

The rainfall analysis did not identify this as a notable event. It would appear from Southern Water files that flooding of gardens occurred in Sandpiper Walk (West Langney), attributable to backing up of water in the surface water sewer "due to the height of water in the dykes" (ie the West Langney and Willingdon Sewers). This type of secondary flooding seems to be thematic of drainage problems on the periphery of the Park which arise because of the very limited hydraulic gradient available.

23 DECEMBER 1985/7 JANUARY 1986 (EVENT W)

Details of the event are contained in Section 6.

NOTABLE RAINFALL EVENTS - EASTBOURNE (1888-1986)

Event key	Antec^{t} condit^{n}	Date	Rainfall (mm) in stated duration (days) 1	2	3	4	6	8	10	12	16
A		25/28 Oct 09	50	66	82	84					
B	wet	7/12 Nov 11	36	52	73	82	115				
C	dry	29Sep/1 Oct 12	36	69	81						
D	v dry	9/10 Jul 23	43	51							
E		24/25 Jul 32	42	81	84						
F		8/15 Nov 34	45	54	56	57	57	107			
G	dry	4/15 Oct 39	53	58	76	109	135	148	167	180	
H	wet	16 Aug 46	65								
J		15/26 Oct 49	48	71	$\underline{103}$	$\underline{114}$	$\underline{136}$	$\underline{173}$	$\underline{184}$	$\underline{203}$	
K	dry	18/21 Oct 55	37	69	93	103					
k	dry	2 Jul 57	62								
L	dry	8/11 Aug 60	49	80	100	111					
M		19Oct/4 Nov 60	25	50	70	85	112	126	139	151	195
m	v wet	24Nov/9 Dec 60	34	45	54	57	61	73	90	108	123
N	dry	13/15 Mar 64	49	70	81						
O	dry	9/19 Jun 71	40	57	57	61	106	115	155	163	
P	wet	13/22 Nov 74	35	45	52	55	71	108	121		
R	wet	4/11 Nov 76	20	35	45	63	83	90			
S		28Nov/1 Dec 76	37	66	80	90					
T	wet	6/ 8 Jul 80	$\underline{77}$	$\underline{84}$	89						
t	dry	11/14 Aug 80	46	47	60	65					
U		21/26 Nov 82	23	34	49	52	69				
V	v dry	23/28 Mar 84	23	37	40	48	65				
W		23Dec/7 Jan 86	27	44	49	66	77	97	97	100	135
X	wet	17/26 Nov 86	23	41	63	72	81	89	93		

NB Underlined values denote the historical maximum for the stated duration.

Table A.1.

