G.P.-S. 028-0365

TR 116

REPUBLIC OF SOUTH AFRICA

DEPARTMENT OF WATER AFFAIRS

Documentation of the January 1981 floods in the South Western Cape

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DEPARTMENT OF WATER AFFAIRS

Division of Hydrology

Technical Report No TR 116

DOCUMENTATION OF THE JANUARY 1981 FLOODS IN THE SOUTH WESTERN CAPE

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ISBN 0 621 07595 7

ACKNOWLEDGEMENTS

The flood surveys and analyses were undertaken by the staff of the Division of Hydrology, and without their enthusiastic co-operation this report would not have been possible. They are mentioned by name at the end of this report.

Thanks are due to the Weather Bureau at Pretoria for providing rainfall data, the Department of Roads of the Cape Provincial Administration and the Chief Civil Engineer of the South African Railways for supplying bridge plans. The co-operation of these authorities has greatly added to the scope and value of the documentation.

Mr W.J.R. Alexander : Manager Scientific Services and Dr C.P.R. Roberts : Chief Engineer Special Tasks of this Department made valuable comments on the report.

The cover photographs and photographs No. 2, 4, 5, 14, and 15 were taken by Mr W.J.R. Alexander. The LANDSAT images were obtained from the Satellite Remote Sensing Centre of the Council for Scientific and Industrial Research.

CONTENTS

1. INTRODUCTION

The storm which occurred during the weekend of 23-25 January 1981 in the South Western Cape has at places produced the heaviest rainfall ever recorded in the region. The ensuing floods were amongst the relatively largest recorded in South Africa and claimed more than a hundred lives. Damage and destruction in urban and agricultural areas and to roads and railways amounted to approximately R100 million. The approximate flood stricken area is shown in Figure 1.

The most tragic aspect of the floods was the destruction of a large part of Laingsburg by the Buffels River resulting in the loss of 104 lives. (Photo's 1-4). The Laingsburg disaster coupled with great havoc in many other districts, such as Montagu where several persons died, focused public attention on the event in a hitherto unparalleled measure. Never before have the media in South Africa given such an extensive coverage to a natural catastrophe. Within months after the floods two books were published on the subject. (1, 2). Thanks to the media interest, private and public assistance was mobilised to compensate for the damage and initiate the redevelopment in the affected zones.

The January 1981 event in the South Western Cape will be remembered for a long time. It will become a basis for comparison in the future, both for laymen and professionals. Consequently, technical, scientific and economic information and analyses that could be derived from the floods may prove to be of great aid for enhancing the reliability of solutions for design flood problems, particularly of those located in the region. The Division of Hydrology recognized the importance of this aspect and immediately after the floods embarked upon a field survey covering an area of more than 50 000 km^2, roughly as shown in Figure 1. The main survey lasted about six weeks and more than 20 civil engineers, hydrologists and technicians were engaged. Data were measured or obtained at 86 sites on 58 rivers. The sites included river gauging weirs, dams, slope-area reaches and bridge contractions. In addition to the Weather Bureau and Departmental stations, rainfall depths were collected at more than 100 farms in the region.

This technical report presents the results of hydrological and hydraulic calculations and analyses based largely on the field survey. In compiling the report the underlying criterion has been to furnish the catchment rainfall data (depth and return period) and flood peak data (peak discharge and return period). Wherever possible flood hydrographs, volumes and runoff percentages were determined. The report also includes results of a limited number of sediment surveys.

Considerable effort was made to carry out the field survey with care and detail and to check all calculations thoroughly. Methods or techniques employed are explained in detail together with their limitations. The calculated flood peak were given an accuracy rating in order to convey to the reader a realistic idea about the accuracy of the results.
Most of the results are summarized in Appendix I with the view to facilitate easy review and comparison.

In Appendix II the application of the slope-area method is illustrated with a fully worked out real case. In Appendix III the various flow types occurring at bridge contractions are shown and the hydraulic principles of flood peak calculation for each type are summarised.

The report contains photographic documentation aimed at showing some of the consequencies of the extraordinary floods and the reaches in which the flood surveys were carried out.

In this report the Laingsburg disaster was not treated in detail because this has already been done by others. Special attention is drawn in this respect to the report and article by W.J.R. Alexander and C.P.R. Roberts who dealt with the complex flood problems and the redevelopment of the town (3, 20).

It is hoped that the flood information published in the report will prove to be useful material for future flood studies.

2. THE STORM

2.1 Synoptic situation

The meteorological causes and synoptic sequence of events were described in detail by K.E. Estiê in reference (4). The following information was taken from above reference with the author's permission.

The cause of the heavy rainfall over the area was a typical black south-easter synoptic situation which had developed over the south-western parts of the country during the weekend of 24-25 January. This situation is one in which a strong high south of the continent feeds moist, warm air into a low-pressure area over the southern parts of the country. Such a low-pression region is also reflected in the upper air in the form of a cold cut-off low which extends well into the upper troposphere.

The development of the situation is illustrated by the synoptic weather maps of the Weather Bureau issued at 12h00 GMT on 23 to 26 January (Figure 2a - d).

During the morning of the 23rd a cold front was situated off the Cape South Coast and extended to north-west of Marion Island. This system was followed by an unusually strong South Atlantic high centred to the north-east of Gough Island. The surface low over the interior was located over the southern Orange Free State. In the upper air a broad trough of low pressure was present to the west of the country (Figure 2a).

By early morning on the 24th the Atlantic high had ridged in south of the continent and the cold front moved to the north-eastern regions of the Karoo. The passage of the cold front on the night of 23/24 January was associated with fairly heavy rain along the South Coast and lighter rain in the southern Karoo. The position of the trough on the surface changed very little, but a strengthening cyclonic circulation around the low over the northern Cape together with the presence of the cold low in the upper layers caused a stronger influx of moist air over the southern parts of the Cape Province (Figure 2b). Later on the 24th heavy rains again occurred in the George-Mossel Bay area and in a belt extending north-westwards across the Langeberg towards the inland regions as far as Ladismith and the mountains of Boland. Also during the 24th the surface low moved westwards and merged with the upper air low which meanwhile became a cut-off low and was gradually moving further into the interior. In the Atlantic High ridge a separate high pressure cell was forming south-east of the continent. The pressure gradient between the low over the interior and the ridge south of the continent become steeper and the south-easterly winds over the Cape South Coast were bringing a stream of moist and relatively warm air into the low which on the afternoon of the 25th was situated over the Cape weatern interior (Figure 2c).

Areas south and south-east of this position were favourably situated for heavy rain which was recorded in a broad belt extending from north-east of Laingsburg south-westwards to the Boland mountains.

Overnight on 25/26 January the steep pressure gradient over the southern coastal regions collapsed. The cut-off low, deprived of its source of energy, started filling rapidly and moved gradually in a southerly direction (Figure 2d).

The author added at the end that the development of black south-easter conditions over South Africa is not uncommon, but it generally takes place further eastwards so that the south-eastern regions of the Cape Province are more prone to heavy rains associated with such systems.

2.2 Rainfall

2.2.1 Area and duration

A few days after the storm all available rainfall data were obtained from the Weather Bureau at Pretoria. These included daily rainfalls at about 100 stations and continuous record at six stations. From above information the approximate boundaries of the storm were identified in south-north direction as the coast and latitude 32° and in the west-east direction as longitudes 19° and 23°.

The daily rainfalls recorded between 23 January 08h00 26 January 08h00 at towns and known places in the region are listed in Table 1 and give an indication of the daily distribution of rainfall over the area.

TABLE 1 : DAILY RAINFALLS AT SELECTED STATIONS IN THE STORM ZONE

Place	Lat.	Long.	Daily rainfall (mm)			3 - day rainfall (mm)
			23 Jan	24 Jan	25 Jan	
Barrydale	33° - 54'	20° - 44'	9	65	32	106
Beaufort West	32° - 18'	22° - 40'	1	48	36	85
Calitzdorp	33° - 32'	21° - 41'	17	60	51	127
Ceres	33° - 20'	19° - 22'	0	1	55	56
Floriskraal Dam	33° - 17'	20° - 59'	6	36	10	52
George	34° - 00'	22° - 23'	58	93	40	191
Ladismith	33° - 29'	21° - 16'	13	91	34	138
Leeu Gamka	32° - 47'	21° - 59'	0	43	48	91
Matjiesfontein	33° - 14'	20° - 35'	45	59	34	138
Merweville	32° - 26'	21° - 36'	8	96	89	193
Montagu	33° - 47'	20° - 07'	2	37	41	80
Mossel Bay	34° - 10'	22° - 08'	44	100	26	170
Oudtshoorn	33° - 36'	22° - 12'	10	55	13	78
Paarl	33° - 43'	18° - 58'	0	0	67	67
Riversdale	34° - 06'	21° - 16'	24	56	26	106
Robertson	33° - 48'	19° - 53'	8	36	70	113
Swellendam	34° - 02'	20° - 27'	32	56	49	137
Touwsrivier	33° - 21'	20° - 03'	5	35	80	120
Worcester	33° - 39'	19° - 26'	13	10	80	103
Villiersdorp	33° - 59'	19° - 18'	8	0	142	150
Van Wyksdorp	33° - 44'	21° - 25'	15	83	31	129

Notes : (i) The dates refer to the 24-hour period starting at 08h00 on that day.
(ii) Rainfalls were rounded off to the nearest mm.

It seems from the table that at most places the major portion of the rain fell during the second and third day. Nevertheless, in the south-eastern segment of the area, shown in Figure 1, it rained heavely on the first day as well. This was also the case at Matjiesfontein which lies between Touwsrivier and Laingsburg. The heavy rainfall recorded during the last two days occurred in the various places at different times. From autographic records of Weather Bureau stations and from local witnesses it appears that in most areas the fairly high-intensity rain began during the evening of the 24th and with shorter periods of softer rain continued through the night and next morning. At many of the places in Table 1 the highest precipitation was recorded in this period (second day). Riversdale was one of these as illustrated by its accumulated rain shown in Figure 3a. In the districts of Laingsburg, Worcester, Touwsrivier and Robertson, where the greatest destruction took place, the extreme floods were caused by relatively short duration heavy storms falling during the afternoon of 25 January (Figure 3b).

In selecting a representative storm duration for the drawing of a storm isohyetal map, for the calculation of catchment rainfall and for the estimation of return periods the most logical choice would have been - in the light of Table 1 - to use the 2-day rainfalls recorded on 24-25 January. However, the difficulty arose that many of the Weather Bureau stations recorded only the total weekend rainfall corresponding to a 3-day period. It was assumed that most of the extra information which had to be gathered during the flood survey from farmers would also be limited to the total weekend rainfall. It was therefore decided to use the 3-day rainfall for all the above mentioned purposes.

2.2.2 Observations and 3-day storm isohyetal map

Once the approximate limits of the storm area and that of the flood survey (Chapter 3) were delimited, the gathering of additional rainfall data was begun. These were obtained (1) from Weather Bureau stations run by voluntary observers, (2) from Departmental rainfall stations and (3) from various individuals, mostly farmers, who were contacted during the flood survey.

The data collection ended with 3-day rainfalls available at more than 300 points covering an area of about 75 000 km^2. The observations were plotted in Figure 4 and reasonably realistic isohyetes were derived. At points where the rainfall on 25 January is known, it is indicated between brackets.

In Figure 4 the maximum 3-day rainfall cells lie north-west of Robertson in the Breë River catchment and north of Laingsburg in the Buffels River catchment. In these areas more than 250 mm rain fell. The maximum observed point rainfalls totalled 375 mm and 288 mm respectively. The 3-day storm rainfall in the Buffels River catchment upstream of Laingsburg was comparable to the mean annual precipitation (MAP) in that area. From the available highest 1-day rainfalls recorded in the vicinity it appears that 60 to 80% of the 3-day rain fell on 25 January causing some of the relatively highest flood peaks in the country. Figure 4 also shows that besides these celles of maxima there were four more distinct cells situated far from each other in the Villiersdorp, Riversdale-Barrydale, Ladismith and George regions. Except in the Villiersdorp region the share of the last day in the total rainfall was much less than in the Robertson and Laingsburg regions.

In Figure 5 the rainfall-area maps of the recent storm and the well remembered 26-27 March 1961 "Karoo" storm were compared. The isohyetal map of the latter was taken from (5). The areas receiving 50 mm or more were 65 00 km^2 in 1961 and 45 00 km^2 in 1981. The mean storm rainfall over these areas was 80 mm in 1961 and 110 in 1981. In spite of lower storm precipitation the 1961 flood peaks were in several rivers the highest ever remembered, especially in the Ongers River. The reason is that the main storm of 26-27 March 1961 was preceded by good rains, while the

January 1981 storm fell on dry catchments. This was a normal condition in a winter-rainfall zone where December and January are the driest months of the year with very low or negligible rainfall.

2.2.3 Return period of point rainfalls

Figure 6 shows the expected 3-day 200-year point rainfall isolines (i.e. rainfall expected to occur on the average once in 200 years) based on Adamson's analysis (6). Owing to the great number of stations, the long records and the method of analysis employed, Adamson's work gives by far the most reliable rainfall frequencies for storm durations of one to seven days in South Africa and South West Africa (Namibia). The shaded surfaces in Figure 6 indicate areas where the 23-25 January rainfall was equal or greater than the 3-day 200-year point rainfall. The total shaded surface is approximately 4 000 km^2. It should be noted that the total surface where the 3-, 2-, 1-day and shorter duration rainfall between 23-25 January was more than the 200- year figure corresponding to these durations, was certainly larger than 4 000 km^2.

2.3 Catchment rainfall

The catchment rainfall (i.e. mean rainfall during 23-25 January over a particular catchment) was required for the calculation of the runoff percentage at all flood peak measurement sites where a reliable flood hydrograph was recorded. It was also needed for a qualitative comparison of flood peaks, especially in neighbouring catchments of roughly the same area. The return period of the catchment rainfall was regarded a useful parameter to compare with the estimated return period of the flood peak obtained by an entirely different approach (Chapter 4.2).

2.3.1 Depth

This was computed from the 1:250 000 scale master plan of Figure 4 by planimetring the areas between isohyetes over the respective catchments of all 86 flood peak measuring sites. The results are listed in column 15 of Appendix I. The accuracy of the results is strongly linked to the density of rainfall observations and the variability of the general rainfall pattern. As the average density of observations is close to 1 in 250 km^2, in catchments much smaller than 250 km^2, say 100 km^2, the accuracy is unknown. In the rest the accuracy should be reasonable to good. The only exception is perhaps a zone south of the lower Touws River situated broadly between longitudes 20° and 20°30'. In that zone too few observations could be gathered.

The computed catchment rainfalls will be discussed further in Chapters 4.3 and 5.1.

2.3.2 Return period

This was estimated for 84 out of the 86 sites in the following way :

1. In addition to Figure 6 similar maps were drawn for the 3-day, 10-year and

50-year return period. The data were taken from (6).

2. The mean point rainfalls were determined in each catchment by planimetring.

3. Using the areal reduction factor diagram from Figure 7 the mean point rainfalls were converted into mean areal rainfalls over each catchment.

4. The mean areal rainfalls associated with T = 10 year, 50-year and 200-year return periods were plotted on log-Normal probability paper. The three points were connected linearly and extrapolated into the $2 < T < 10$ year range for each catchment.

5. Catchment rainfalls listed in column 15 of Appendix I were projected to the respective frequency lines and the corresponding return periods were read off. These are listed in column 16 of Appendix I. Return periods longer than 200 years were not specified. The reason is that frequency distributions - theoretical or empirical - should not be extrapolated beyond a period approximately twice to three times the record length. In this case the 200 year limit was based on the 40 to 100 year record lengths used in Adamson's analysis. Note also that as yet no reliable regional upper limits of areal rainfall have been established in South Africa. It was decided not to use terms like regional probable maximum precipitation (PMP).

3. THE FIELD SURVEY

The area covered by the survey is shown in Figure 8. In broad terms it coincides with the area of storm (Figure 4). The flood peak measurement sites appear in circles and are numbered from 1 to 86. The selection of sites was based on a preliminary storm isohyetal map and an aerial and ground inspection of the flood ravaged areas. The Laingsburg district was flown over immediately after the floods by the Manager : Scientific Services and the Chief Engineer : Special Tasks. Later the whole area was inspected from the air by senior staff of the Division who also inspected river reaches most affected by the floods from the ground.

In Figure 8, apart from the flood peak measurement sites, the 23-25 January storm isohyetes taken from Figure 4 and the boundaries of the main drainage regions are also shown. It was decided not to include the coastal area around George in the survey in spite of the fairly high storm rainfall recorded, because similar events are not uncommon in that zone.

Due to the preliminary inspections and the time required for the organization of staff and equipment the field survey could only begin three weeks after the floods. Ideally such surveys should start soon after the flood when the rivers have dropped to their normal level and the roads are negotiable. Fortunately the delay in this case did not bring with it disadvantageous consequences as flood marks could still be found with relative ease.

The field survey comprised river gauging stations, dams, slope-area reaches and bridge sites. By far the largest share of field work was needed for the two latter kinds of survey, and for this purpose the whole flood region was divided into five separate sections. Five surveying teams were formed each consisted of four to five members under the leadership of a person with adequate field experience.

3.1 Slope-area stations (SA)

Total number : 27

At slope-area stations the field work included the survey of a longitudinal flood profile defined by flood marks, two to five cross-sections and the estimation of roughness at each cross-section. Photographs were also taken and in most cases a sketch of the whole reach was drawn.

Slope-area surveys were carried out principally in those rivers where the preliminary inspection indicated a high or exceptionally high flood peak (Photo 5). In many cases the survey was conducted near to existing river gauging weirs which were damaged or washed away by the flood (Photo's 6 - 8). The approximate sites of slope-area stations were located during the preliminary visit or from 1:50 000 scale maps. The final position of the cross-sections was chosen by the team leader during the survey in such a way as to satisfy, or at least approach, the conditions required for succesful application of the method. These conditions are described in (7, 8).

The surveyed stations are characterized by the following statistics :

- number of cross-sections : 2 to 5, average : 3,2
- length of reach : 100 to 3 200 m, average : 750 m
- width of cross-sections : 20 to 720 m, average : 200 m
- average number of flood marks per station : 21
- average number of ground level points surveyed per cross-section : 20
- average number of photographs per station : 12

Great importance was attached to surveying a large number of flood marks at each station in order to obtain reliable longitudinal flood profiles. Flood marks were also surveyed upstream of the first cross-section and downstream of the last one. As a whole the quality of flood marks was fair to good. It is likely, however, that at sites with very turbulent flow where the velocity was high near the river banks (i.e., in case of steep banks) the flood marks represented a higher level than the true one. (Photo 9).

At cross-sections enough ground levels were surveyed to define the shape of the section adequately. At wide channels, where the flood width was more than about 200 m, it was on occasions impossible to determine the true perpendicular line to the flow direction. This meant that surveyed corss-sectional areas could have been up to 10% larger than the perpendicular one. (Photo 10)

At most stations where the river bed was erodible (sand, cultivated soil, gravel) for instance in the Buffels, Touws, Groot and Gouritz rivers, it was apparent that the surveyed cross-sections were different from the pre-flood conditions. Often the post-flood section showed either signs of fresh sediment deposits or erosion or both. (Photo's 5, 8, 11). When the whole perimeter of the cross-section was formed by fresh deposits, it was generally impossible to establish whether the area was smaller or larger than before the flood. It could have happened that during high flows the section was enlarged by erosion, and later during the rapid decrease of discharge and velocity it was filled with sediments. It is possible that in these rivers the surveyed cross-sections were somewhat smaller than during peak flow. Nevertheless, an examination of photographs and evidence by the surveying teams seemed to indicate that the reduction at most of the doubtful sites was only moderate. The thickness of fresh deposit was generally less than ½ m. At site 84 in the Gouritz River however, the thickness was in the order of 1 m.

The essential field conditions to be satisfied in a slope-area reach are summarised in Appendix II.

3.2 <u>Bridge contractions</u> (B)

Total number : 9

The field work included a survey of flood marks upstream and downstream of the bridge in order to define the drop in waterlevel, the contracted cross-section, an approach section, and sometimes, a typical river section at least 50 to 100 m downstream of the structure. In addition photographs and sketches were made at all bridge sites. In cases when the approach section was fairly representative for the river reach, no extra downstream section was surveyed. For most bridges plans were obtained from the Cape Provincial Administration and the South African Railways. This greatly reduced the field work and was valuable for the estimation of the maximum possible erosion depth at the constriction.

As in the case of the slope-area surveys, bridge surveys were carried out at sites where preliminary inspection indicated high or exceptional flood peaks. (Photo's 12 - 13). Unfortunately, many prospective good bridge sites could not be included because of serious damage or wash-away or heavy or complete blockage of openings by debris. Bridges with more than 25% reduction of openings were not considered for inclusion. Also excluded were those bridges where removal of debris and/or reconstruction took place before the survey (Photo 14). The representative contracted cross-section could not be established with certainty at bridges where the river bed was erodible or the opening(s) were reduced by less than 25% by debris. At five of the nine sites the river bottom was not erodible. At the rest the representative bottom level was assumed to be between the surveyed one and the solid rock level shown in the plans.

The essential field conditions to be satisfied at bridge sites are summarised in Appendix III.

3.3 River gauging weirs (G)

Total number : 32

These included stations where either a reasonable stage hydrograph was recorded or at least the maximum flood level could be established. Many stations were washed away or did not produce an acceptable record due to serious damage (Photo's 6 - 8). As already mentioned, at these sites or in the nearest suitable reach of the river, a slope-area or bridge survey was carried out.

The field work at river gauging stations was limited to the inspection of the station, the retrieval of the autographic chart, and, in some cases, the post-flood establishment of the maximum flood level. This work was carried out by the Sandhills regional office of the Division.

The flood peak at more than the half of the 32 gauging stations was relatively modest. These stations were nevertheless included in the survey because the good stage-hydrograph records provided discharge hydrographs from which flood volume and runoff percentage were computed and the time of peak was fixed.

3.4 Dams (D)

Total number : 18

The field work was similar to that at river gauging stations. At dams where the stage hydrograph was not recorded or did not include the maximum water level, this was established during the survey. An attempt was made to estimate the measure of spillway obstruction by floating debris. At the Floriskraal Dam in the Buffels River (site 67) an extra field survey was carried out in July 1981 to verify the maximum flood level at the spillway and in the immediate upstream and downstream vicinity (Photo 15). These were required by the Division of Special Tasks for a detailed study of the flood at the dam. Plans of some of the oldest dams, such as the Prinsrivier Dam (site 56) were obtained from the Cape Western Circle Office. The information at the Rietvalley farm dam in the Dwyka River (site 72) was obtained from a report by consulting engineers (9) via the Division of Irrigation and Engineering Services.

About half of the dams experienced only moderate floods, but have been included in the survey for the same reason as river gauging stations.

Information on the location of the 86 flood survey sites, the corresponding catchment areas and the type of survey are listed in columns 1 to 7 of Appendix I.

4. FLOOD PEAKS

The primary purpose of the flood survey was to gather sufficient information for the calculation of peak discharges. The value of these is twofold :

(i) coupled with a return period essential information is provided for design flood calculation,

(ii) coupled with corresponding flood level an important, if approximate, point is given for the stage-discharge curve of river gauging stations. The more exceptional the flood peak, the more valuable is the calibration point.

4.1 Methods of peak discharge calculation

4.1.1 Slope-area method

In this method the peak discharge is calculated by a uniform or quasi-uniform flow equation from the flood profile, the cross-sectional area and the estimated roughness. The method is described in detail in reference (7) and summarised in Appendix II where its practical use is illustrated by a worked example. Certain aspects of the application of the method are described below :-

1. The peak discharge was calculated at each slope-area site by the -

   (a) Chēzy equation (i.e., a uniform flow equation) applied at each cross-section with the local flood profile slope (Q_{CH}), and

   (b) the slope-area equation was applied for each sub-reach between two cross-sections. The slope-area equation is based on the Chēzy equation, but the change in velocity-head between the two cross-sections is taken into account. In other words, the flow is treated as gradually varied (Q_{SA}).

2. The slope of the flood profile was fixed after a joint review of plotted flood marks, the plan (or sketch) of the whole reach, the change of cross-sectional area and roughness. Wherever possible one single uniform slope was determined (Figure 9a). This practice was, however, never forced upon the surveyed flood profile and at many stations the established slope consisted of lines of different inclination (Figure 9b, c).

   As mentioned in chapter 3.1, at stations with high velocities and steep-banks or outcrops it is likely that the surveyed flood marks represent a somewhat higher levels than the actual levels. In spite of this no attempt was made to speculate about the probable position of the "true" profile. The slope whether uniform or composite, was drawn through the middle of the longitudinal plot of flood marks. Flood marks surveyed in river bends were adjusted to allow for super elevation.

It is interesting to note that the established flood profile slope at most stations was fairly close to the mean slope of the river bottom derived from 1:50 000 scale maps (Table 2). This is an indirect indication that during large floods the slope of the flood profile tends to approximate the average channel bottom slope.

3. Depending on the denseness of vegetation, a reduction factor of 0,5 to 0,9 was applied to the area below water level of those parts of the cross-section covered by vegetation. For details consult Appendix II. No attempt was made to correct surveyed cross-sections because of presence of fresh sedimentation or for not being perpendicular to the flow direction.

4. The Manning coefficient "n" has been replaced by Chézy's coefficient "C" which was computed from the hydraulic radius and the absolute roughness by formula

$$C = 18 \log \frac{6R}{\varepsilon} \qquad \text{..........} \qquad \text{(Eq. 4.1)}$$

where R = hydraulic radius (m)
ε = absolute roughness (m)

Note that the relationship between n and C is

$$C = \frac{R^{1/6}}{n} \qquad \text{...........} \qquad \text{(Eq. 4.2)}$$

The reason for the preference of the absolute roughness ε was that comparative trial roughness estimations proved beyond doubt that ε could be estimated with greater consistency and ease than n. In the trial 75 independent roughness estimates in five river reaches were made by one experienced and three unexperienced persons. The result showed that the error made by the unexperienced persons relative to the experienced observer was reduced to about half when ε was used instead of n (both estimates were expressed by C). The use of ε is also in line with the modern tendency of using structurally more correct resistance formulae. The disadvantage of using n is, even by an experienced person, that the influence of water depth on roughness remains hidden.

Eq. (4.1) is in good agreement with measurements carried out in characteristic river reaches in Central Europe in the period 1954-1971 (10). It should be noted that the often used Colebrook-White resistance equation

$$C = 18 \log \frac{12R}{\varepsilon} \qquad \text{............} \qquad \text{(Eq. 4.3)}$$

is applicable only in channels where the cross-section and roughness are uniform. In natural channels, especially in South Africa where seasonal flow is characteristic, Eq. (4.3) gives systematic overestimation of C by about 20%.

Figure 10 is a graphical presentation of the relationship between Manning's n and ε in function of the hydraulic radius according to Eqs. (4.4) which is the combination of Eqs (4.1 - 4.2) :

$$n = \frac{R^{1/6}}{18 \log \frac{6R}{\varepsilon}} \qquad \ldots\ldots\ldots\ldots\ldots \qquad \text{(Eq. 4.4)}$$

In selecting absolute roughness ε the following practical rules were observed:

(i) ε_{min} = the smaller of 0,15R or 0,20m in uniform river reaches with a smooth sandy bed. It is based on the appearance of sand-dunes during the rising stage of flood (Photo 16).

(ii) ε_{max} = mean water depth. It was applied mainly in the vegetated parts of cross-sections where the vegetation was higher than the water depth during peak flow conditions (Photo 17).

(iii) ε = 1 to 2 times the diameter of dominant obstacles in non-uniform river reaches where the bottom was undulating (Photo's 18, 19).

5. The conveyance factor $K = CAR^{\frac{1}{2}}$ was calculated separately for each sub-section of a cross-section. The Chézy coefficient C was calculated separately for each sub-division of a cross-section (Appendix II, Figure 1).

6. The mean peak discharge at a station was calculated as

$$\overline{Q}_{CH} = \frac{\sum_{1}^{N} Q_{CH}}{N} \qquad \text{from the Chézy equation.}$$

and

$$\overline{Q}_{SA} = \frac{\sum_{1}^{N-1} Q_{SA}}{N-1} \qquad \text{from the slope-area equation}$$

where N = number of cross-sections.

Anomalous values of Q_{CH} or Q_{SA} were discarded in the calculation of mean values. Generally $\overline{Q}_{CH}$ and $\overline{Q}_{SA}$ were very similar. In such cases the representative peak discharge at the station was taken as the mean of $\overline{Q}_{CH}$ and $\overline{Q}_{SA}$. At stations where the river reach was far from uniform the difference between $\overline{Q}_{CH}$ and $\overline{Q}_{SA}$ was sometimes considerable. In such cases $\overline{Q}_{SA}$ was chosen as representative peak discharge.

The slope-area calculations were carried out on standard forms shown in Appendix II.

The mean velocity, energy coefficient, Froude numbers and some of the basic hydraulic parameters are given in Table 2.

Noteworthy conclusions from the table are the following.

- The mean velocity never reached 5 m/s in spite of extremely high peak discharges or steep channels.

- The value of the energy coefficient, α, was between 1,2 and 1,8. The often employed practice of using α = 1,0 in hydraulic calculations of natural channels can, consequently, not be justified.

- Unstable or supercritical flow ($F > 0,8$) occurred at only about one third of the sites. These were typically small and steep. At sites associated with large catchments the peak flow was always subcritical, in spite of the occasionally very high mean velocities.

TABLE 2 : HYDRAULIC PARAMETERS AT 27 SLOPE-AREA STATIONS[1]

SITE	N[2]	PEAK[3] DISCHARGE Q (m^3/s)	SLOPE: FLOOD LEVEL (SURVEYED) S	SLOPE: BOTTOM (1:50000 MAP) S_B	CROSS-SECTION A (m^2)	TOP WIDTH T (m)	MEAN DEPTH A/T(m)	MEAN VELOCITY (m/s)	ENERGY[4] COEFFICIENT α	FROUDE[4] NUMBER F
15	2	230	0,0048	0,0086	76,3	50,5	1,52	3,00	1,39	0,92
20	3	418	0,0042	0,0043	234	141	1,66	1,79	1,50	0,54
21	2	244	0,0082	0,0060	78,7	49,3	1,59	3,10	1,79	1,05
25	2	1 450	0,0100	0,0106	362	140	2,59	4,00	1,52	0,98
27	3	588	0,0217	0,0200	143	48,3	2,97	4,11	1,78	1,02
32	3	673	0,0110	0,0051	177	116	1,53	3,80	1,61	1,25
35	3	1 330	0,0025	0,0040	527	164	3,22	2,52	1,46	0,54
52	4	358	0,0026	0,0033	212	107	1,99	1,69	1,70	0,50
55	4	1 670	0,0055	0,0063	557	228	2,44	3,00	1,39	0,72
57	4	840	0,0065	0,0065	316	158	2,00	2,66	1,43	0,72
59	3	3 650	0,0019	0,0034	1 294	383	3,38	2,82	1,44	0,59
61	3	375	0,0046	0,0053	223	151	1,48	1,68	1,83	0,60
62	3	1 230	0,0076	0,0075	344	122	2,81	3,58	1,49	0,83
63	5	4 350	0,0024	0,0033	1 603	397	4,03	2,71	1,39	0,51
65	3	6 020	0,0038	0,0038	1 890	421	4,49	3,19	1,35	0,56
66	3	132	0,0058	0,0060	77,3	112	0,69	1,71	1,78	0,88
68	5	3 630	0,0028	0,0041	967	164	5,90	3,75	1,25	0,55
69	4	4 680	0,0020	0,0038	1 029	107	9,62	4,55	1,19	0,51
70	4	5 250	0,0036	0,0038	1 645	500	3,29	3,19	1,32	0,65
71	4	11 000	0,0026	0,0027	2 255	278	8,12	4,88	1,23	0,61
73	2	930	0,0031	0,0047	530	322	1,65	1,75	1,71	0,57
74	3	2 240	0,0025	0,0023	1 029	441	2,33	2,18	1,64	0,58
77	3	236	0,0106	0,0042	64,1	31,3	2,05	3,68	1,37	0,96
79	4	185	0,0250	0,0312	41,8	20,6	2,03	4,43	1,59	1,25
83	2	105	0,0018	0,0028	103	77,0	1,33	1,02	1,49	0,34
84	3	11 400	0,0022	0,0018	2 369	312	7,58	4,81	1,34	0,65
86	3	660	0,0008	0,0015	520	147	3,54	1,27	1,32	0,25

Notes :

(1) Parameters in columns 4 - 11 are mean values for the station.

(2) Number of cross-sections.

(3) Representative value.

(4) Consult Appendix II for formulae.

4.1.2 Bridge contraction method

In this method the peak discharge was calculated from the difference in flood level between the upstream and downstream sides of the bridge and from the section-parameters (area, perimeter etc.) of the approach section and the constricted section.

There are a great variety of possible hydraulic conditions at bridge contractions and it is of paramount importance that the calculations be based on the condition during a particular flood at a particular site. Peak discharge calculations at bridge contractions carried out in the Division of Hydrology during recent years revealed that practically each site needs an individual solution adapted to the prevailing hydraulic conditions and the quality of available data. Accordingly, standard methods for calculation procedures would prove extremely cumbersome and undesirable. In Appendix III the main types of hydraulic conditions at bridges are shown together with a broad outline of recommended solutions. For more details consult (7, 11, 12).

At the nine bridge sites included in this flood survey the flow-type during peak flow could be established satisfactorily. For the reconstructed hydraulic conditions the peak discharge was calculated by various methods in most cases. It was found that at sites 24, 34, 36 and 64, where the survey provided a well defined flood profile in the vicinity of the constriction, alternative methods gave very similar results. At the remaining sites, where either an insufficient number or poor quality flood marks were available, iteration techniques were employed to obtain a realistic approximation of the drop in water level. These are also mentioned in Appendix III. The lesson learnt from the calculations was that in future the greatest possible number of flood marks, say 15 or more, should be surveyed at bridge constrictions.

The peak discharge calculation at the SAR bridge in Laingsburg (site 64) needs some comments (Figure 9c, Photo 2). Due to the very good flood profile obtained from the adjacent slope-area reach (site 65), the downstream water level could be fixed with fair accuracy as RL 648,3 m. On the upstream side the complex flow pattern (confluence of three rivers) and high surges made it difficult to determine the representative maximum flood level. Information obtained from eye-witnesses, the results of an aerial survey and references (1, 3) seemed to point to a most likely flood elevation of RL 650,6 m. The hydraulic calculations (three alternative methods) were thus based on a difference of 2,3 m. The bed level during the flood could be estimated with reasonable accuracy by comparing the cross-section shown on the original bridge drawing with the contour plan of the aerial survey. The reduction of bridge openings by debris was estimated as 10% from site inspection and photographs. The calculation was complicated by the washing away of the two approach embankments along a total length of approximately 200 m. From accounts from the SAR station-master at Laingsburg and (1) it appears that the collapse of the approaches took place at about the time when the maximum flood level was reached and was followed by a sudden and considerable drop in upstream water level. For this reason the realistic assumption seemed to be not to include the flow through the collapsed embankments in the peak discharge. This assumption was proved to be correct after the calculated peak discharge were compared with those at sites 61, 62, 63 and 65 (Chapter 4.3).

In Table 3 various hydraulic parameters and information at the nine bridges are listed.

TABLE 3 : HYDRAULIC INFORMATION AT BRIDGES

SITE NO.	PEAK DISCHARGE Q (m^3/s)	TOTAL NET WIDTH B (m)	HYDRAULIC DROP Δ Z (m)	CONTRACTION RATIO[1] m	SKEWNESS[2] ϕ ^{o}	FLOW OVER DECK
7	520	175	0,12	0,01	90	no
17	160	11,5	2,18	0,45	44	yes
19	442	51,8	0,92	0,02	29,5	no
24	225	28,3	0,45	0,50	90	no
34	1 460	34,4	2,30	0,48	90	yes
36	1 210	72,0	2,00	0,61	90	negligible
64	5 680	198	2,30	0,60	90	no
76	3 510	91,0	2,20	0,37	90	yes
80	490	40,0	0,30	0,32	70	no

Notes : (1) m = o means no contraction. For details consult Appendix III.
(2) ϕ = 90° means bridge crossing at right angles to flow.

4.1.3 River gauging stations

At river gauging stations the peak discharge was calculated from the recorded maximum flood level (HFL) by using the stage-discharge relationship (DT) valid for the station. At many station the recorded maximum flood level was higher, in spite of rather moderate flood peaks, than the limiting height of existing calibration (HCL). This is seen in Table 4 where HFL and HCL are compared. The problem was solved approximately by a combination of the existing weir calibration and the calibration of the river cross-section (at the weir) using uniform flow principles (13). Though in most cases the above technique yielded acceptable results, the lesson from the flood event is that there is an urgent need for the extension of accurate calibration at gauging weirs.

TABLE 4 : MAXIMUM RECORDED FLOOD LEVELS (HFL) vs LIMIT OF CALIBRATION (HCL) AT GAUGING STATIONS

SITE NO.	HFL	HCL	SITE NO.	HFL	HCL	SITE NO.	HFL	HCL
1	0,84	1,52	22	4,12	4,99	43	1,92	1,52
2	1,44	1,52	23	1,95	2,00	45	2,94	1,15
3	2,32	1,52	26	3,14	1,20	46	5,00	5,40
4	2,50	6,39	29	0,93	1,00	47	1,95	0,78
5	2,03	4,00	30	1,46	1,20	51	1,96	3,60
6	2,60	0,85	33	3,17	1,30	53	1,72	2,59
8	2,18	0,62	37	8,10	8,40	54	0,84	1,00
9	1,82	3,38	38	2,43	2,60	78	1,99	1,21
11	1,42	1,16	39	2,88	1,52	82	1,60	1,36
14	2,42	1,59	40	0,74	0,50	85	3,20	7,62
16	2,26	2,70	41	2,14	1,52			

Note : All levels are in meters and refer to the zero level of the gauging station.