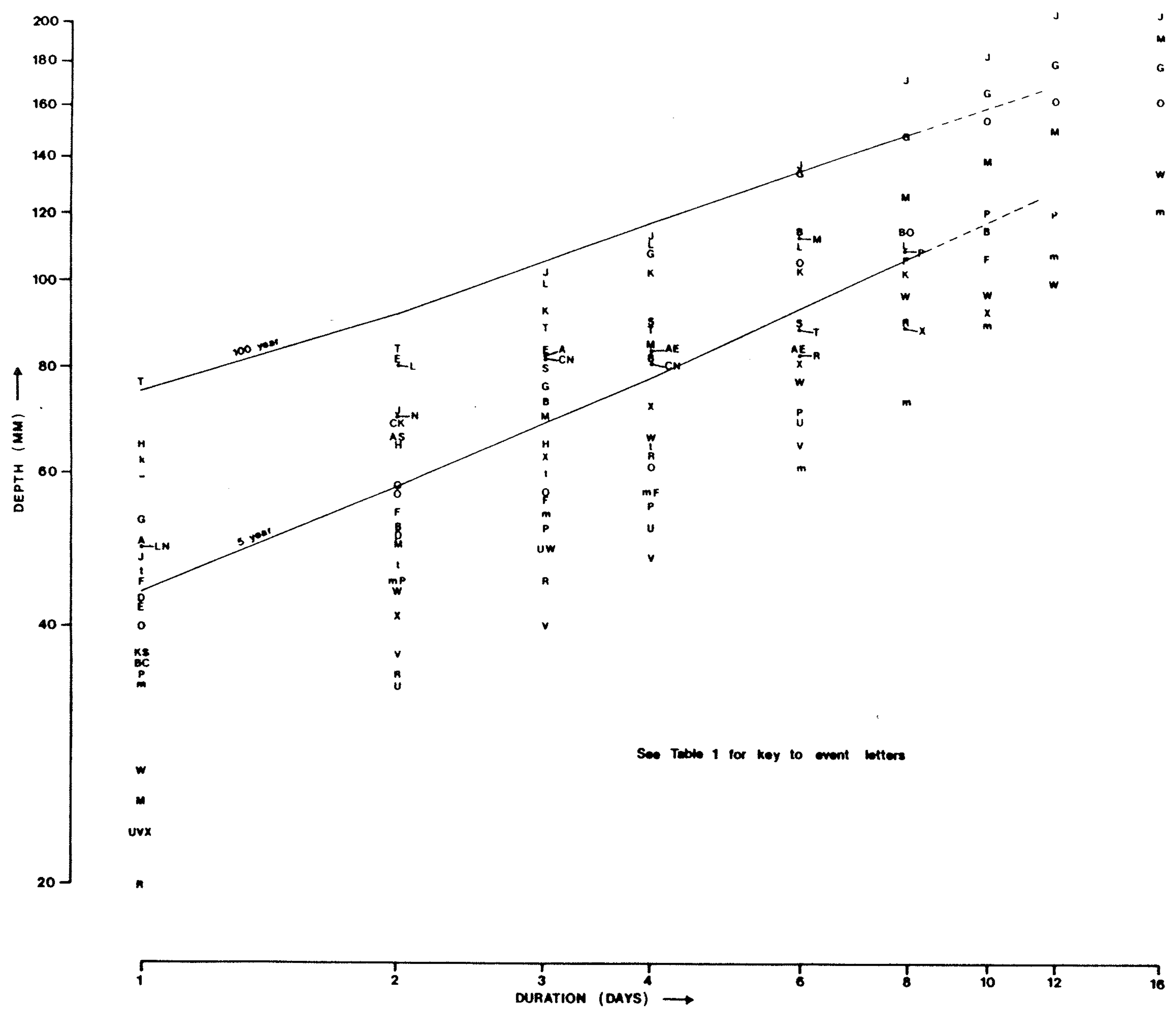


Fig. A.1 Depth-duration plot of notable rainfalls at Eastbourne (Wilmington Square) in period 1888 - 1986

APPENDIX B

100 YEAR FLOOD CALCULATIONS

1. Preliminaries

1.1 Catchment characteristics (from FSR method):

AREA	27.9	km^2
MSL	8.5	km
S1085	2.2	m/km
URBAN	0.41	(but see also Section 1.4 below)
SAAR	825	mm
M5-2day	59	mm
r	0.35	

1.2 Time to peak of unit hydrograph (from FSSR 16 method)

$$Tp(0) = 283\ S1085^{-0.33}(1+URBAN)^{-2.2}SAAR^{-0.54}MSL^{0.23}$$
$$= 4.46\ h$$

Choose data interval: $T = 1$ h
Then: $Tp(1) = Tp(0) + T/2 = 4.96$ h

1.3 Baseflow allowance (from FSSR 16 method - with amendment):

design CWI = 118
$$ANSF = (\ 33\ (\ CWI - 125) + 3.0\ SAAR + 5.5\ )\ 10^{-5}$$
$$= 0.0225\ cumecs/km^2$$
Hence, baseflow allowance = AREA x ANSF = 0.63 cumecs

Including further allowance of 0.75 cumecs for contribution from Bourne Stream yields a baseflow of 1.4 cumecs to be added to the response runoff hydrograph.

1.4 Soil/geology types and subdivision of urbanization:

Geology/soil	Area fraction	Area km^2	Urban fraction	Urban km^2	WRAP class
Chalk/Lower Greensand	0.25	6.97	0.27	1.88	1
Gault Clay/ Weald Clay	0.45	12.56	0.67	8.41	5*
Alluvium	0.30	8.37	0.14	1.17	5*
Total	1.00	27.90	0.41	11.46	

* See Sections 5.6 and 5.7 of main report for discussion of these WRAP class assignments.