4.1.4 Dams

At dams the peak inflow was calculated from the recorded dam levels, the spillway depth-discharge relationship, other outlet discharges and the dam depth-capacity relationship, by employing the basic continuity equation :-

$$I = \theta + \frac{\Delta S}{\Delta t} \qquad (m/s) \qquad \ldots\ldots\ldots \qquad \text{Eq (4.5)}$$

where I = inflow rate
θ = outflow rate
$\frac{\Delta S}{\Delta t}$ = rate of change in dam storage with respect of time.

With the exception of Floriskraal Dam (site 67) horizontal water levels were assumed. In other words, for a given spillway the spillway discharge was assumed to be a function of the dam water level only. With few exceptions all dams, including those provided with gates, functioned during the flood as uncontrolled spillways.

At all but two dams both the inflow and outflow hydrographs could be calculated. At sites 31 and 72 only the peak outflow could be determined.

In the Buffels River between Laingsburg and the Floriskraal Dam the Division of Special Tasks undertook dynamic flood routing calculations. This method rests on the solution of the basic differential equations for unsteady flow in which not only the continuity, but also the balance of the momentum of flow are taken into account. In other words, the knowledge of cross-section properties, the river bottom slope and the friction slope of the flow are essential. In the exercise the dynamic flood routing was based on the flow hydrograph at Laingsburg (Figure 14i), a number of surveyed cross-sections in the Buffels River and the Floriskraal Dam, stage records in the dam, and model tests on the overspill of the dam. The application of the dynamic flood routing technique for this case is described by C.P.R. Roberts (14).

Under normal conditions, especially in larger dams, level pool reservoir routing (i.e. horizontal dam levels) usually yields reliable flood peaks. On the 25th of January, however, at some of the dams which experienced extreme floods the peak inflow rate was so high relative to the full supply capacity (FSC) of the dam that in all likelihood it caused rapidly varied unsteady flow i.e., surges which were superimposed upon the inflow. This group included the Keerom Dam (site 18), Pietersfontein Dam (site 31), Prinsrivier Dam (site 56) and the Floriskraal Dam (site 67). To emphasize the extreme magnitude of peak inflows these and the corresponding full supply capacities are compared with similar data at two of the largest dams in the country in Table 5. Parameter Q_{max}/FSC is an indicator for surge generation by inflow impact upon the dam.

TABLE 5 : PEAK INFLOW RATE vs FSC AT SIX DAMS

DAM	Q_{max} (m^3/s)	DATE	FSC(5) ($10^6 m^3$)	$\frac{Q_{max}}{FSC}$
Keerom	948(1)	81.01.25	8,4	113
Pietersfontein	(1 000)(2)	"	2,7	370
Prinsrivier	1 030(1)	"	2,9	355
Floriskraal	5 740(1)	"	68,0	84,4
Vaal	5 475(3)	57.09.27	2 330	2,35
H.F. Verwoerd	10 647(4)	67.02.02	5 950	1,79

Notes : (1) Appendix I, column 8 .
(2) Estimated.
(3) Reference (17) : Appendix I.
(4) Reference (17) : Appendix I, recorded at nearby river gauging station D3M02, closed in 1970.
(5) Figures refer to year of completion or last raising of dam.

It appears from Table 5 that Q_{max}/FSC was about 100 times larger in the four relatively smaller dams than in the two large dams during the maximum observed inflow rates. A reliable estimate of surge heights would require systematic hydraulic model tests in which the influence of the inflow rate relative to dam volume, dam geometry, debris barrier at the point of inflow etc. upon surge height could be investigated. Simplified theoretical calculations have indicated that in the four smaller dams the inflow of extreme flash floods could have generated surges of significant heights, say higher than 0,3 m (15). This possibility seems to have been confirmed by accounts of persons washed down by the Buffels River from Laingsburg into the Floriskraal Dam where the water surface was described as very rough (1). The rough surface was probably caused by surges as, according to information from the Weather Bureau in Pretoria, wind speeds in the region were low in the evening and night of 25 January. The presence of surges in dams will cause the recorded stage to be higher than the corresponding representative mean levels, especially during the peak inflow period. This in turn implies that both the reservoir and dynamic flood routing procedures will result in overestimated inflow and outflow peaks. In Chapter 4.3.3 evidence is furnished to support the above suggestion.

During the inflow of extraordinary floods the water surface in dams is not horizontal. If this is ignored in reservoir routing, particularly in smaller dams, it may result in an inflow hydrograph with too short and steep a rising limb and an overestimated peak (16).

In conclusion, the accuracy of peak inflow and outflow obtained by level pool reservoir routing can be expected to deteriorate for extreme floods.

Flood peaks calculated at the 86 sites are listed in column 8 of Appendix I. The circle beside a peak discharge figure indicates that the particular peak was the biggest on record or in living memory.

4.2 Return period

Amongst the 86 sites there were only about 10 where the minimum desirable record length for flood peak frequency analysis, i.e.,20 years was satisfied. Most of these sites did not experience rare floods. It was decided therefore that more benefit could be derived by assigning to each calculated peak a broadly defined return period class than to compute theoretical frequency distributions at the few suitable sites.

The basis for the estimation of return period was the ratio of the calculated peak discharge to regional maximum flood peak (RMF) which can easily be calculated from the Francou-Rodier equation (18).

$$RMF = 10^6 \left(\frac{A}{10^8} \right)^{1-0,1K} \quad (m^3/s) \quad .... \quad \text{Eq (4.6)}$$

where A = catchment area (km^2)

K = regional coefficient. For the area of flood survey K = 5.

On page 13 of reference (17) it was suggested that in South Africa the 200 year peak is about 0,65 RMF. Due to regional differences above coefficient can be expected to vary between 0,55 and 0,75. The lower limit is applicable in the drier parts of the country such as the area of flood survey. Further, by comparing the RMF with empirical flood peak frequency curves at 30 river gauging stations, each with at least 40 years of record, it was found that the 50 year peak, with a few exceptions, corresponded to 0,4 to 0,6 RMF. Again, the lower limit is valid in the drier areas. Finally, a review of annual flood peaks at a number of stations in the surveyed zone revealed that the 10 year peak is approximately 0,1 to 0,2 RMF. Obviously such a rudimentary approach is not suitable for estimating discrete flood peak frequencies. It was, however, judged acceptable enough to classify the surveyed peaks into a few return period categories or to indicate whether a given peak had a return period close to the chosen category limits. The categories fixed were the following :-

	<	10	year
10	-	50	"
50	-	200	"
	>	200	"
	∿	RMF	(without a fixed return period, but probably more than 500-year)

The estimated return period categories are shown in column 9 and the Francou-Rodier *K* in column 10 of Appendix I.

Figures 11a - d show the observed and theoretical flood peak frequency distributions at sites 26, 47, 64 and 78. These were the only stations where the return period of the surveyed flood peak was longer than 10 years and the record of the annual maximum peaks was more than 20 years. The data were ranked according to the Weibull formula :-

$$T = \frac{N+1}{m} \quad \left(or \ \ p = \frac{100}{T}\right) \qquad \ldots\ldots\ldots\ldots\ldots \qquad \text{Eq. (4.7)}$$

where T = return period (year)

p = probability of exceedance (%)

N = length of record (year)

m = rank 1, 2 .... i .... N in descending order.

The theoretical distributions plotted were the LN2 (Log-Normal 2-parameters) and LP3 (Log Pearson III). The continuous lines were calculated for annual peaks including the extraordinary event of January 1981 (N). The dashed lines were calculated without including it (N-1). The plotting positions of the observations correspond to the first case. (The correct plotting for the second case would show a shifting of points to the right). The most striking impression obtained from the graphs is the great sensitivity of the theoretical lines, especially the LP3, to the inclusion or omission of the January 1981 peaks. The difficulty of arriving at reliable flood peak frequency estimates from the curves of Figures 11a - d is illustrated by Tables 6a - b.

TABLE 6a : COMPARISON OF RETURN PERIODS OBTAINED FROM THE JANUARY 1981 FLOOD PEAKS AT FOUR SITES.

SITE NO.	A (km^2)	N (YEAR)	JAN 1981 PEAK / 2ND LARGEST PEAK	ESTIMATED RETURN PERIOD (YEAR)				
				COLUMN 9 APPENDIX I	LN2		LP3	
					N[2]	N-1[3]	N[2]	N-1[3]
26	24	31	13,7	± RMF	>10 000	>>10 000	150	>>10 000
47	28	30	1,69	10 - 50	55	100	35	55
64	3 070	36[1]	7,65	± RMF	110	210	400	>>10 000
78	25	21	14,7	50-100	430	> 10 000	95	>>10 000

Notes : (1) Covers period 1925 - 1981
(2) January 1981 peaks included
(3) January 1981 peaks excluded

It is obvious from above that at sites 26, 64 and 78 where the January 1981 peak was much bigger than the second largest peak in the record (i.e., it was an outlier) the corresponding return periods of the outliers could not be estimated, not even approximately, from the LN2 or LP3 distributions. It is, for instance, particularly disturbing that the LN2 line indicated a return period of only 110 years for the flood peak at Laingsburg (site 64).

The difficulty in deriving realistic flood peak estimates for given return periods is shown in Table 6b.

TABLE 6b : INFLUENCE OF OUTLIERS ON FLOOD PEAK ESTIMATES FOR FIXED RETURN PERIODS AT FOUR SITES.

SITE NO.	A (km^2)	N (YEAR)	JAN 1981 PEAK / 2ND LARGEST PEAK	Q_{TN}/Q_{TN-1}			
				LN2		LP3	
				T=10 y	T=200 y	T=10 y	T=200 y
26	24	31	13,7	1,64	2,38	1,74	12,6
47	28	30	1,69	1,18	1,29	1,18	1,38
64	3 070	36	7,65	1,33	1,57	1,47	2,71
78	25	21	14,7	1,96	3,12	2,34	17,3

Notes : Q_{TN} = flood peak obtained by including outlier in record.
Q_{TN-1} = flood peak obtained by excluding outlier from record.

It is seen that even at site 47 where the outlier was only 1,69 times larger than the next largest peak in the record, certainly not an uncommon case in South Africa, its inclusion meant a 18% increase in the Q_{10} estimate. It appears that except for site 47 the listed Q_{TN}/Q_{TN-1} are prohibitively high.

4.3 Evaluation of results

The evaluation of flood peaks is expressed by an accuracy rating (column 11 of Appendix I). The meaning of the rating symbols is the following :-

Rating	Error in peak discharge
1	less than ± 10%
2	less than ± 30%
u	unknown

In deriving the ratings the main criterion was the quality of the measurement. In addition, comparisons were made with peaks at neighbouring sites and particularly those along the same river.

Special problems related to peak flows at dams are discussed below.

4.3.1 Evaluation at individual sites

Depending on the method of peak discharge calculation the following criteria were used :-

(i) Slope-area method (SA) : Experience with past slope-area meaurements which were checked by other independent measurements (current-meter, dam) showed that in a reasonably good river reach surveys carried out with sufficient care will yield rating '2' or better. To obtain rating '1' a very favourable reach (straight, fairly uniform cross-sections and roughness) and very good flood profile are needed.

(ii) Bridge-contractions (B) : In the absence of comparison with nearby sites a single bridge site can yield a peak with rating '2' only under very favourable conditions (stable channel, 90° crossing, numerous flood marks). Rating '1' requires exceptionally favourable conditions and very well defined flood profile on both sides of the constriction.

(iii) River gauging stations (G) : Rating '1' is justified if the observed maximum flood level was lower or, depending on the shape and size of the flood cross-section at the weir, only moderately higher than the limit of the calibration curve. During the January 1981 flood many gauging weirs recorded much higher stages than the calibration limit. In these cases, in spite of the approximate calibration applied for the weir-section (Chapter 4.1.3), rating 'u' had to be allocated.

(iv) Dams (D) : The reservoir routing procedure was assumed to render rating '1' in the case of small or medium floods (say with return periods of less than 50 y) if the autographic record was of satisfactory quality. In case of rare or extreme floods, owing to conditions mentioned in Chapter 4.1.4, rating '2' seemed to be the appropriate one. Sites 58 and 72 were given rating 'u' because of doubtful or insufficient information.

The accuracy ratings are summarised according to the method of measurement in Table 7.

TABLE 7 : SUMMARY OF PEAK DISCHARGE RATINGS ACCORDING TO METHOD OF MEASUREMENT.

METHOD	NUMBER OF SITES	RATING			NUMBER OR RARE OR EXTREME PEAKS
		1	2	u	
Slope-area	27	2	24	1	14
Bridge	9	2	5	2	3
Gauging station	32	15	10	7	2
Dam	18	10	6	2	5
TOTALS	86	29	45	12	24
%	100	34	52	14	28

The above figures clearly reveal that the ratings of the so called 'indirect' methods i.e., the slope-area and bridge-contraction, compare well with those of the other two more 'direct' methods (the meaning of 'direct' in above context is : The peak discharge can be calculated by the weir-equation from a level or successive levels recorded in a calibrated cross section of regular shape. 'Indirect' implies on the other hand the absence of a calibrated section which is replaced by a large number of less accurate water levels and estimated coefficients in the hydraulic equations). The gauging stations and dams yielded proportionately more rating '1' peaks than the indirect methods. However, most of the extreme peaks were calculated by the latter. The conslusion from Table 7 and also from past experience is that in South Africa the two indirect methods are indispensable for the documentation of rare or exceptional floods and are likely to remain so for some time.

Due to the lack of flood peak frequency data there was no direct criterion for the evaluation of return periods estimated by Eq. (4.6). An indirect way of testing whether return periods (T_Q) listed in column 9 of Appendix I are realistic is to compare them with the return periods of the 3-day catchment storm rainfall (T_p) shown in column 16 of the same table. It is noted that in principle the correct procedure would be to compare T_Q with the return period of the rain which fell during the critical storm duration, which is approximately the time of concentration t_c. Such an investigation will unfortunately remain impossible until rainfall data and depth-duration-frequency analyses can be based on autographic records. For above reason the comparison of T_Q with T_p is expected to be more realistic for the larger catchments where the difference between t_c and 3 days is less than in smaller catchments. The comparison is shown in Table 8 for 84 sites.

TABLE 8 : COMPARISON OF T_Q AND T_p ACCORDING TO CATCHMENT AREA CATEGORIES

CATCHMENT AREA A (km²)	NUMBER OF SITES			
	TOTAL	$T_Q > T_p$	$T_Q \sim T_p$	$T_Q < T_p$
< 100	30	4	12	14
100 - 500	21	0	10	11
500 - 5 000	23	1	11	11
> 5 000	10	2	7	1
TOTALS	84	7	40	37

On the whole the result is satisfactory, for in more than 90% of the catchments $T_Q \leq T_p$. This condition is realistic, because the three weeks preceding the storm were practically dry. Another reason for the $T_Q < T_p$ condition is that at several sites the upstream storage in the catchment resulted in flood peak reduction. Due to the drier than average antecedent conditions the $T_Q > T_p$ case is not realistic.

In small catchments the comparison of T_Q and T_p has less meaning, partly because of the short t_c and also owing to possible errors in the determination of 3-day storm rainfalls as pointed out in Chapter 2.3.1.

4.3.2 Evaluation along rivers

The evaluation of peak discharge accuracy could in some cases be made more reliable by examining flood peaks, rainfall, return periods and other characteristics in conjunction at several stations along a river reach or in zones that experienced similar storm rainfall. This is illustrated along the Buffels-Groot-Gouritz Rivers. The important data and parameters listed in Table 9 were taken from Appendix I. Consult also Figures 4 and 8.

TABLE 9 : SUMMARY OF PARAMETERS FOR THE COMPARISON OF PEAK DISCHARGES ALONG THE BUFFELS- GROOT- GOURITZ RIVERS

SITE NO.	RIVER	METHOD	A(km^2)	Q(m^3/s)	K	P(mm)	T_Q(y)	T_p(y)	ACCURACY RATING
61	Bobbejaans	SA	217	375	3,95	123	10-50	$\geq$ 200	2
62	Wilgehout	SA	361	1 230	4,65	164	> 200	> 200	2
63	Buffels	SA	2 375	4 350	4,89	165	> 200	> 200	2
61+62+63			2 953	5 955					
64	Buffels	B	3 070	5 680	5,02	162	± RMF	> 200	1
65	Buffels	SA	3 072	6 020	5,08	162	± RMF	> 200	1
66	Geelbek	SA	338	132	2,91	72	< 10	20	2
67	Buffels	D(in)	4 001	5 740	4,90	142	> 200	> 200	1
		D(out)		4 620	4,69		> 200		2
68	Buffels	SA	4 005	3 630	4,45	142	$\leq$ 200	> 200	2
69	Buffels	SA	4 700	4 680	4,62	133	> 200	170	2
70	Groot	SA	5 565	5 250	4,64	134	> 200	170	2
59	Touws	SA	5 803	3 650	4,25	166	50-200	> 200	2
60	Brand	D(out)	251	104	2,89	124	< 10	18	2
70+59+60			11 619	9 004					
71	Groot	SA	12 466	11 000	4,98	148	± RMF	> 200	2
76	Gamka	B	17 396	3 510	3,47	70	10-50	14	2
80	Huis	B	390	490	3,88	170	10-50	200	u
81	Nels	D(out)	170	72	2,82	130	< 10	70	2
83	Olifants	SA	10 927	105	0	49	< 10	2	u
71+76+80+81+83			41 349	15 177					
84	Gouritz	SA	43 451	11 400	4,22	91	$\leq$ 200	30	2

Upstream of Laingsburg : sites 61, 62, 63

In catchments 62 and 63 the 3-day storm rainfall (P) was the same and most of it fell during the last day within a period of 10 hours. In catchment 61 the rainfall was less and more evenly distributed over the 3 days. The calculated Q and K seem to be in accord with the rainfall pattern. The greater K at site 63 compared to site 62 indicates that the same rain depth caused a relatively higher peak in a large catchment where t_c is nearer to the storm duration. The catastrophic nature of the rainfall in catchments 62 and 63 is highlighted by the fact that even the 123 mm over catchment 61 was associated with $T_p \geq 200$ year return period. T_p and T_Q compare well in the three catchments. The $T_Q << T_p$ condition at site 61 is explained by the very short t_c relative to the storm duration and that lower rainfall is linked to higher storm loss (Figure 62 in reference (18). The above considerations together with reasonably good slope-area reaches justify an accuracy rating '2' (i.e., error less than 30%).

Laingsburg : sites 64, 65

These sites were just downstream of the confluence of the Bobbejaans, Wilgehout and Buffels Rivers and had practically the same catchment size with was only 4% larger than the sum of sites 61, 62 and 63. Site 64 was at the SAR bridge, and site 65 just downstream of it. It is seen in Table 9 that $Q_{61+62+63}$, Q_{64} and Q_{65} differ less than 10% thereby warranting rating '1' at sites 64 and 65. Should these peaks have been rated solely on individual basis, the rating would have been '2' : T_Q and T_p compare well. (As explained in Chapter 2.3.2 it is preferable to use category T_p > 200 year without referring to maximum regional limits of areal rainfalls which are not yet reliably established in South Africa).

Floriskraal Dam : sites 67, 68

Site 67 is immediately upstream of the Floriskraal Dam basin some 6 km upstream of the dam-wall. The outflow refers to the dam spillway. Site 68 was a slope-area station located just downstream of the dam. From Figures 4 and 8 it is seen that there was relatively little rain over the catchment downstream of Laingsburg. The peak of 132 m^3/s calculated for site 66 which comprises about one-third of the intermediate catchment was modest and suggests that the sum of peaks from the entire intermediate catchment could not have been very different from 350 to 400 m^3/s. Consequently, the peak inflow of 5 740 m^3/s seems fairly accurate with a rating '1'. The relatively large difference between the outflow peak of 4 620 m^3/s and the 3 630 m^3/s calculated at site 68 warranted rating '2' for both. It can be safely assumed that the figure of 4 620 m^3/s is too high, mainly because of surges in the dam (Chapter 4.1.4 and 4.3.3). On the other hand, comparison of respective Q and P at sites 68, 69 and 70 suggests that the true peak at site 68 was higher than 3 630 m^3/s. It is concluded that the actual peak outflow from Floriskraal Dam was in the region of 4 000 m^3/s.

Between Floriskraal Dam and Groot-Gouritz river confluence : sites 68, 69, 70, 59, 71

These were all fair to good slope-area stations where three to five cross-sections were surveyed. The generally unstable channels and, to a lesser extent, the somewhat inconsistent flood marks (caused in places by high velocity flow along steep, rough banks) were unfavourable factors in the calculations. When comparing A, Q, K and P at sites 68, 69 and 70 it appears that the peak at site 68 is too low, and the one at site 69 is probably too high. (Note that in the catchment downstream of site 68 the heaviest rain fell on 24 January, in other words the outflow from the Floriskraal Dam was superimposed on strongly flowing rivers). Due to the above only rating '2' could be accorded to the three sites. At sites 69 and 70 $T_Q > T_p$ which is an unrealistic proposition (Chapter 4.3.1) and is perhaps an indication that the reduction factors given by Figure 7 for the calculation of areal rainfall from point rainfall might be too high for larger catchments in South Africa.

At site 59 in the Touws River, just upstream of its confluence with the Groot River, Q and K seem to be too low when compared with the high P (even slightly higher than in catchments 64 and 65 in Laingsburg). The discrepancy has, however, an obvious cause. In catchment 59 the 3-day rainfall was more evenly spread in time i.e., associated with lower intensity, than in the Laingsburg catchments. Rating '2' was therefore adopted for this site.

Site 71, ± 25 km downstream of the Groot and Touws River confluence, was one of the best slope-area stations where four well defined simple cross-sections were surveyed. The calculated peak of 11 000 m^3/s seems to be too high, and is considerably more than the sum of the upstream tributary peaks : $Q_{59+60+70}$ = 9 000 m^3/s. According to local inhabitants (2) the Groot and Touws Rivers were in full flood concurrently. However, even in the unlikely event of exact coincidence of the two peaks, it is improbable that the intermediate catchment of 847 km^2 could have generated an extra 2 000 m^3/s synchronized peak, because in that zone the heaviest rain fell on 24 January. A plausible explanation for the inflated peak at site 71 is that the coincidence of two extraordinary flood waves at the confluence caused a surge which propagated downstream with an appreciable height (15). If this is so then the flood marks surveyed at site 71 represent the wave crests and not the actual mean maximum flood level. As a point of interest, had the flood profile been 1 m lower, the estimated peak discharge would be decreased to about 9 600 m^3/s. Rating '2' was deemed appropriate for this site.

Gouritz River : site 84

Site 84 is situated just south of the gorge through the Langeberg mountains where the river channel widens and the slope decreases. Three reasonably good cross-sections were surveyed and a good flood profile could be fixed from 40 flood marks. The calculated peak of 11 400 m^3/s is much less than the sum of upstream tributaries i.e., $Q_{71+76+80+81+83}$ = 15 200 m^3/s. The intermediate catchment of approximately 2 000 km^2 had 100 to 125 mm rain which fell mainly during 23-24 January and could easily have generated a flow of 500 to 1 000 m^3/s in the Gouritz River. At site 84 the sediment deposition was important, probably 1 m deep on the average. The surveyed cross-sections were therefore definitely smaller than during the peak flow. Therefore the above aspects suggest that the true peak flow at the site was higher than 11 400 m^3/s. Rating '2' has been adopted. Attention should be drawn to the very evident $T_Q >> T_p$ condition. As mentioned earlier, it is a clear indication that the areal rainfall reduction factors of Figure 7 are not realistic for South African conditions in large catchments, say larger than 5 000 km^2.

It should be added that rating '2' for site 76 in the Gamka River was given because of the satisfactory agreement with the peak outflow from Gamkapoort Dam (site 75) after having taken into account the heavy rainfall in the small intermediate catchment (Figure 8). Ratings at the few remaining sites not discussed in detail were based on the quality of survey and available data.

To summarise the case-study, from Table 9 it is apparent that the survey along the Buffels-Groot-Gouritz River system has supplied flood peaks of satisfactory accuracy.

This supports the great value of the slope-area and bridge contraction methods which were those applied at most sites in the case-study.

Combined evaluation of results and parameters has proved equally useful along the Breë River and elsewhere.

4.3.3 Particular aspects of peak flow evaluation at dams

(1) Reduction of flood peak

Table 10 gives interesting data on the reduction of flood peaks at dams as a function of initial dam content and flood volume (flood volume determination is dealt with in chapter 5.1). These parameters were expressed in dimensionless form as percentage or ratio related to the Full Supply Capacity of dam (FSC). The reduction of flood peak was represented as the ratio of peak outflow to peak inflow (θ/I).

TABLE 10 : REDUCTION OF FLOOD PEAK AT DAMS

SITE NO.	DAM	FSC(1) ($10^6 m^3$)	INITIAL DAM CONTENT % OF FSC	FLOOD VOL. / FSC	REDUCTION (2) θ/I	NOTES
49	Duivenhoks	5,68	99	2,68	∿ 1,00	uncontrolled outlet
42	Elandskloof	11,3	97	0,21	0,75	"
48	Buffelsjagt	5,21	96	7,29	∿ 1,00	"
50	Korinte-Vet	8,48	91	0,47	0,55	"
13	Roode Els Berg	8,20	82	0,70	0,65	"
60	Miertjeskraal	1,57	66	2,68	∿ 1,00	"
10	Settynskloof	15,1	62	0,32	0,07	controlled outlet
56	Prinsrivier	2,86	61	20,6	∿ 1,00	uncontrolled outlet
75	Gamkapoort	54,3	41	3,66	0,84	"
18	Keerom	8,40	36	5,77	0,59	"
28	Klipberg	1,98	28	0,39	0,00	"
67	Floriskraal	68,0	28	2,13	0,80	"
81	Calitzdorp	4,98	25	1,27	0,57	"
12	Lakenvalley	10,3	21	0,29	0,00	"
58	Bellair	11,1	19	0,34	0,00	"
44	Theewaterskloof	483	17	0,06	0,05	controlled outlet

Note : (1) Figures refer to year of completion or last raising of dam.
(2) For I and θ consult column 8 of Appendix I

Apparently both factors exert a marked influence on flood peak reduction which becomes negligible if the initial dam content is high and the flood volume is comparable with or larger than FSC. Note that at Floriskraal Dam the probable peak outflow of approximately 4 000 m^3/s would lower the reduction to 0,70 which would appear a more realistic figure in relation to other data in Table 10. The relatively large reduction at Keerom Dam in spite of the very large flood volume can be explained by the shape of the flood hydrograph (Figure 14d).

(2) Evidence of superelevation of dam levels

As pointed out in Chapter 4.1.4 the possibility exists that during heavy flash floods, surges of significant height can be generated in dams where the peak inflow rate is high relative to dam capacity (Table 5). The comparison of calculated peak outflows at three dams (sites 18, 56 and 67) with peak discharges determined from slope-area surveys downstream of these dams (sites 19, 57 and 68) seem to support above possibility. Unfortunately the Pietersfontein Dam, which also experienced an extreme flood, could not be included in the comparison because of the poor record.

In the following table the two kinds of peaks are compared, the difference in m^3/s is converted to water depth over the spillway which is contrasted with the theoretical surge height.

TABLE 11 : SUPERELEVATION OF DAM LEVEL IN THREE DAMS

SITE NO.	DAM	CATCHMENT A (km^2)	PEAK OUT= FLOW (D) θ (m^3/s)	PEAK FLOW Q (m^3/s)	θ - Q (m^3/s)	Δ h' (m)[1]	Δ h" (m)[2]
18	Keerom	377	557		115	0,48	0,37
19	d/s	573		442			
56	Prinsrivier	757	1 030		190	0,35	0,96
57	d/s	881		840			
67	Floriskraal	4 001	4 620		990	0,47	0,39
68	d/s	4 005		3 630			

Notes : (1) Δ h' = superelevation of dam level at spillway corresponding to θ - Q. Spillway discharge curves were used.

(2) Δ h" = surge height in dam calculated from theoretical wave-celerity and continuity equations by assuming sudden rise of flow rate from zero to peak (15).

Two important qualitative observations can be made. Firstly, at all three dams θ was larger than Q. In other words : The recorded maximum dam levels were too high. Note that the given θ - Q at Keerom and Prinsrivier dams are probably underestimated values, because Q included flows from the not negligible intermediate catchments where heavy rain also fell (Figures 4, 8). Secondly, there is a reasonable agreement between $\Delta h'$ and $\Delta h''$. More accurate surge heights $\Delta h''$ should be derived from hydraulic model tests in which the transformation of surge in the dam and the effect of gradually increasing inflow rate can be investigated.

Notwithstanding the approximate character of above calculations, it is believed that sufficient evidence has been furnished for the existence and importance of surges in dams under certain conditions. It is therefore recommended attention be given to this phenomenon in the determination of maximum design water levels in dams.

4.4 Critical review of technical report TR 105 in the light of calculated flood peaks

Figure 12 was taken from TR 105 (17). The January 1981 floods covered a zone which in the original version of TR 105 belonged to maximum flood peak region 2 and 3 characterized by Francou-Rodier K values of 5,0 and 4,6 respectively. The latter region comprised the upper Touws and Buffels river catchments. The original boundary between regions 2 and 3 in the zone is drawn with dashed line. After the first flood peak estimates for the Buffels River at Laingsburg and the Floriskraal Dam indicated K = ± 5,0 values, it became evident that the entire flood zone should form part of region 2 and the dashed boundary line ought to be replaced by the continuous one.

In report TR 105 it was explained that in the delimitation of the five regions, in addition to the observed maximum flood peaks, the regional topography and storm rainfall pattern were the primary factors that were considered. The upper Touws and Buffels River catchments were originally included in region 3, because in these areas the previously recorded maximum 1-day rainfalls were distinctly lower than elsewhere in the Gouritz River catchment (drainage basin 'J') and were rather similar to the data of the area situated northwest. That area comprised the Doorn and Tanqua River catchments (drainage basin 'E'). The January 1981 storm proved that parts of the Touws and Buffels River catchments can experience as high storm rainfall as the rest of drainage region J and it is incorrect to attach too much importance to past rainfall figures in zones where the density of the raingauge network is sparse. While in the past the highest 1-day rainfall in the Touwsrivier-Laingsburg zone was 60 to 80 mm, on 25 January 1981 several places recorded more than 150 mm. The lesson from the above is that the maximum flood peak region boundaries in Figure 12 should be considered as approximate and, depending on future floods, are prone to adjustments. This is understandable as nobody should expect that nature will strictly follow empirical (or theoretical !) rules set up by humans.

In Figure 13 all maximum flood peaks observed in region 2 and associated with K > 4,0 value were plotted against catchment area on double logarithmic scale. Flood peaks obtained in January 1981 were marked with the site number written in a circle. Sites 26 and 65 were omitted as being non-representative for this purpose. Figure 13 reveals the following :-

(i) Quite a number of the January 1981 peaks surpassed the previously recorded highest K = 4,86 peak in region 2 (observed in March 1974 in the Pauls River, drainage basin of the Great Fish River). At sites 25, 27 and 64 the calculated peak slightly exceeded the regional envelope value of K = 5,0. Before January 1981 the highest recorded K was 4,64 in drainage region H (since 1929) and 4,70 in drainage region J (since 1916). In terms of K, which is the most suitable index for relative flood peak magnitude, these recent peaks were amongst the highest recorded anywhere in South Africa as shown in Table 12.

TABLE 12 : RELATIVELY HIGHEST FLOOD PEAKS RECORDED IN SOUTH AFRICA

RIVER	STATION OR DRAINAGE REGION	A (km^2)	Q (m^3/s)	K	DATE OF PEAK Y M D	MAX FLOOD PEAK REGION NO
Loerie*	L9R01	147	1 750	5,27	81 03 26	1
Van Stadens*	M2	74	1 110	5,18	81 03 26	1
Hluhluwe	W3R01	734	3 060	5,10	63 07 04	1
Willem Nels	H4	32	588	5,03	81 01 25	2
Vink	H4	194	1 450	5,03	81 01 25	2
Nahoon	R3R01	473	2 266	5,03	70 08 28	1
Blyde	N3	130	1 165	5,02	22 01 11	1
Buffels	J1	3 070	5 680	5,02	81 01 25	2
Papenkuils*	M2	39	638	5,01	81 03 26	1

* Note that the documentation of the March 1981 Eastern Cape floods is in preparation.

The fact that three out of the nine peaks listed in Table 12 were recorded in January 1981 might suggest the inclusion of certain areas of drainage region J into the maximum flood peak region 1 where RMF is defined by K = 5,25. Another alternative would be to establish a separate maximum flood peak region which could comprise the southern flank of actual region 2, say drainage basins G to S. Here the envelope K value could be between 5,0 and 5,25. It is felt, however, that important changes in regional boundaries or envelope K values should be based on more data. The most sensible course to adopt would appear to be to wait until the next general updating of the countrywide maximum flood peak catalogue published in report TR 105. There it was mentioned that world wide experience seems to indicate that regional envelope K values based on a fair number of reasonably good data are not expected to increase significantly, say by more than 0,1 to 0,2. (Note that an increase of 0,1 in K means about 16% increase in RMF in a catchment of 100 km^2 and 10% increase in a catchment of 10 000 km^2).

(ii) The second important conclusion from Figure 13 is that the peaks with K > 4,0 recorded during the recent floods lie aligned with the direction of the K = constant lines. This is particularly obvious in the case of the highest peaks which follow the direction of the K = 5,0 line very closely. The realistic nature of the Francou-Rodier approach of representing envelope lines for regional maximum flood peaks has thus been confirmed once again.

5. FLOOD HYDROGRAPHS

These were computed at 41 sites from water level records and were used for the calculation of flood volume, runoff percentage and for the determination of the time to peak and the time of peak. Figures 14a - k show hydrographs at 11 sites which experienced moderate to extraordinary flood peaks. These unfortunately do not include sites along the Buffels River downstream of Floriskraal Dam and along the Touws, Groot and Gouritz Rivers where all gauging weirs were washed away or seriously damaged. At dams both inflow and outflow hydrographs were computed wherever possible.

5.1 Volume and runoff factors

The flood volume was taken as the total volume between the time of the apparent sudden rise of the hydrograph and the time when the descending limb again reached the initial discharge. In determining volumes care was taken to exclude those parts of the hydrograph which were generated by rains which fell outside the adopted standard 3-day period. The calculated volumes appear in columns 13 and 14 of Appendix I expressed in $10^6 m^3$ and mm respectively. The listed values are gross volumes i.e., volumes retained by upstream dams in the catchment were included. Net volumes, wherever applicable, are indicated in column 18.

The runoff percentages were calculated from column 14/column 15 and appear in column 17. An examination of figures listed in column 17 reveals a very great variation, from less than 10% to more than 90%, which seems to be related to the 3-day rainfall P. In Figure 15 the runoff percentage was plotted against P for catchments larger than 200 km^2. The general tendency of the correlation is surprisingly clear and it represents essentially dry antecedent catchment conditions. The figure hints that dry catchments in the region need 30 to 50 mm storm rainfall to produce runoff. In comparison with Figure G2 in reference (18), Figure 15 indicates considerably lower percentages for storm rainfalls less than about 150 mm. This could be expected, as Figure G2 was based on average antecedent catchment wetness conditions. The comparison also reveals that the importance of antecedent conditions diminishes with increasing storm rainfall depth and becomes insignificant for storm rainfalls of 200 mm or more.

Smaller catchments were not considered in plotting Figure 15, partly, because of less representative rainfall figures (Chapter 2.3.1) and also because in small catchments the 3-day standard storm duration is far too long for the purpose. On the whole, runoff percentages calculated for small catchments were higher than for larger ones, because of the generally much steeper surfaces. It should be added that steep surfaces tend to minimize the importance of antecedent wetness in the runoff process. In a few small catchments runoffs higher than 80% were obtained (sites 10, 27 and 78).

Figure 15 also discloses that the runoff percentages in the Buffels River at Laingsburg and Floriskraal Dam (sites 65, 67) are somewhat low. This may be attributed most likely to unavoidable inaccuracies in the levels reported by eyewitnesses during those tragic hours (Figure 14h). Figure 15 suggests that the probable true runoff percentage at sites 65 and 67 could have been between 30% and 40% of the rainfall.

5.2 Time to peak (t_p)

The time to peak used in this study is the duration of the rising limb of the hydrograph. The starting time was chosen to coincide with the sudden increase of flow rate which was easily detected on the recorder graphs. In a few cases the flow increase was gradual. At such stations the starting time could be fixed only roughly.