2. Calculations for a 31-hour storm (assuming a reservoir lag of 12 hours)

2.1 Design storm duration (from ICE guide to floods & reservoir safety):

$$D = (1 + SAAR/1000)(Tp + RLAG)$$
$$= 1.825 \times (4.96 + 12.0) = 31.0 \text{ h}$$

2.2 Rainfall depth (from FSR):

M5-2day = 59 mm
r = 0.35

Hence: M5-31hour = 0.94 x 59 = 55.5 mm

Rainfall growth factor for 100-year flood is: M140/M5 = 1.88

Hence: M140-31hour = 1.88 x 55.5 = 104.4 mm

Areal reduction factor: ARF = 0.963

Hence design storm depth: P = 0.963 x 104.4 = 100.5 mm

2.3 Percentage runoff (from FSSR 16 method - with amendment)

$$DPR_{CWI} = -1.75$$

$$DPR_{RAIN} = 0.45\ (P-40)^{0.7} = 7.95$$

Now $PR_{rural} = SPR + DPR_{CWI} + DPR_{RAIN} = SPR + 6.2$

and $PR = PR_{rural}(1.0 - 0.3\ URBAN) + 70\ (0.3\ URBAN)$

<u>For Chalk/Greensand</u>
SPR = 10
URBAN = 0.27 (from Section 1.4 above)

Hence: $PR = (10 + 6.2)(1.0 - 0.081) + 70 \times 0.081$
$= 16.2 \times 0.919 + 70 \times 0.081$
$= 14.89 + 5.67 = 20.56$

<u>For Clays and Alluvium</u>
SPR = 53
URBAN = 0.46 (combining Clays and Alluvium from Para. 1.4)

Hence: $PR = (53 + 6.2)(1.0 - 0.138) + 70 \times 0.138$
$= 59.2 \times 0.862 + 70 \times 0.138$
$= 51.03 + 9.66 = 60.69$

<u>Combined</u>
$PR = 0.25\ PR_{chalk} + 0.75\ PR_{clay}$
$= 0.25 \times 20.56 + 0.75 \times 60.69 = 50.66$

2.4 Calculation of flood hydrograph

The remaining calculations were made by a standard computer package for which the following two pages provide a summary. The hydrograph is illustrated in Fig. 5.1 of the main report.

<u>3. Calculations for a 53-hour storm</u>

Calculations for a 53-hour storm are summarized in an attached computer listing (run reference EAS53). A modelling interval of 3.0 hours was adopted and consequently the storm duration was rounded to 51 hours.

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Willingdon Levels - 100 year flood based on a 31 hour storm

Run reference - EAS31

Catchment characteristics

Area	27.9	sq km	Soil 1	0.25
Length	8.5	km	Soil 2	0
Slope	2.2	m/km	Soil 3	0
SAAR	825	mm	Soil 4	0
M5-2D	59	mm	Soil 5	0.75
Jenkinson's r	0.35			
Urban	0.41			
Smdbar	9.5	mm	RSMD 35.15	mm

Unit hydrograph option . 1

Unit hydrograph time to peak 0.00 Data interval 1
Design storm duration 31 hours

Return Period for design flood 100.0 years
requires rainfall event depth of 140.0 years

M5-31.0hour/M5-2dav = 0.94
MT/M5 = 1.88
ARF = 0.96

M5-2dav = 59 mm
M5-31.0hour = 55.4 mm
M140.0-31.0hour =104.4 mm (point)
M140.0-31.0hour =100.5 mm (area)

Rainfall profile option 4

Design storm depth 100.5 mm

Percentage runoff 50.7 % (PR option 2)

Response hydrograph peak 26.45 cumecs
Baseflow 1.40 cumecs (Baseflow option 6)

Design hydrograph peak 27.85 cumecs

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CONVOLUTION TABLE

Willingdon Levels - 100 year flood based on a 31 hour storm

10mm 1 hour unit hydrograph (cumecs)

Rain	net rain -(cm)-	8.94	17.88	26.82	35.76	44.12	38.24	32.35	26.47	20.59	14.71	8.83	2.94																	
0.10	0.05	0.12	0.25	0.37	0.49	0.61	0.53	0.45	0.36	0.28	0.20	0.12	0.04																	
0.10	0.05		0.12	0.25	0.37	0.49	0.61	0.53	0.45	0.36	0.28	0.20	0.12	0.04																
0.10	0.05			0.12	0.25	0.37	0.49	0.61	0.53	0.45	0.36	0.28	0.20	0.12	0.04															
0.14	0.07				0.18	0.36	0.54	0.71	0.88	0.76	0.65	0.53	0.41	0.29	0.18	0.06														
0.15	0.07					0.18	0.37	0.55	0.74	0.91	0.79	0.67	0.55	0.42	0.30	0.18	0.06													
0.15	0.07						0.18	0.37	0.55	0.74	0.91	0.79	0.67	0.55	0.42	0.30	0.18	0.06												
0.16	0.08							0.20	0.40	0.60	0.80	0.99	0.86	0.73	0.59	0.46	0.33	0.20	0.07											
0.19	0.09								0.24	0.47	0.71	0.94	1.16	1.01	0.85	0.70	0.54	0.39	0.23	0.08										
0.26	0.13									0.33	0.66	0.98	1.31	1.62	1.40	1.19	0.97	0.76	0.54	0.32	0.11									
0.30	0.15										0.39	0.77	1.16	1.54	1.90	1.65	1.39	1.14	0.89	0.63	0.38	0.13								
0.36	0.18											0.45	0.91	1.36	1.81	2.24	1.94	1.64	1.34	1.04	0.75	0.45	0.15							
0.55	0.28												0.70	1.39	2.09	2.79	3.44	2.98	2.52	2.06	1.61	1.15	0.69	0.23						
0.63	0.32													0.80	1.59	2.39	3.18	3.93	3.40	2.88	2.36	1.83	1.31	0.79	0.26					
0.68	0.35														0.87	1.73	2.60	3.46	4.27	3.70	3.13	2.56	1.99	1.42	0.85	0.29				
0.76	0.39															0.97	1.93	2.90	3.86	4.76	4.13	3.49	2.86	2.22	1.59	0.95	0.32			
0.81	0.41																1.03	2.05	3.08	4.10	5.06	4.38	3.71	3.04	2.36	1.69	1.01	0.34		
0.76	0.39																	0.97	1.93	2.90	3.86	4.76	4.13	3.49	2.86	2.22	1.59	0.95	0.32	
0.68	0.35																		0.87	1.73	2.60	3.46	4.27	3.70	3.13	2.56	1.99	1.42	0.85	0.29
0.63	0.32																			0.80	1.59	2.39	3.18	3.93	3.40	2.88	2.36	1.83	1.31	0.79
0.55	0.28																				0.70	1.39	2.09	2.79	3.44	2.98	2.52	2.06	1.61	1.15
0.36	0.18																					0.45	0.91	1.36	1.81	2.24	1.94	1.64	1.34	1.04
0.30	0.15																						0.39	0.77	1.16	1.54	1.90	1.65	1.39	1.14
0.26	0.13																							0.33	0.66	0.98	1.31	1.62	1.40	1.19
0.19	0.09																								0.24	0.47	0.71	0.94	1.16	1.01
0.16	0.08																									0.20	0.40	0.60	0.80	0.99
0.15	0.07																										0.18	0.37	0.55	0.74
0.15	0.07																											0.18	0.37	0.55
0.14	0.07																												0.18	0.36
0.10	0.05																													0.12
0.10	0.05																													
0.10	0.05																													
	Baseflow	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40
	Total flow	1.52	1.77	2.14	2.69	3.41	4.11	4.82	5.55	6.31	7.15	8.14	9.48	11.27	13.46	16.05	19.00	21.86	24.40	26.41	27.66	27.85	27.07	25.47	23.16	20.41	17.64	15.02	12.69	10.76

****
* Percentage runoff is 50.70 % *
* Peak of response hydrograph is 26.45 cumecs *
* Baseflow 1.40 cumecs *
*
* Peak of design flood 27.85 cumecs *
*
****

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Willingdon Levels - 100 year flood based on a 51 hour storm

Run reference - EAS53

Catchment characteristics

Area	27.9	sq km	Soil 1	0.25
Length	8.5	km	Soil 2	0
Slope	2.2	m/km	Soil 3	0
SAAR	825	mm	Soil 4	0
M5-2D	59	mm	Soil 5	0.75
Jenkinson's r	0.35			
Urban	0.41			
Smdbar	9.5	mm	RSMD 35.33	mm

Unit hydrograph option 0

Unit hydrograph time to peak 0.00 Data interval 3
Design storm duration 51 hours

Return Period for design flood 100.0 years
requires rainfall event depth of 140.0 years