Information on t_p is useful, because it can give an approximate idea of the duration of the most intense storm that caused the characteristic upper part of the flood hydrograph. Further, the comparison of t_p and the time of concentration t_c should indicate whether or not the volumes of recorded hydrographs (column 13 of Appendix I) are realistic enough to be used as design flood volumes associated with the recorded peaks (column 8 of Appendix I). If, t_p and t_c do not differ too much, say $0,5 < t_p/tc < 2$, then the flood volumes obtained from the survey can be considered as being fairly typical and to occur simultaneously with the recorded peak. Table 13 shows t_p, t_c, t_p/t_c and the catchment areas for all 41 sites. In case of multi-peaked hydrographs only t_p of the biggest flood peak was determined. The time of concentration was computed by the well known and widely used U.S. Bureau of Reclamation formula :

$$t_c = \left(\frac{0,87\ L^2}{1000\ S} \right)^{0,385} \quad \text{(h)} \ \ldots\ldots\ldots\ldots\ldots \quad \text{Eq. (5.1)}$$

where L = length of longest watercourse (km)
S = mean slope of longest watercourse (km/km)

In Table 13 it is seen that t_p (i.e., the approximate duration of the heaviest storm) was, with a few exceptions, from 2 to 12 hours. Notable exceptions are sites 48 and 49 where the long t_p was caused by fairly uniform intensity, long duration rain as shown by the autographic record at Riversdale (Figure 3a). It is also seen that t_p/t_c was more than 2,0 in nearly all catchments smaller than 200 km^2. At such sites the recorded flood volume was thus larger, sometimes very much larger, than the one which could be associated with a design hydrograph. In catchments larger than 4 000 km^2, on the contrary, t_p/t_c was 0,5 or less, indicating that the recorded volume was less than the volume of a probable design hydrograph. For example, the approxi= mate volume of a design hydrograph at Laingsburg with a peak of 5 680 m^3/s could be nearly twice as big (from Table 13 : $^1/0,61$) as the 144x10^{6}m^3 obtained from the survey. The very short t_p of the main peak at sites 37 and 46 in the Breë River was caused by the extraordinary peaks from relatively small tributary catchments in the Robertson-Montagu area (Figure 14c). A realistic design flood volume corresponding to the recorded peak of 1 540 m^3/s at site 46 should be several times bigger than the

TABLE 13 : TIME TO PEAK (t_p) VS TIME OF CONCENTRATION (t_c) IN 41 CATCHMENTS

SITE NO.	RIVER	A (km^2)	t_p (h)	t_c (h)	t_p/t_c
1	Koekedoe	53	1,0	2,5	0,40
2	Rooikloof	11	6,9	0,45	15
4	Breë	657	5,2	6,9	0,75
6	Wit	84	6,2	2,3	2,8
8	Elands	61	6,4	1,0	6,4
9	Molenaars	113	4,0	2,0	2,0
10	Settynskloof	55	6,0	1,3	4,6
11	Hartbees	13	11,2	0,43	26
12	Sanddrifskloof	80	10,0	2,4	4,2
13	Sanddrifskloof	139	6,2*	3,5	1,8
14	Sanddrifskloof	175	8,8*	4,0	2,2
16	Hex	718	12,0	8,4	1,4
18	Nuy	377	3,6	4,6	0,78
22	Breë	4 140	11,3	22,6	0,50
23	Waterkloofspruit	14	3,7	0,86	4,3
26	Willem Nels	24	2,2	1,1	2,0
28	Konings	54	7,8	2,3	3,4
29	Houtbaais	25	9,6	1,6	6,0
30	Keisers	117	11,4	3,0	3,8
33	Keisie	78	7,6	2,7	2,8
37	Breë	6 690	6,0	40	0,15
38	Boesmans	25	12,0	1,4	8,6
41	Du Toits	46	5,6	1,1	5,1
42	Elands	50	6,8	1,4	4,9
43	Waterkloof	15	7,3	0,82	8,9
44	Riviersonderend	497	5,7	6,4	0,89
46	Breë	9 842	10,0	59	0,17
48	Buffelsjagt	601	19,2	10,3	1,9
49	Duivenhoks	148	24	4,8	5,0
50	Korinte	37	8,8	0,87	10
53	Smalblaar	30	4,2	1,3	3,2
54	Bok	8,8	4,3	0,47	9,2
56	Prins	757	8,5	7,8	1,1
60	Brand	251	5,2	4,0	1,3
64	Buffels	3 070	10,0	16,3	0,61
67	Buffels	4 001	10,6	20	0,53
75	Gamka	17 076	15,6	36	0,43
78	Wilgehout	25	2,0	1,1	1,8
81	Nels	170	8,6	2,6	3,3
82	Grobbelaars	151	14,0	3,1	4,6
85	Doring	6 903	12,0	23	0,52

Note : * these values were influenced by dam retention.

recorded net volume of $136 \times 10^6 m^3$. Note that the second lower peak in Figure 14c corresponds to the main peak recorded further upstream at site 22.

It is significant that at sites 18, 26, 56, 64, 67 and 78 where the relatively biggest peaks occurred (high K, return period 50 year or longer), ratio t_p/t_c was between 0,5 and 2,0 which is a reasonable approximation of design flood conditions.

5.3 Time and travelling time of peaks

The time of peak could be established at 58 sites, mostly from the recorded hydro= graphs, but also from information by local inhabitants. These times are listed in column 12 of Appendix I. At 44 sites out of 58 the peak occurred between 12.00 - 24.00 on 25 January. The existence of multiple peaks, the accounts of local people and the relatively great difference between the time of peaks in the smaller catchments are undeniable evidence that the floods were generated in most areas by separate high intensity storms. The flood peaks with the highest K in catchments smaller than 4 000 km^2 were observed around 18.00 to 19.00.

The travelling time of flood peaks could be determined only in the Breë River and the Buffels-Groot-Gouritz Rivers where information was available at more than one site. The times, distances and the propagation velocity of peaks are shown in Table 14a-b.

TABLE 14a : PROPAGATION OF FLOOD PEAKS IN THE BREË RIVER

SITE NO.	DISTANCE (km)		MAIN PEAK*			SECOND PEAK *		
	L	ΔL	TIME	Δt (h)	$\frac{\Delta L}{\Delta t}$(km/h)	TIME	Δ (h)	$\frac{\Delta L}{\Delta t}$(km/h)
4	0		-			25 Jan, 18.00		
		70					22	3,2
22	70					26 Jan, 16.00		
		64					18	3,6
37	134		26 Jan, 02.00			27 Jan, 10.00		
		69		7	9,9		7	9,9
46	203		26 Jan, 09.00			27 Jan, 17.00		

Note : * refers to sites 37, 46. At sites 4, 22 the 'second peak' was the only one.

TABLE 14b : PROPAGATION OF FLOOD PEAKS IN THE BUFFELS-GROOT-GOURITZ RIVERS

SITE NO.	DISTANCE (km)		PEAK		
	L	Δ L	TIME	Δt (h)	Δ L/Δt (km/h)
64	0		25 Jan, 17.45		
		21		1,25	16,8
67	21		25 Jan, 19.00		
		36		2,0	18,0
69	57		25 Jan, 21.00		
		63		5,5	11,5
71	120		26 Jan, 02.00		
		140		∿ 20	∿ 7
Gouritz mouth	260		26 Jan, 22.00*		

Note : * Estimate based on press reports.

The above figures show that the velocity of flood peak propagation was much higher in river reaches experiencing extreme floods. For the sake of comparison it may be mentioned that during the January 1978 Pretoria flood the highest recorded flood peak propagation velocity was 10,7 km/h in the Hartbeesspruit (tributary of Pienaars River) see Table 5 in reference (16).

It should be kept in mind that the calculated propagation velocities are only a rough approximation. The reason is that the upstream flood wave shape is modified by natural storage and the superimposition of tributary flood waves on the main wave as it is propogated downstream.

6. SEDIMENT

6.1 General

Soil loss during floods and deposition of sediments further downstream is probably the most widespread form of flood damage in the drier regions of South Africa.

Landsat images of the Buffels River catchment upstream of Floriskraal dam before and after the flood (10.11.80 and 8.2.81) show a complete scouring out of vegetation of all river channels. From the images it appears possible that the main channels of the Bobbejaans, Wilgehout and Buffels Rivers were widened on average by about 30-40% . (Photo's 20, 21).

Very severe sheet erosion was observed in the upper reaches of the Wilgehout, Prins and Touws Rivers. Near the town of Touws River new erosion gullies up to 20 cm deep were observed.

Generally however, the main erosion seems to have taken place along river banks and in the flood plains which were often scoured out to bedrock, i.e. in the upper Buffels, the Keisies, Groot, Touws and Prins Rivers. Severe scouring of river channels took place downstream of river channel constrictions, dams and at river confluences. Various types of erosion are shown in Photo's 22 - 24.

Heavy deposition of sediments on the other hand took place in areas of reduced flow velocities, i.e. on flood plains, especially cultivated lands, which were often completely covered with sediment. (Photo's 10, 25, 26). Downstream of Laingsburg a sediment deposit approximately 350 m wide and 0,5 m deep was observed over a river-length of 2 000 m, representing a sediment volume of 350 000 m^3 in an area of agricultural lands on the right bank. In Laingsburg itself sediment deposits up to 3 m deep were observed and Alexander and Roberts (20) estimate the total volume of sediment deposited in Laingsburg as 200 000 m^3. (Photo 27). By comparison about 45 times this volume was deposited in Floriskraal Dam downstream of Laingsburg ($9{,}2.10^6 m^3$).

In most river reaches erosion and deposition occurred simultaneously. (Photo's 5, 11, 15).

6.2 Size of transported material

A total of 50 samples from 12 rivers were taken for sieve analysis. Because of the very limited number of samples and the large variation, even at single sites, this information can only provide a general description.

Grading curves were prepared for all samples and examples are shown in Figure 16. Median grain size information is summarized in Table 15.

Only a few samples were obtained from sites of surface erosion in the catchment itself. The grading curve for material from erosion gullies in the upper Touws River catchment is shown in Figure 16a. This eroded material as well as sheet erosion material from the Wilgehout catchment had a median grain size aroud 0,1 mm. Material from river bank erosion, as shown for the Prins River in Figure 16b was generally more non-uniform and coarser. The coarsest material came from flood plain erosion, with the highest median grain size sampled being 2,2 mm for the Gamka River. As far as maxi= mum sizes are concerned, it was observed in the upper Keisies River, that boulders up to 200 mm in diameter were transported by the flood.

Grain sizes of deposited material also varied considerably. In Figure 16c the size difference between middle of river and riverbank deposition is shown for the Prins River. Figure 16d for the Bobbejaans River and Figure 16f for the Kogmanskloof River show that eroded river-bank material is often very similar in grain size to the deposited material at the same cross-section, possibly indicating a displacement of river-bank material over relatively short distances. As can be seen from Table 15, the coarsest material was deposited in the Buffels River. Most samples show a median grain size between 0,5 and 1,0 mm. Grading analysis examples are shown in Figure 16e. Deposits in the other rivers such as the Prins, Dwyka, Groot and Kogmanskloof were generally finer.

TABLE 15 : MEDIAN GRAIN SIZES OF ERODED AND DEPOSITED MATERIAL (mm)

SITE	EROSION					DEPOSITION
	SHEET	GULLY	FLOOD PLAIN	RIVER BANK	RIVER BED	
Bobbejaans River				0,3	2,3	0,15 0,05
Wilgehout River	0,09(2)*					0,7
Buffels River : upstream of Laingsburg						0,9(1 000 mm layer) 0,03 (200 mm layer) 1,0 (2) 0,5 0,11
downstream of Laingsburg						0,6 0,11
Geelbek River						2,0 (2)
Touws River		0,10(3)		0,04(3) 0,15		
Prins River			0,25(2)	0,4(2) 6,0		2,1(2) middle of river 0,13(2) river bank
Groot River				0,15(2)		
Dwyka River			0,8			0,02
Gamka River			2,2			0,06
Pietersfontein River				0,05(2) 0,04(2)		0,8(2)
Kogmanskloof River			0,06			0,15(2) 0,03

Note : * indicates number of samples with same characteristics, if more than one.

6.3 Sediment concentration

Estimates of sediment concentrations during the peak flood period were made from measurements of depth of deposited sediment and maximum water depth in buildings with high openings and no through flow possiblility.

(a) Buffels River in Laingsburg at "Seunskoshuis" on left bank of river in the main flood stream. (Room on the second floor with broken window facing flow direction and door in room completely closed.).

Sediment depth = 0,22 m
Max. water depth = 1,45 m
Estimated sediment concentration = 14,8% (probably too high).

Other estimates in the Buffels River of 19% in the "Dogterskoshuis" and 33% in the "Kerksaal" were rejected because of indications of turbulent flow through the buildings.

(b) Wilgehout River in Laingsburg at Moden in motor cars on left bank of river. (In small toilet room with small high window slightly open in quieter flow area between larger buildings.)

Sediment depth = 0,10 m
Maximum water depth + 1,36 m
Estimated sediment concentration = 7,4%.

6.4 Sedimentation of reservoirs

A number of reservoir basins were resurveyed shortly after the 1981 floods to assess their loss in capacity due to sedimentation and to obtain an estimate of the sediment inflow rate.

The surveys were completed as follows :-

Pietersfontein	April 81
Keerom	April 81
Prins River	March 81
Bellair	April 81
Floriskraal	February 81
Leeugamka	July 81
Gamkapoort	June 81
Kammanassie	October 81
Stompdrift	October 81

In this report only Pietersfontein, Prins River and Floriskraal Dams will be dealt with, because they had no major additional flood between the January 1981 flood and the resurvey. Keerom Dam's results could not be used for the purpose of this report, because major floods had occurred since the previous survey in 1954. All other dams had been surveyed in 1978 or 1979 with no significant flood events up to January 1981.

Besides the above-mentioned three, all other dams had very large floods in March and April 1981, before the resurvey. Their results will be discussed in a follow-on report by D.B. du Plessis (1982).

The results for Pietersfontein, Prins River and Floriskraal Dam are shown in Table 16. As shown in the table the dams have lost 21%, 63% and 23% of capacity in their lifetime.

For calculation of total sediment inflow since the last survey i.e., mainly during the January 1981 floods, the total capacity change including the accumulation above full supply level (FSL) was considered. This amounted to 0,54, 1,26 and $9{,}22 \times 10^6 m^3$ total additional sediment volume respectively for the three reservoirs mentioned above. However this only represents the fraction of sediment retained in the reservoir. For calculating the sediment fraction passing through the dam, the Churchill curve (19) was used. This empirical relationship, based on United States flood and sediment data, was modified slightly to give more realistic results for the very short retention time (i.e. low sedimentation indices). As shown in Figure 17, the curve was adjusted downward to give greater weight to a data set with low sedimentation indices.

In this way sediment retention of 55%, 45% and 70% was obtained for Pietersfontein, Prins River and Floriskraal Dams respectively. The total sediment inflow was then estimated at 0,98, 2,80 and 13,2x10^6m^3 respectively. This sediment inflow, expressed as a percentage of the total flood inflow for the January 1981 flood, gave an average sediment concentration (volume of sediment per volume of water) of 5,2%, 4,7% and 9,1% respectively. The very high average sediment concentration for the Buffels River into Floriskraal Dam may indicate an underestimation of the total flood volume (Chapter 5.1). However, the estimation of total sediment inflow via the Churchill curves may also have a large margin of error. The sediment concentration figures are however of the same order as those discussed in Chapter 6.3 based on sediment surveys in buildings in Laingsburg.

TABLE 16 : SEDIMENT CONCENTRATIONS IN FLOOD INFLOW INTO DAMS

DAM	PIETERSFONTEIN		PRINS RIVER		FLORISKRAAL	
River	Keisie		Prins		Buffels	
Station No.	H3R02		J1R01		J1R03	
Site No.	31		56		67	
Catchment area (km^2)	166		757		4 001	
Original capacity (10^6m^3) and survey date	2,69 (April 1969)		2,86 (Sep. 1962)		68,0 (Nov. 1956	
Capacity before 1981 floods and survey date	2,50 (Feb. 1979)		2,39 (Oct. 1979)		58,47 (1977)	
Capacity after 1981 floods survey date	2,06 (April 1981)		1,26 (March 1981)		51,95 (Feb. 1981)	
Sediment volume in dam (10^6m^3) (total and up to FSL)	0,69	0,57	1,97	1,81	18,19	15,49
% of original capacity	21%		63%		23%	
Sediment volume since previous survey (10^6m^3)(total up to FSL)	0,54	0,44	1,26	1,13	9,22	6,52
Peak inflow (m^3/s) and date	576(1) (25.1.81)		1030 (25.1.81)		5740(25.1.81)	
Total inflow (10^6m^3)	19(1)		59		145	
Fraction of total sediment inflow retained in reservoir based on Churchill curves (%)	55(2)		45(2)		70(2)	
Estimated total sediment inflow during flood (10^6m^3)	0,98		2,80		13,2	
Average sediment concentration as percentage of the inflow (%)	5,2		4,7		9,1	

Notes :- (1) Only peak outflow available for Pietersfontein; outflow volume estimated form triangular hydrograph with peak of 576 m^3/s and base of 18 hours. For inflow volume the remaining reservoir capacity before the flood was added.

(2) The Churchill curve was adjusted downward (higher retention in reservoir) by putting more emphasis on a lower series of data points.

7. SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Some significant figures :

1. The storm was caused by a black south-easter synoptic condition. The 3-day storm rainfall was the highest in living memory over large areas of the South Western Cape. Over an area of 4 000 km^2 it exceeded the 3-day 200 year rainfall.

   In the Buffels River catchment upstream of Laingsburg it was comparable with or higher than the mean annual rainfall. The maximum observed 3-day point rainfall was 375 mm near Robertson and 288 mm north of Laingsburg.

2. The flood survey covered more than 50 000 km^2. Field data were measured at 86 slope-area, bridge, gauging weir or dam sites in 58 rivers.

3. The storm resulted in some of the relatively highest flood peaks ever recorded in South Africa at :-

   Site 27 : Willem Nels River near Robertson, A = 32 km^2, Q = 588 m^3/s.
   Site 25 : Vink River near Robertson, A = 194 km^2, Q = 1 450 m^3/s.
   Site 64 : Buffels River at Laingsburg, A = 3 070 km^2, Q = 5 680 m^3/s.

   These peaks were roughly equivalent to RMF and associated with a Francou-Rodier K of slightly higher than 5,00.

   At 24 sites the return period of the flood peak was estimated to be 50 years or longer.

4. Runoff percentages of up to 90% of the rainfall were calculated.

5. In the Buffels River flood peak propagation velocities of up to 18 km/h were observed.

6. Sediment concentration in the Buffels River in the Floriskraal Dam reached 9%.

Methods and accuracy of flood peak calculation

7. Slope-area and bridge surveys : About 40% of all flood peaks and more than 70% of the extraordinary peaks were obtained from the above methods. During the calculations practical rules were developed for the estimation of roughness, the computation of the Chêzy resistance factor and the reduction of gross cross-sectional area in vegetated parts (4.1.1). This flood survey proved that in

South Africa the slope-area and bridge-contraction methods are indispensable (and are likely to remain so) in the calculation of extraordinary flood peaks (4.3.1).

8. Gauging weirs : At 50% of these sites the recorded maximum flood level was higher than the limit of available stage-discharge tables. In such cases an approximate extrapolation of the latter became unavoidable (4.1.3).

9. Dams : Evidence was found of superelevation of dam levels caused by surges during extraordinary flood inflows. In such cases both the peak inflow and outflow discharges calculated from the spillway discharge equation and reservoir routing procedures were overestimated (4.1.4, 4.3.3).

10. Accuracy : In 86% of the cases the estimated error of the flood peaks was less than 30%. In the remaining cases the accuracy could not be established. The accuracy of flood peaks calculated from slope-area and bridge surveys compared fabourably with that at gauging weirs and dams (4.3.1).

Analyses

11. Statistical analyses of annual maximum flood peaks at individual stations cannot be used to assign reliable return periods for extraordinary peaks (outliers). It is preferable to express outliers in terms of RMF and relate the ratio to broadly defined return period categories (4.2).

12. The realistic nature of the Francou-Rodier method of characterizing regional maximum flood peaks was confirmed (4.4).

13. Flood peak reduction in uncontrolled dams was negligible if the dam content at the beginning of flood was high and the flood volume was larger than the full supply capacity of the dam (4.3.3).

14. Approximate theoretical surge heights of 0,3 to 1,0 m were calculated in dams affected by extraordinary floods (4.3.3).

15. A marked relationship was found between total storm rainfall and runoff percentage. It revealed that in the area of the flood survey 30 to 50 mm storm rain is needed to cause perceptible runoff after dry antecedent conditions (5.1).