M5-51.0hour/M5-2day = 1.08
MT/M5 = 1.82
ARF = 0.97

M5-2day = 59 mm
M5-51.0hour = 63.6 mm
M140.0-51.0hour =115.5 mm (point)
M140.0-51.0hour =112.4 mm (area)

Rainfall profile option 4

Design storm depth 112.4 mm

Design CWI 119.2

Percentage runoff 52.3 % (PR option 1)

Response hydrograph peak 19.89 cumecs
Baseflow 1.40 cumecs (Baseflow option 6)

Design hydrograph peak 21.29 cumecs

****

CONVOLUTION TABLE

Willingdon Levels - 100 year flood based on a 51 hour storm

10mm 3 hour unit hydrograph (cumecs)

Rain	net rain -(cm)-	18.50	36.90	24.60	12.30																
0.20	0.10	0.54	1.07	0.71	0.36																
0.23	0.12		0.62	1.23	0.82	0.41															
0.30	0.16			0.80	1.60	1.07	0.53														
0.32	0.17				0.86	1.71	1.14	0.57													
0.48	0.25					1.29	2.58	1.72	0.86												
0.67	0.35						1.81	3.61	2.41	1.20											
1.18	0.62							3.18	6.34	4.22	2.11										
1.44	0.75								3.89	7.77	5.18	2.59									
1.62	0.85									4.37	8.71	5.81	2.90								
1.44	0.75										3.89	7.77	5.18	2.59							
1.18	0.62											3.18	6.34	4.22	2.11						
0.67	0.35												1.81	3.61	2.41	1.20					
0.48	0.25													1.29	2.58	1.72	0.86				
0.32	0.17														0.86	1.71	1.14	0.57			
0.30	0.16															0.80	1.60	1.07	0.53		
0.23	0.12																0.62	1.23	0.82	0.41	
0.20	0.10																	0.54	1.07	0.71	0.36
	Baseflow	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40	1.40
	Total flow	1.94	3.08	4.14	5.03	5.88	7.47	10.48	14.90	18.96	21.29	20.74	17.63	13.12	9.36	6.84	5.62	4.80	3.82	2.52	1.76

Percentage runoff is	52.30 %
Peak of response hydrograph is	19.89 cumecs
Baseflow	1.40 cumecs
Peak of design flood	21.29 cumecs

APPENDIX C

INFRASTRUCTURE COSTS

Introduction

C.1 This appendix provides details of a method to assess developers contributions to the infrastructure costs of a flood alleviation scheme for the Willingdon Levels catchment. As the actual costs are not available and the 100 year flood level and effective outlet capacity are uncertain, assumptions have been made to demonstrate the method proposed. Clearly other methods of assessment could be made but the method outlined below is reasonably straight forward and should be understood comparatively easily.

C.2 We believe that it would be unreasonable and inequitable to charge new developers for the cost of the works required to make good the increased risk of flooding caused by previous development. We have therefore assumed that a substantial proportion of the infrastructure costs that would result from the works required to achieve the chosen flood standards will be met from general funds. The method of assessment detailed below is based on the impact a given development would have on existing flood conditions and the costs of the works required to alleviate the additional flood problems.

Existing situation

C.3 When the flood runoff from the Willingdon Levels catchment exceeds the combined outlet capacities of West Langney and Crumbles Sewers, the Levels act as a natural flood storage reservoir. We calculate that, over a tidal cycle during a major flood, the average combined effective outlet capcity of the two sewers is of the order of 2.0 to 3.5 m^3/s. This capacity is insufficient to cope with flood runoff from the catchment and according to the Eastbourne Park District Plan report, a water level of:-

(1) 2.0 m OD is not an unusual occurrence; and

(2) 2.9 m OD is possible in a rare event.

Therefore, for the purposes of assessing developers contributions, we have assumed that 2.9 m OD is the limit of the existing flood plain.

C.4 If no flood alleviation works are implemented, any future urban development within the catchment draining to Willingdon Levels will exacerbate the primary flood problem by increasing the total volume of flood runoff (see Section 3.7). In addition, any development on land on Willingdon Levels which is currently below 2.9 m OD will also worsen the flood problem by reducing the natural storage available on the flood plain. Therefore, whether or not a particular development should be allowed, and what infrastructure costs should be levied must depend on:-

(i) the impact the development would have on existing flood conditions; and

(ii) the feasibility and costs of the works required to alleviate the additional flood problems.

Possible solutions to the flood problems

C.5 In the EPDP report the Council set the design standard that the maximum water level on Willingdon Levels must not exceed 2.0 m OD during a 100 year flood. We calculate that the proposed flood storage lakes alone will be insufficient to reduce maximum flood levels to 2.0 m OD and that additional outlet capacity will also be required.

C.6 We calculate, for example, that with a normal retention level of 1.36 m OD in the proposed lakes the design standard could be met if:-

(i) there is about 980 Ml of storage between 1.36 and 2.0 m OD; and

(ii) the average effective outlet capacity over a tidal cycle during the design flood is $7m^3/s$.

C.7 Other combinations of flood storage volume and outlet capacity are possible and the Council could adopt different design standards. In either case the values used in the calculations set out below would need to be modified, but the general principles will stay the same.

Impact of different classes of development

C.8 There are three classes of development to be considered:-

(1) Class A: development on higher land above the level of the existing flood plain (ie. above say 2.9 m OD);

(2) Class B: development of land on the existing flood plain between the selected maximum design flood level (i.e. 2.0 m OD) and 2.9 m OD; and,

(3) Class C: development of land which is below 2.0 m OD.

Each will have a different impact of flood conditions within the catchment.

C.9 Development of Class A land will increase the flood runoff volume during the design event but will not reduce the existing flood plain storage or directly effect the passage of water across Willingdon Levels. The increase in flood runoff volume will depend on:-

(i) the total area developed and the portion of impermeable surfaces;

(ii) the SOIL type of the developed land as defined in Flood Studies Report (Ref 3); and

(iii) the critical storm duration.

Annex 1 provides an example of how the increased runoff volume can be calculated and what additional outlet capacity would be required as a result to maintain the design flood standard.

C.10 Class B land is currently part of the flood plain, and therefore subject to inundation, but would be above the design flood level if a flood alleviation scheme was implemented. It is a matter for debate as to whether the developers contribution should be assessed on the basis of the impact any development would have on existing flood conditions that would prevail after the proposed flood alleviation scheme had been implemented.

C.11 Under present day conditions the development of any land below 2.9 m OD would increase the flood runoff volume and, more importantly, would reduce the natural flood plain storage, thereby affecting the passage of floods across Willingdon Levels.

C.12 After the proposed scheme is implemented, because land between 2.0 and 2.9 m OD would be above the 100 year flood level, the impact of any developments would be exactly the same as for Class A developments. Class B developments would, however, also effect the passage of floods with return periods of greater than 100 years.

C.13 The use of existing (ie. 1987) conditions as the reference standard would produce a significantly higher contribution and we believe that this is equitable as the prime beneficiaries of the proposed flood alleviation scheme would be the developers of land lying between 2.0 and 2.9 m OD which is currently subject to planning restrictions.

C.14 Annex 2 shows an example of the calculation of the increased runoff volume and the loss of natural flood plain storage which would result from the development of Class B land, together with the additional outlet capacity that would be required to maintain the design standard.

C.15 Urban developments on Class C land should be discouraged because:-

(i) they would reduce flood plain storage even with the proposed scheme; and,

(ii) most of this land part of the core of the Levels and will thus be required for the amenity lakes and associated flood channels. Any loss of the land is likely to increase scheme construction costs.

C.16 Any requests for the development of this land should be considered as special cases. If it were decided to allow urban developments on Class C land, as might be the case on the margins of the Broadwater sector, then the impact could be determined as for Class B land and then an appropriate surcharge imposed. Note, these changes should also be applied to any landfill development.

Charges to the developer

C.17 To determine the charges to the developer, it is unrealistic to cost small incremental increases in outlet capacity. It will be necessary to estimate the development that is likely to take place in a reasonable time span (say 15 years) and then calculate the outlet capacity required to pass the 100 year flood at a level of 2.0 m OD after that estimated development has taken place.

C.18 For the purposes of this demonstration calculation we have assumed that:-

(i) the existing outlet capacity over a tidal cycle is 3 m^3/s;

(ii) the outlet capacity currently required to meet the design flood standard is $7m^3/s$; and

(iii) the outlet capacity required in 15 years time will be 8 m^3/s, ie. future development will necessitate an increase in outlet capacity of 1 m^3/s.