16 In catchments where extraordinary flood peaks had occurred the duration of the rising limb of the flood hydrograph was half to twice the time of concentration which is a reasonable approximation of design flood hydrograph conditions (5.2).

7.2 Recommendations

1. Future flood surveys should start as soon as possible after the rivers have dropped to their normal levels and roads are negotiable.

2. It is desirable that a countrywide network of good slope-area stations at carefully selected sites be established where the recording of extraordinary peaks would be the most rewarding. Such sites should be surveyed prior to floods in order to obtain representative pre-flood cross-sections i.e., to improve the accuracy of flood peak calculations. Good bridge sites should be included in the network.

3. There is an urgent need to extend the calibration limit well into the high flood peak range at many gauging weirs.

4. Surge height should be considered in the design of dam spillways. Dam level recorders should be installed at several points in the dam in order to gather information during heavy floods. Hydraulic model tests are needed for the improvement of surge height estimation.

5. Departmental autographic rainfall recorder network is needed for a more reliable documentation of storm intensity, duration and runoff percentage.

8. STAFF

The present documentation was made possible through the combined effort of hydrologists engineers and technicians of the Division. The following persons took active part in the work :

Hydrologists : J. Benadie*, H. Keuris*, D. Lynch.
Engineers : B. du Plessis*, P. Ellis, D. Marais, W. van der Westhuizen, A. Venter, A. Wensley.
Technicians (from the Sandhills Regional Office) : G. Joubert* (Principal Technician), J. Knoetzen* (Principal Technician), A. van Rooyen.

(* indicates field survey team leaders who were responsible for the quality of field work).

Assistent Technicians (from the Sandhills Regional Office participated in the field survey) : M. Acker, H. Batt, F. Binneman, J. Germishuys, H. Jooste, H. Lourens, G. Malherbe, S. Naude, J. van Bosch, A. van Rensburg, A. van Zyl and M. Zondach.

The preparation and organization of the field survey rested mainly with H. Wolfaardt (Control Technician) and P. Vorster (Principal Technician, Sandhills).

The Chapter on sediments was written by E. Braune (Director of Hydrological Research Institute).

REFERENCES

(1) MARAIS, G.J. : Ons het op die dakke gesit. 1981. *(Eyewitness reports on the Laingsburg flood disaster).*

(2) KüHNE, W.O. : Vloedramp. *Tafelberg Uitgewer. 1981.*

(3) ALEXANDER, W.J.R. and ROBERTS, C.P.R. : Report on Buffels River flood : 25 January, 1981. *Department of Environment Affairs. 14 February, 1981.*

(4) ESTIé, K.E. : The Laingsburg flood disaster of 25 January, 1981. *Weather Bureau News Letter. February, 1981.*

(5) WEATHER BUREAU : *News Letter. March, 1961 .*

(6) ADAMSON, P.T. : Analysis of extreme n-day point rainfall depths for South Africa and South West Africa. *Directorate of Water Affairs, Division of Hydrology. Technical Report No. TR 102. 1980.*

(7) WORLD METEOROLOGICAL ORGANISATION : Measurement of peak discharge by indirect methods. *Technical Note No. 90. Geneva. 1968.*

(8) DIVISION OF HYDROLOGY : Guide for collection of flood peak data. May, 1979.

(9) VORSTER, VAN DER WESTHUIZEN EN VENNOTE : Herstel en verhoging van damwal in Dwykarivier - Merweville. *Verslag KS 229-81, Oktober 1981.*

(10) MARTINEC, J. : Extrapolation of stage-discharge relations for flood computations. *Proceedings of International Symposium on River Mechanics, Vol. 2. IAHR, Bangkok. 1973.*

(11) VEN TE CHOW : Open-channel hydraulics. *McGraw-Hill 1959.*

(12) U.S. BUREAU OF PUBLIC ROADS : Hydraulics of bridge waterways. *Hydraulic Design Series No. 1. Washington, D.C. 1970.*

(13) VENTER, A.P. & BENADIE, J.M. : Extrapolatie van aflooptabelle vir riviervloei= meetstasies. *Interim nota. Oktober 1981.*

(14) ROBERTS, C.P.R. : The behaviour of a large dam during an extreme flood. *(Paper in preparation for later publication in The Civil Engineer in South Africa).*

(15) PERSONAL COMMUNICATION WITH DR J.M. JORDAAN (Chief Engineer : Hydraulic Section, Design Division, Directorate of Water Affairs).

(16) KOVáCS, Z.P. : Documentation of the January 1978 floods in Pretoria and in the Crocodile River catchment. *Department of Water Affairs, Division of Hydrology. Technical Report No. TR 88. 1978.*

(17) KOVáCS, Z.P. : Maximum flood peak discharges in South Africa : An empirical approach. *Department of Water Affairs, Forestry and Environmental Conservation, Division of Hydrology. Technical Report No. TR 105. 1980.*

(18) HYDROLOGICAL RESEARCH UNIT : Design flood determination in South Africa. *University of the Witwatersrand, Department of Civil Engineering. Report No. 1/72. 1972.*

(19) U.S. BUREAU OF RECLAMATION : Design of small dams. *Washington, D.C. 1974*

(20) ROBERTS, C.P.R. & ALEXANDER, W.J.R. : Lessons learnt from the 1981 Laingsburg flood. *The Civil Engineer in South Africa. January 1982.*

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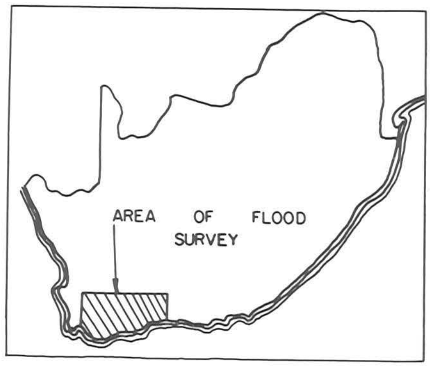


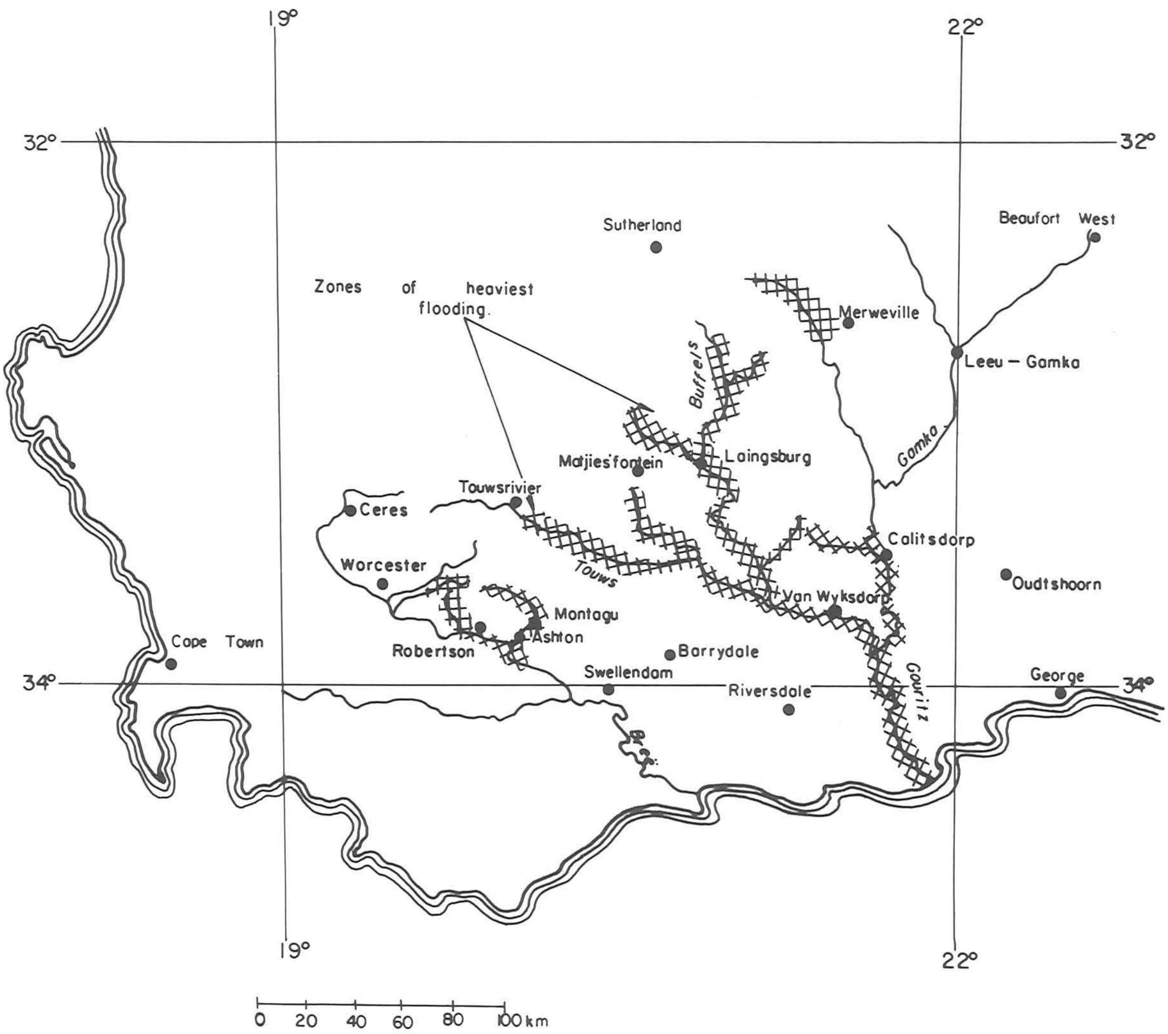


FIG. 1. AREA OF FLOOD SURVEY

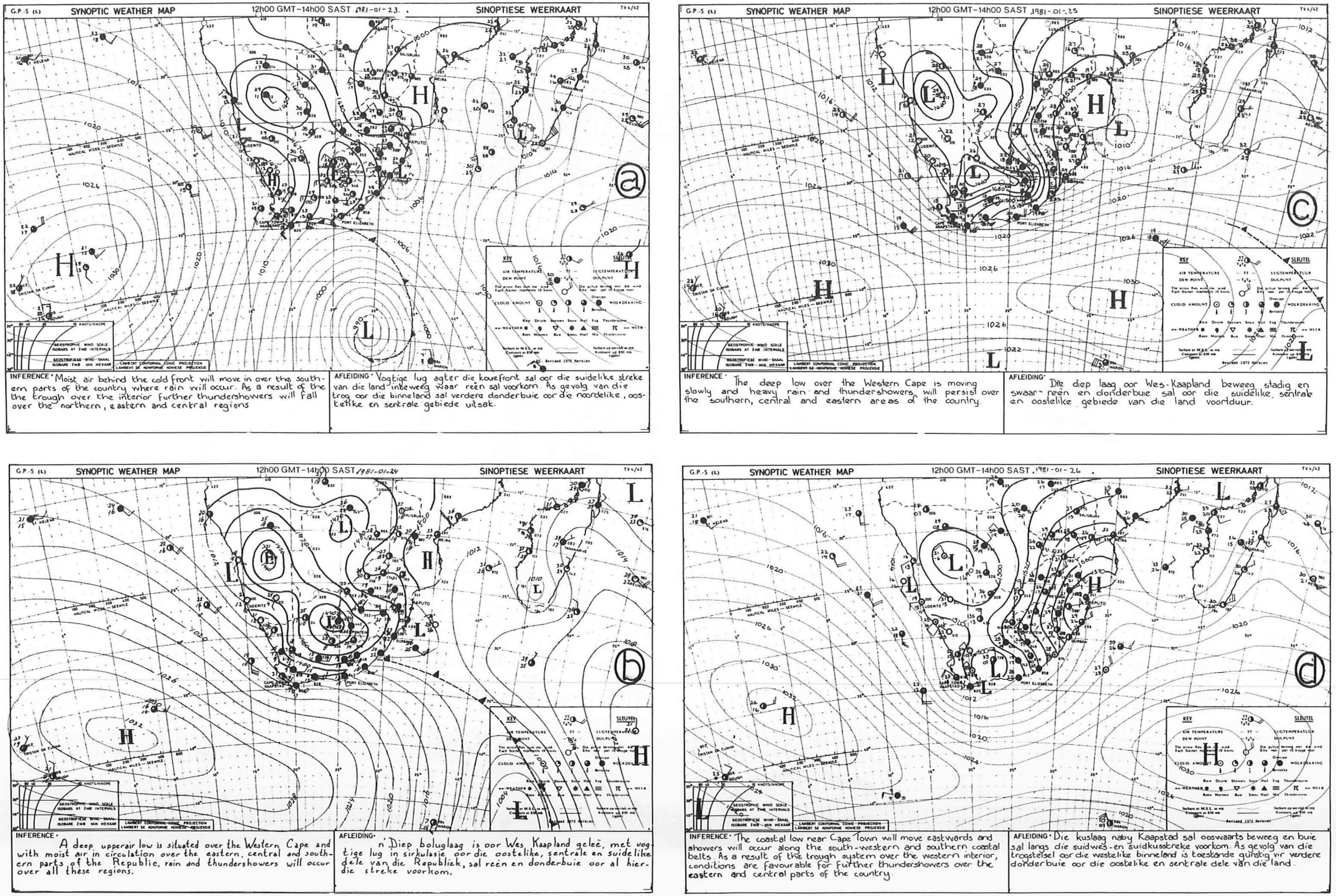


FIG. 2
SYNOPTIC WEATHER CHARTS AT 12 GMT ON 23 – 26 JAN 1981

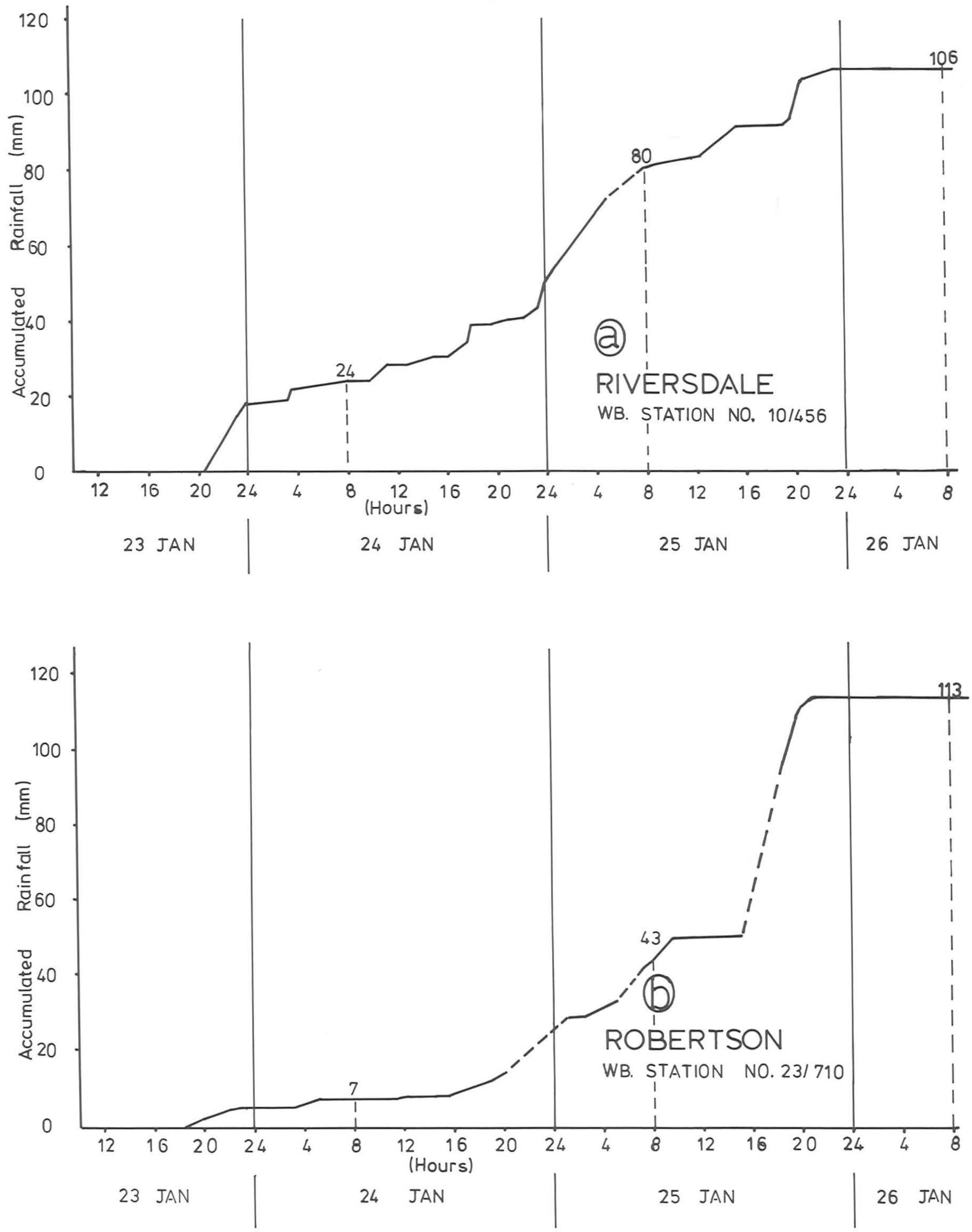


FIG. 3

ACCUMULATED STORM RAINFALL

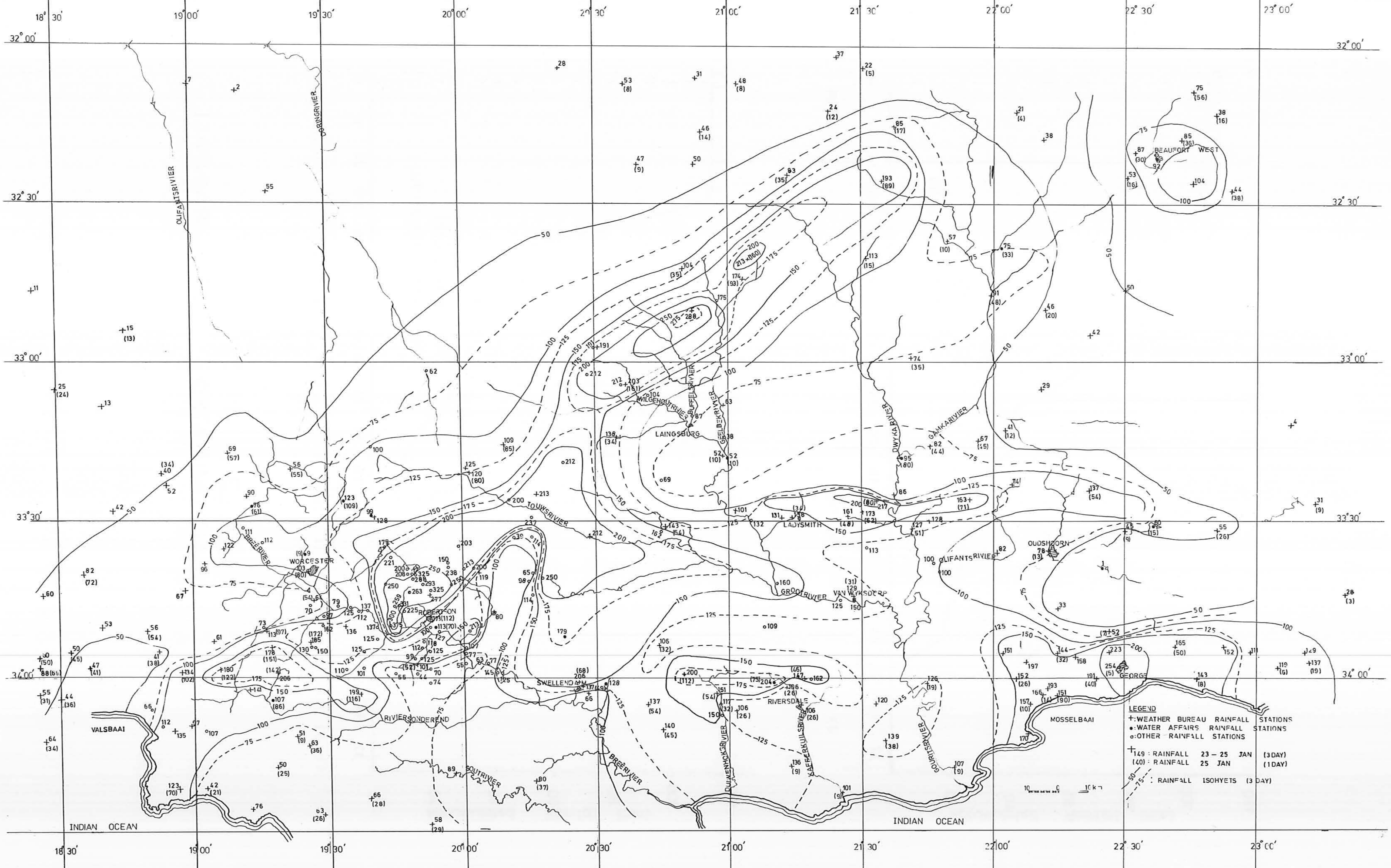


FIG. 4 3 DAY RAINFALL ON 23 — 25 JANUARY 1981 (23 JAN 8AM – 26 JAN 8AM)