C.19 If the cost of increasing the existing outlet from 3 m^3/s to 8 m^3/s if £X, then the cost is split between past and future developers in the ration (7 - 3) : (8 - 7) = 4:1. Thus the total cost to all new developers is £X/5, so that if a proposed development would require an outlet capacity of 0.1 m^3/s then the developer would be required to pay £X/50.

C.20 Clearly X will vary with time and it would be equitable for the developer to pay for inflation between the date that the outlet works are augmented and the date of the development.

ANNEX 1 - IMPACT OF URBAN DEVELOPMENT ON CLASS A LAND

With an average effective outlet capacity of about 7 m^3/s over a tidal cycle, the critical storm duration for a 100 year flood for the Willingdon Levels catchment is about 31 hours. Based on the procedures contained in the Flood Studies Report (FSR) and its supplements the total catchment rainfall during the design storm is calculated to be 100.5 mm.

The FSR divides areas into five basic 'soil' (or Winter Rainfall Acceptance Potential) classes. Our hydrological studies assume (see Sections 5.6 and 5.7) that two of these classes are represented in the Willingdon Levels catchment:-

(1) Soil Class 1 (Chalk / Lower Greensand);

(2) Soil Class 5 (Gault Clay / Weald Clay and Alluvium).

During the design storm we calculate that approximately 16% of the rainfall will runoff from the undeveloped areas on soil class 1 and 59% of the rainfall on soil class 5.

Data from fully sewered catchments suggests that the so called 'impervious' surfaces of an urban area produce a percentage runoff of 70% (Refs 4 and 5). Therefore if 20 ha of each soil class is covered by urban development and 50% of the development is comprised of 'impervious' surfaces then the additional flood runoff volume during a 100 year flood based on a 31 hour storm would be:-

(1) 5.4 Ml from the soil class 1 land; and

(2) 1.1 Ml from the soil class 5 land.

Based on trial routings we estimate that each additional 5 Ml of runoff will increase the required outlet capacity by 0.045 m^3/s.

ANNEX 2 - IMPACT OF URBAN DEVELOPMENT ON CLASS B LAND

Any development which results in the filling of land on the flood plain will reduce the natural flood storage and hence raise the design flood level if no remedial action is taken. The reduction in storage volume caused by any development could be determined from:-

(i) a local site survey, if the existing topographic survey data is inadequate; and,

(ii) the plans for the proposed development.

If, for example, 1 2 ha site with a mean level of 2.15 mOD were built up above the flood level of 2.9 m OD then 15 Ml of natural flood plain storage would be lost. Similarly if a 2 ha site with a mean level of 2.4 mOD were built up above the flood level of 2.9 m OD then 10 Ml of natural storage would be lost. Trial routings show that each 10 Ml loss of storage increases the required outlet capacity by 0.09 m^3/s. There would also be additional runoff from the development site as for Class A land, which would marginally raise the required outlet capacity.

APPENDIX D

FURTHER WORK REQUIRED

Introduction

D.1 To identify the optimum flood alleviation strategy for Willingdon Levels additional studies will be required. A number a key studies which were identified during the current review study are outlined below.

Detailed survey of the whole of Willingdon Levels

D.2 In order to define in detail the existing flood problems of Willingdon Levels and its margins, a detailed togographic survey is required of the whole area (including the low lying areas below, say, 3.0 m OD upstream of Shinewater Bridge) to the standard of the recent 1:500 scale survey of the Shinewater Sector. In view of the apparent discrepancy between field levels in the Shinewater Sector on the recent 1:500 scale survey maps and the older 1:2500 plans (see section 4.5) particular care should be taken to ensure the new maps are to the correct datum.

D.3 From these maps it would be possible:-

(1) to identify the numbers of properties that would be flooded at different flood levels;

(2) to compute an elevation / area, or elevation / available storage, curve for the whole flood plain upstream of Langney Bridge / Crumbles Sluice of the type shown in Section 4.4. of the main report.

Definition of the existing flood problem

D.4 Section 3.11 of the main report lists the work required to define the maximum flood level on Willingdon Levels during a 100 year flood with existing catchment conditions. The process should be repeated for floods of lesser return period eg. 10 and 50 years, to define the relationship between flood return period and the maximum flood height on Willingdon Levels. This relationship, together with a knowledge of the numbers of properties that would be flooded at different flood heights, would define the extent of the primary flood problem.