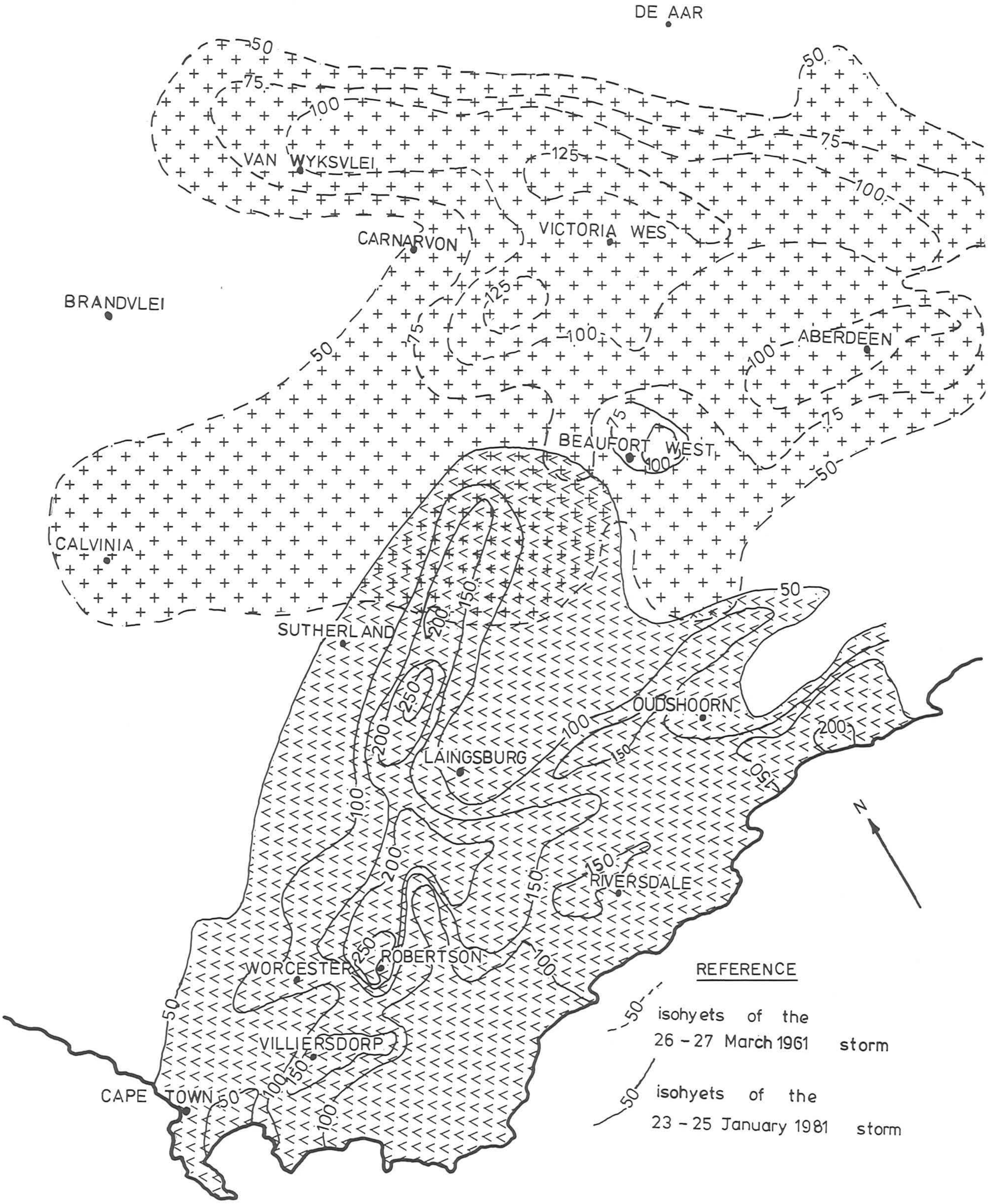


FIG. 5. RAINFALL — AREA COMPARISON WITH THE MARCH 1961 KAROO STORM

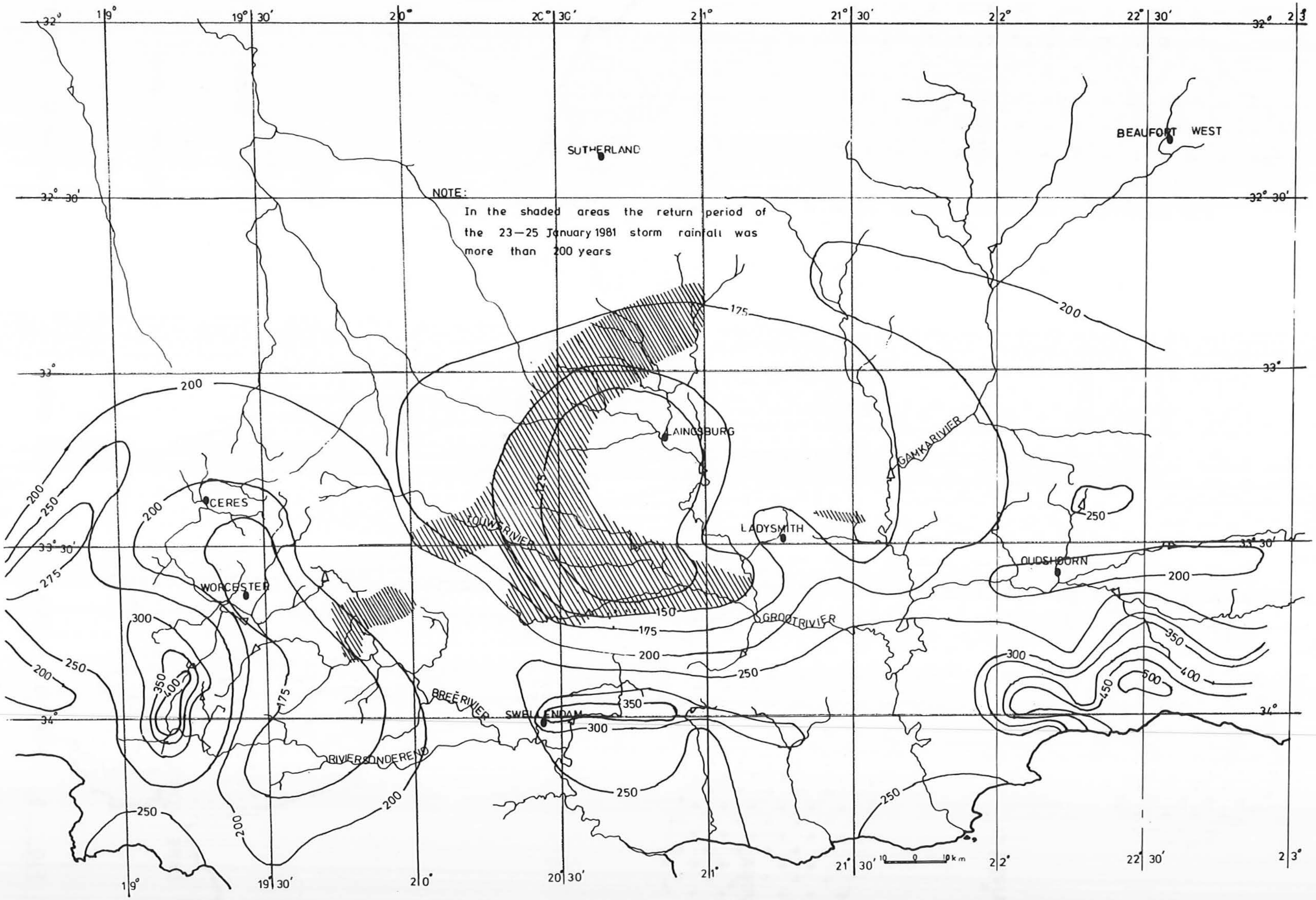


FIG. 6. 3 DAY 200 YEAR POINT RAINFALL (based on Technical Report TR 102)

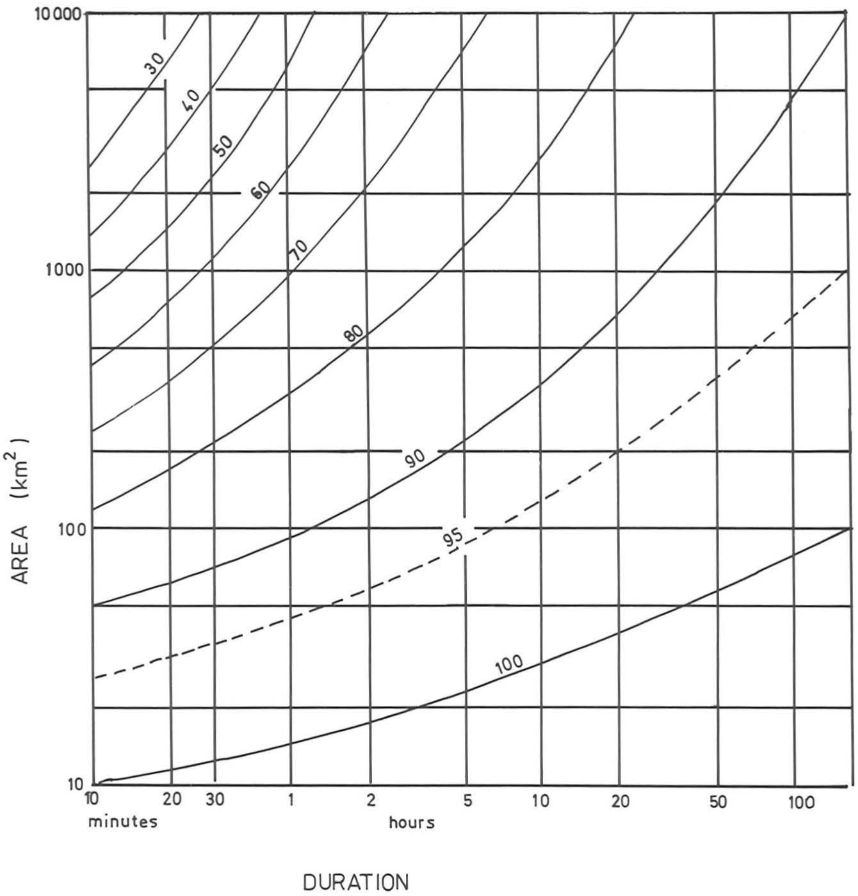


FIG. 7 AREAL REDUCTION FACTORS DERIVED FROM FIXED LOCATION STORM DATA SHOWING AVERAGE PRECIPITATION DEPTHS (from TR 103, based on U.K. Flood Studies Report, 1975)