Confirmation of catchment areas

D.5 There is some uncertainty about the size of the catchment draining to Willingdon Levels (see Sections 3.2 to 3.4). The location of the catchment boundary between Central Eastbourne and Roselands appears to be the source of the descrepancy between the value of $27.9km^2$ quoted in this study and the area quoted in the EPDP report. The catchment boundary in the coastal area should be defined on large scale maps and checked by visits to key sites to define the areas draining:-

(1) to Willingdon Levels;

(2) to Crumbles Sewer downstream of Crumbles Sluice;

(3) directly to the sea through the sands and gravels.

Effective outlet capacity of Crumbles Sewer

D.6 The effective outflow capacities of West Langney and Crumbles Sewers during major floods are uncertain. To refine the estimate of the outlet capacity of West Langney Sewer would entail a detailed hydrology and hydraulic study of conditions on the Pevensey Levels. This work would be largely irrelevant if it were decided:-

(1) to construct a new outlet to the sea, via Crumbles marina; and

(2) to construct a new sluice on West Langney Sewer to restrict flows across Mountney Level.

However, to help determine the optimum size of any new outlet it will certainly be necessary to refine the current estimates of the effective outlet capacity of Crumbles Sewer (ie between 0 and $1.5 m^3/s$). To reduce the range of uncertainty it will be necessary to have more detailed information on the channel geometry, and likely flood inflows between Crumbles Sluice and Princes Park.

Optimum size of the flood alleviation scheme

D.7 To size the flood alleviation scheme it is necessary to define the relationships between:

(1) the flood storage available in the proposed amenity lakes and adjacent water meadows;

(2) normal retention level in the lakes;

(3) effective outlet capacity;

(4) flood return period; and,

(5) maximum flood height.

Figure 7.3 illustrates the relationship between flood height and outlet capacity for a 100 year flood assuming 112 ha of storage lakes with a normal retention level of 1.36 m OD. A series of similar diagrams should be produced to embrace a much wider range of alternatives so that the most appropriate flood alleviation scheme for Willingdon Levels can be evaluated.

D.8 When determining the exact form of the scheme it is most unlikely that it will be sufficient to base the design on economics alone. It will be necessary also to take a broad view in order to give proper emphasis to amenity considerations and the interests of relevant authorities such as the Borough of Eastbourne, Southern Water, and the Marina operators.

Effect of normal retention levels on secondary flooding.

D.9 In section 3.7 of the main report and in Appendix A we mention the local surface water drainage problems which arise in the urban areas around Willingdon Levels as the result of the limited sizes and gradients of various storm water sewers. The possibility exists that the effectiveness of the local surface water drainage system is significantly reduced by the high water levels which are currently maintained throughout the arterial drainage system across Willingdon Levels during the summer months. At present during the winter months, levels are allowed to fall to aid the discharge of floodwaters. Before a normal retention level of, say, 1.4m OD is adopted for the proposed lakes throughout the whole year, it should be established that the selected water level will not have any adverse effects on surface water drainage from the urban areas.

Lake system hydraulics

D.10 In Section 8 of the main report we provide general guidance on the type of cross drainage arrangements needed to interlink the proposed flood storage lakes and the arterial drainage channels. Once maximum flood levels and the lake areas are fixed a more detailed study of the cross drainage arrangements will be required to ensure that the various channels, culverts and weirs are sized to ensure that the lake system operates in the most appropriate manner.

Water quality in the proposed lakes.

D.11 In the EPDP report, little attention was given to the question of the water quality in the proposed lakes, which is likely to decline throughout most summers. It will be necessary to investigate the most appropriate method of maintaining satisfactory water quality standards in the lakes. This will involve both consideration of appropriate methods before construction starts and provision to modify the arrangements in the light of operating experience.

Outlet arrangements from Willingdon Levels to Crumbles Marina, and from the Marina to the sea.

D.12 It will be necessary to make a detailed study of alternative outlet arrangements from Willingdon Levels to Crumbles Marina and from the Marina to the sea. This will involve both the sizing of the various channels and outlet arrangments and also consideration of sediment and water quality aspects.

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