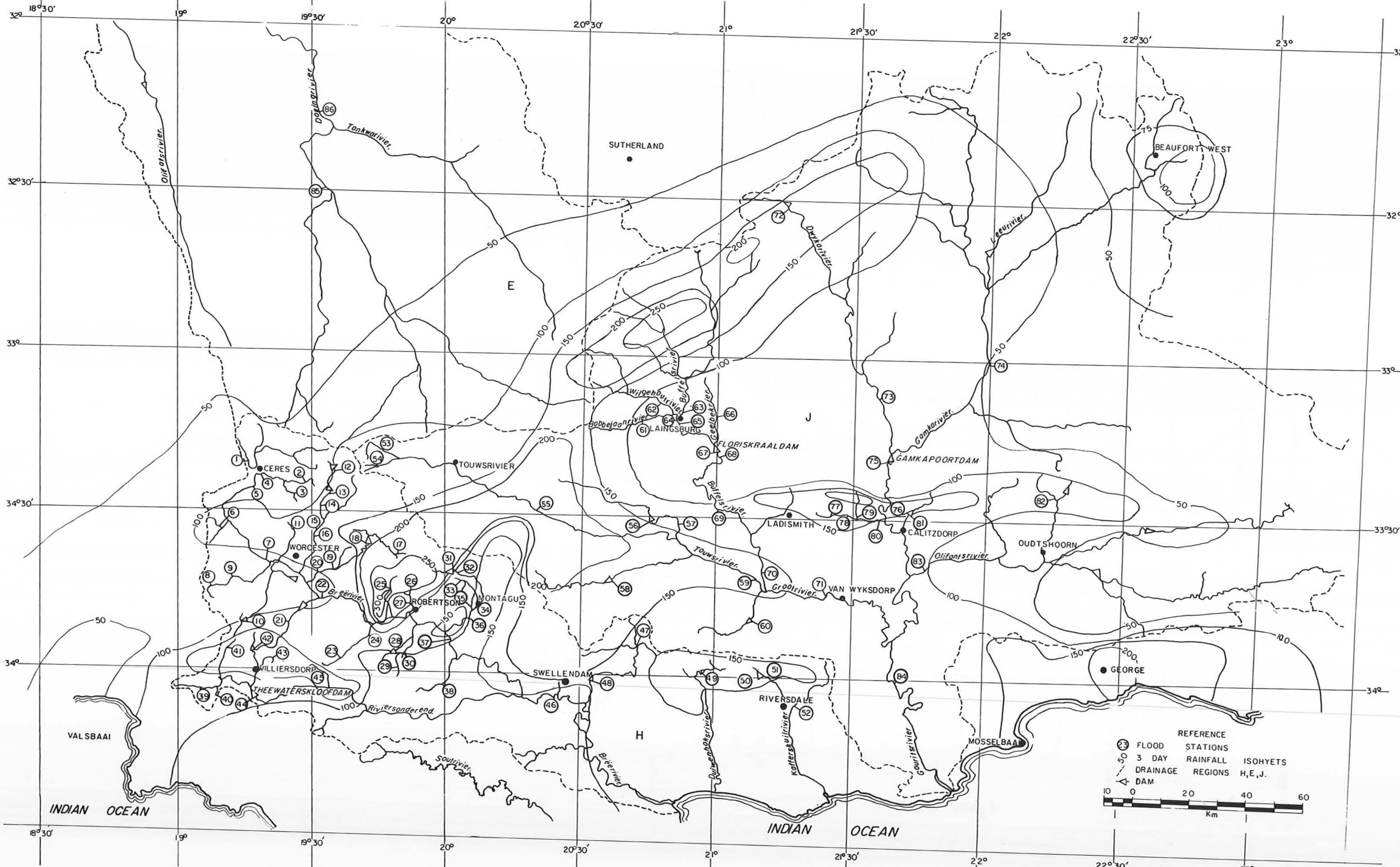


FIG. 8 FLOOD PEAK MEASUREMENT SITES

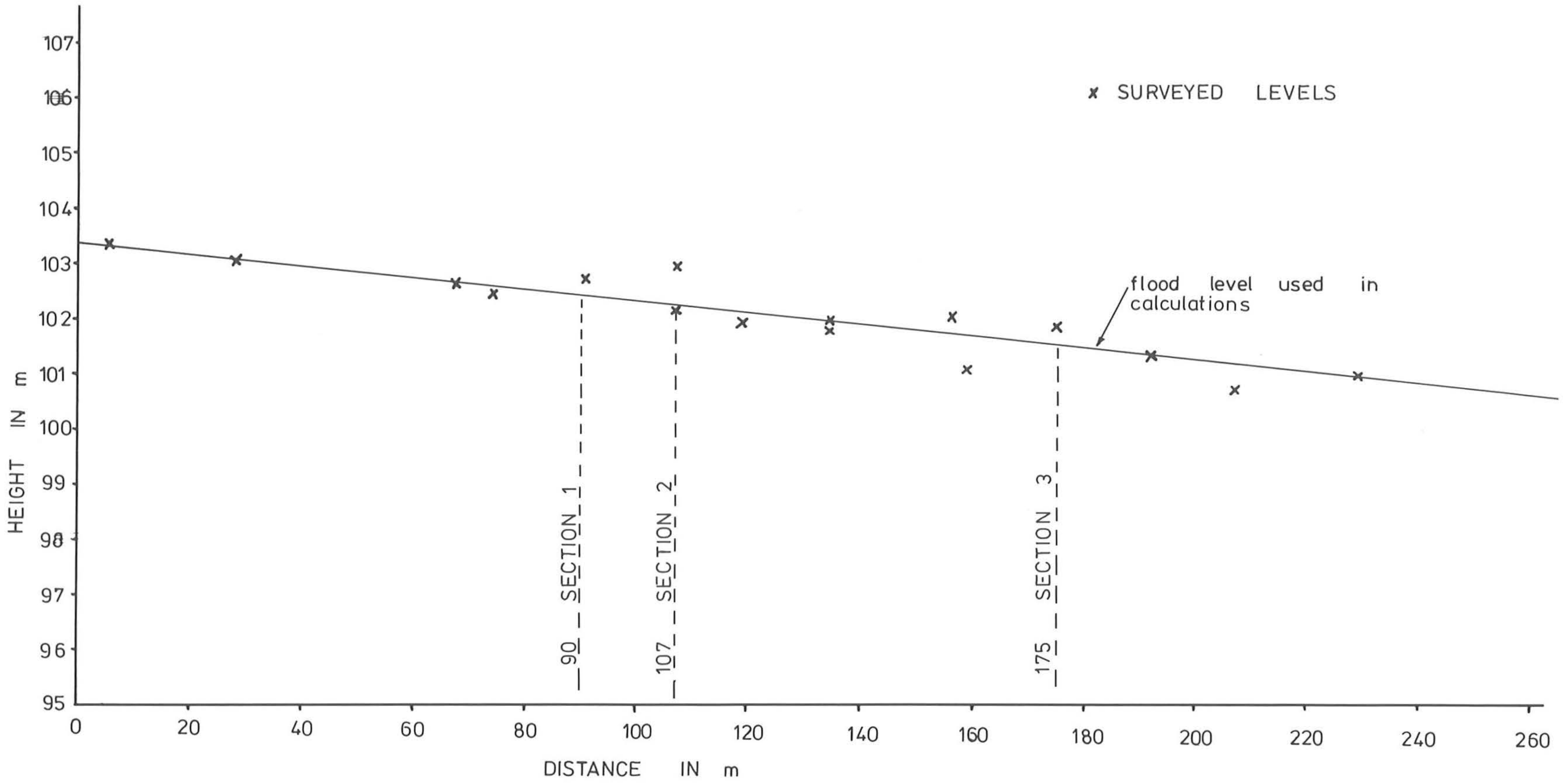


FIG. 9 a. FLOOD PROFILE IN THE HUIS RIVER AT SITE 77

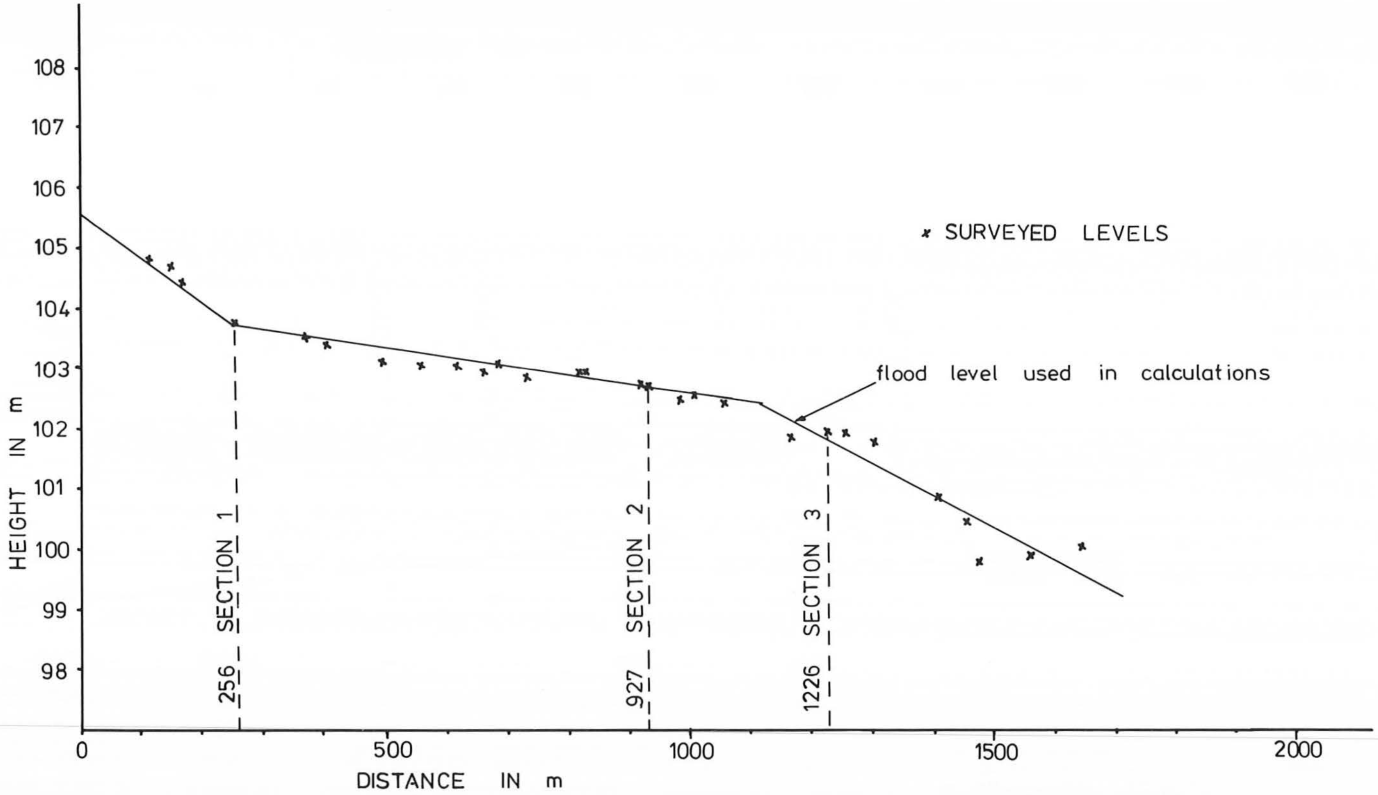


FIG. 9 b. FLOOD PROFILE IN THE TOUWS RIVER AT SITE 59

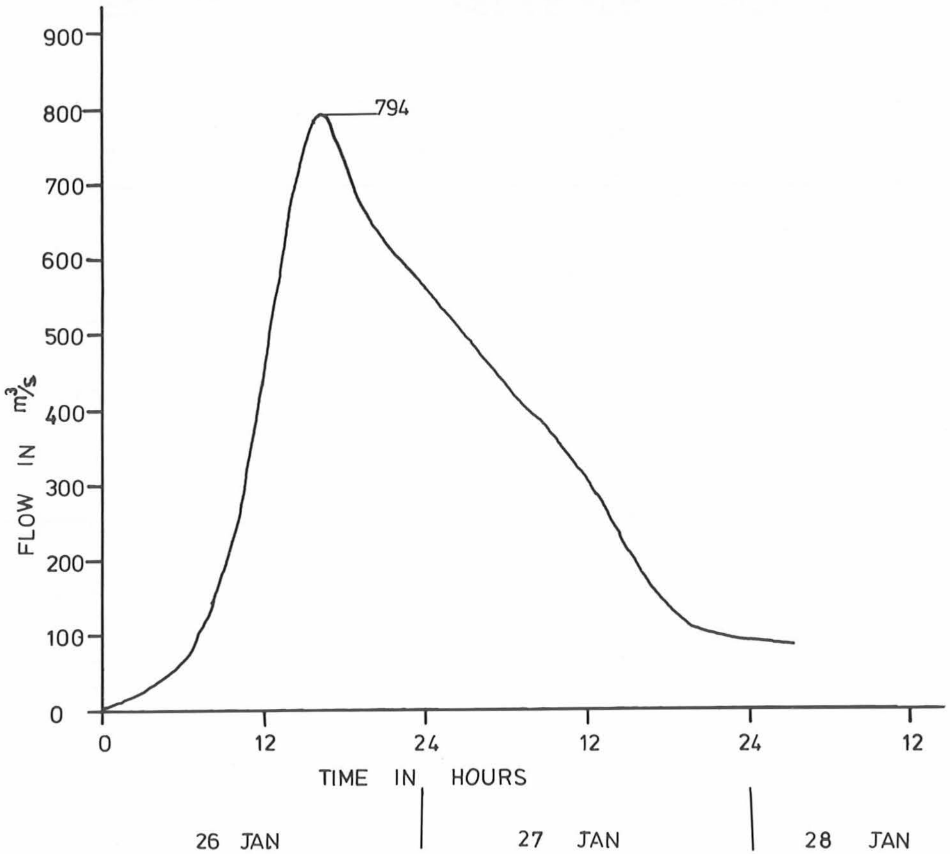


FIG. 14 b. BREËRIVER SITE 22

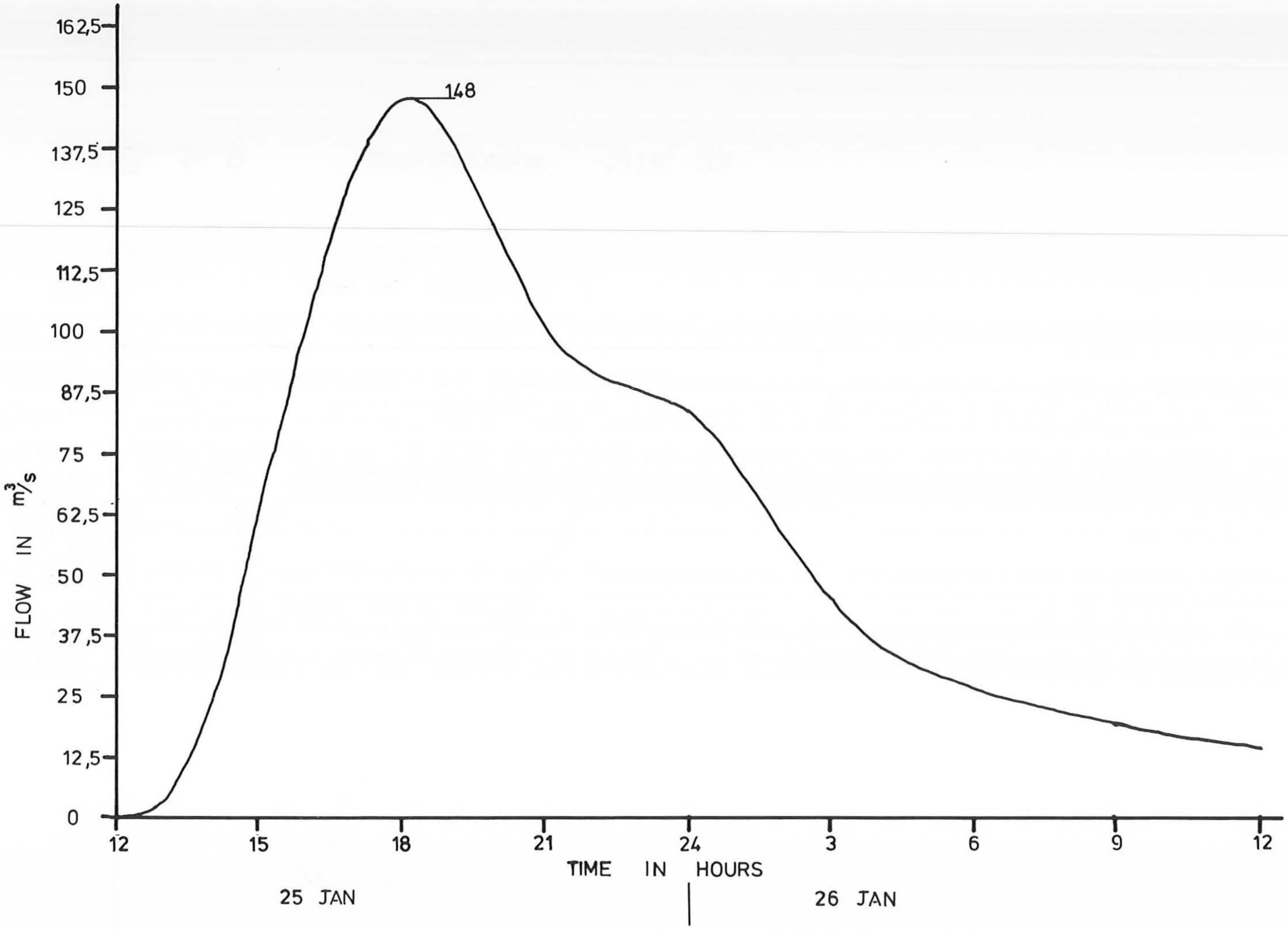


FIG. 14 a BREËRIVER SITE 4

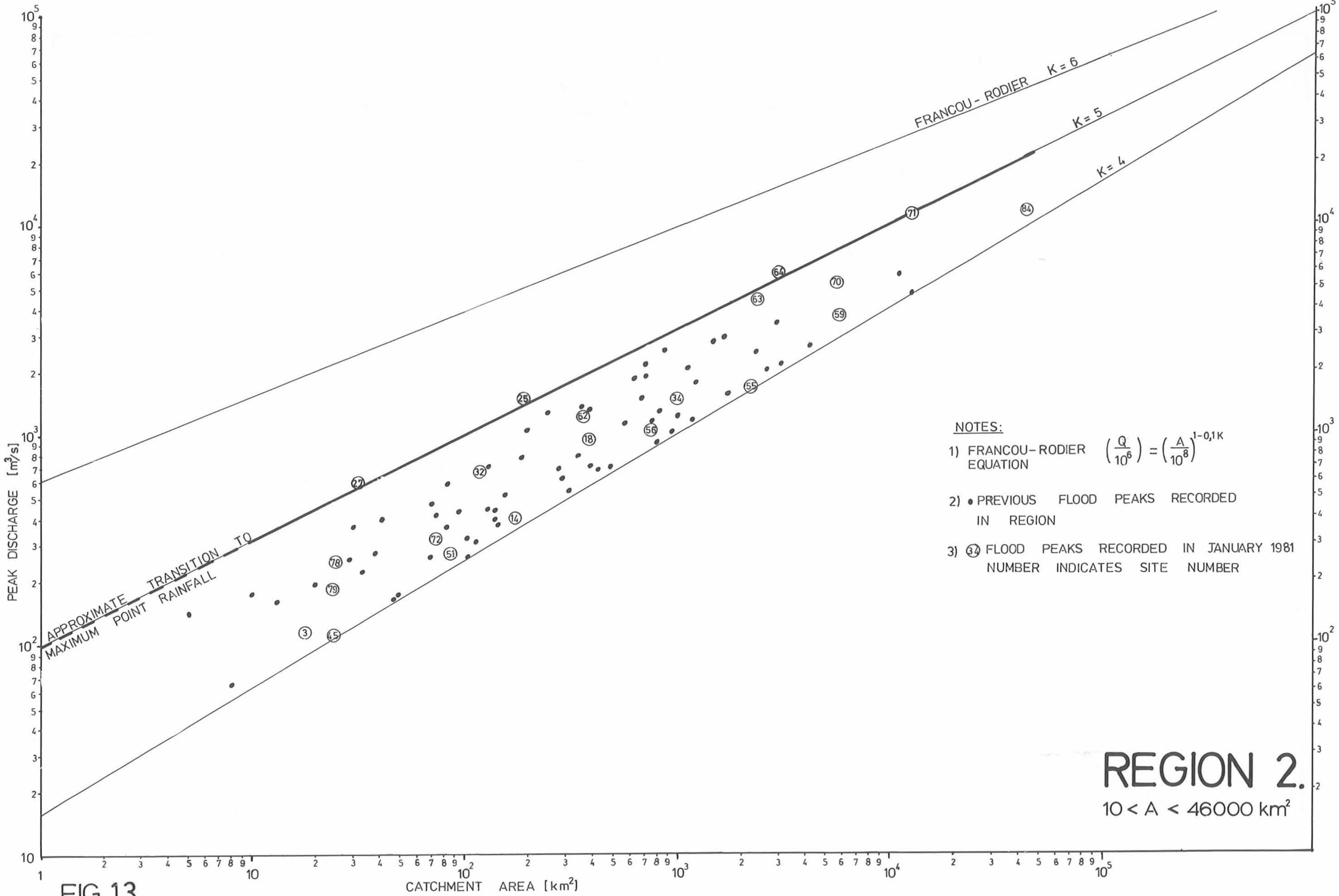


FIG. 13

MAXIMUM PEAK DISCHARGES WITH FRANCOU - RODIER K ≥ 4

MAX. FLOOD PEAK REGION	1	2	3	4	5
FRANCOU – RODIER ENVELOPE 'K'	5,25	5,0	4,6	3,6	2,5

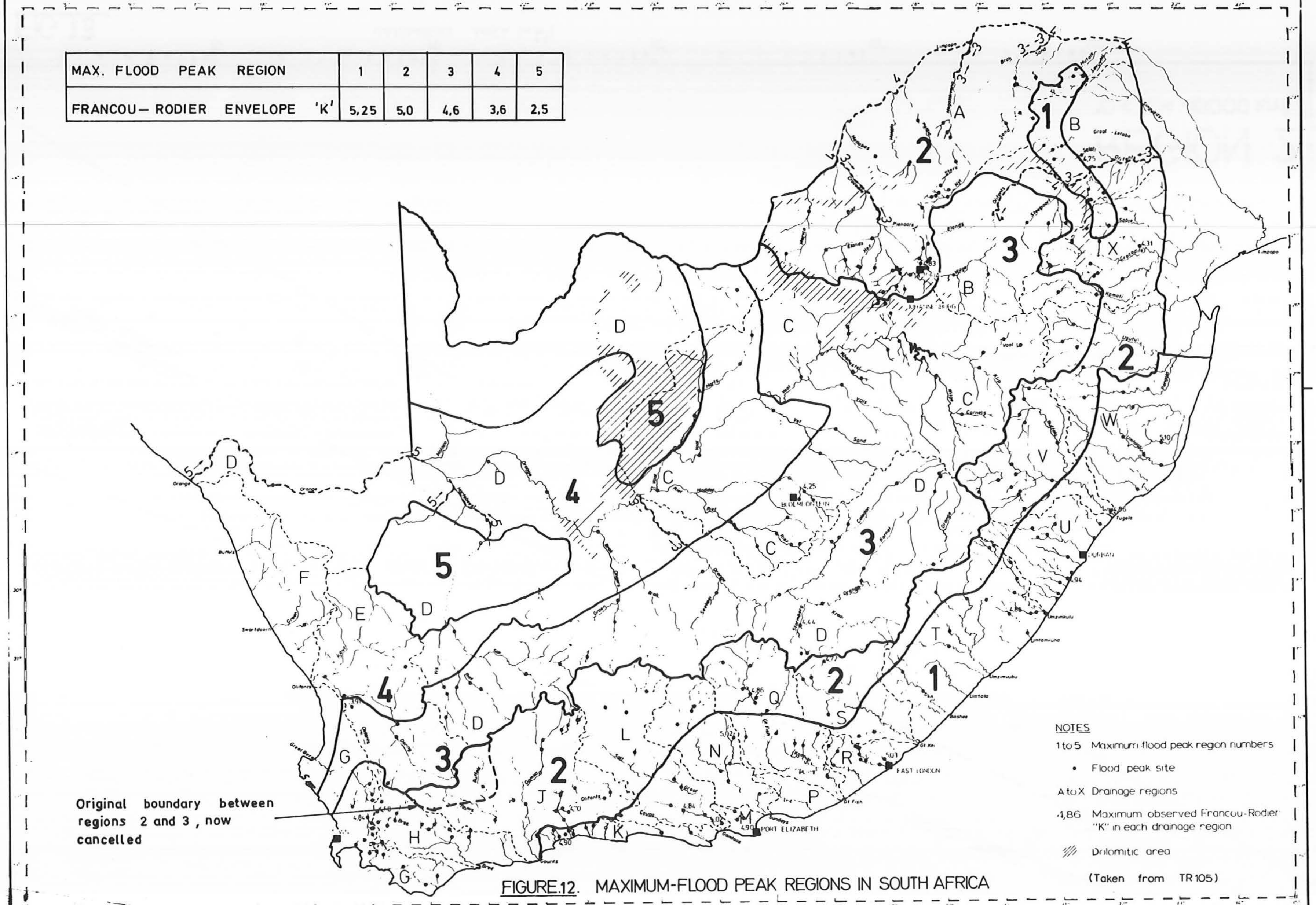


FIGURE.12. MAXIMUM-FLOOD PEAK REGIONS IN SOUTH AFRICA

(Taken from TR105)

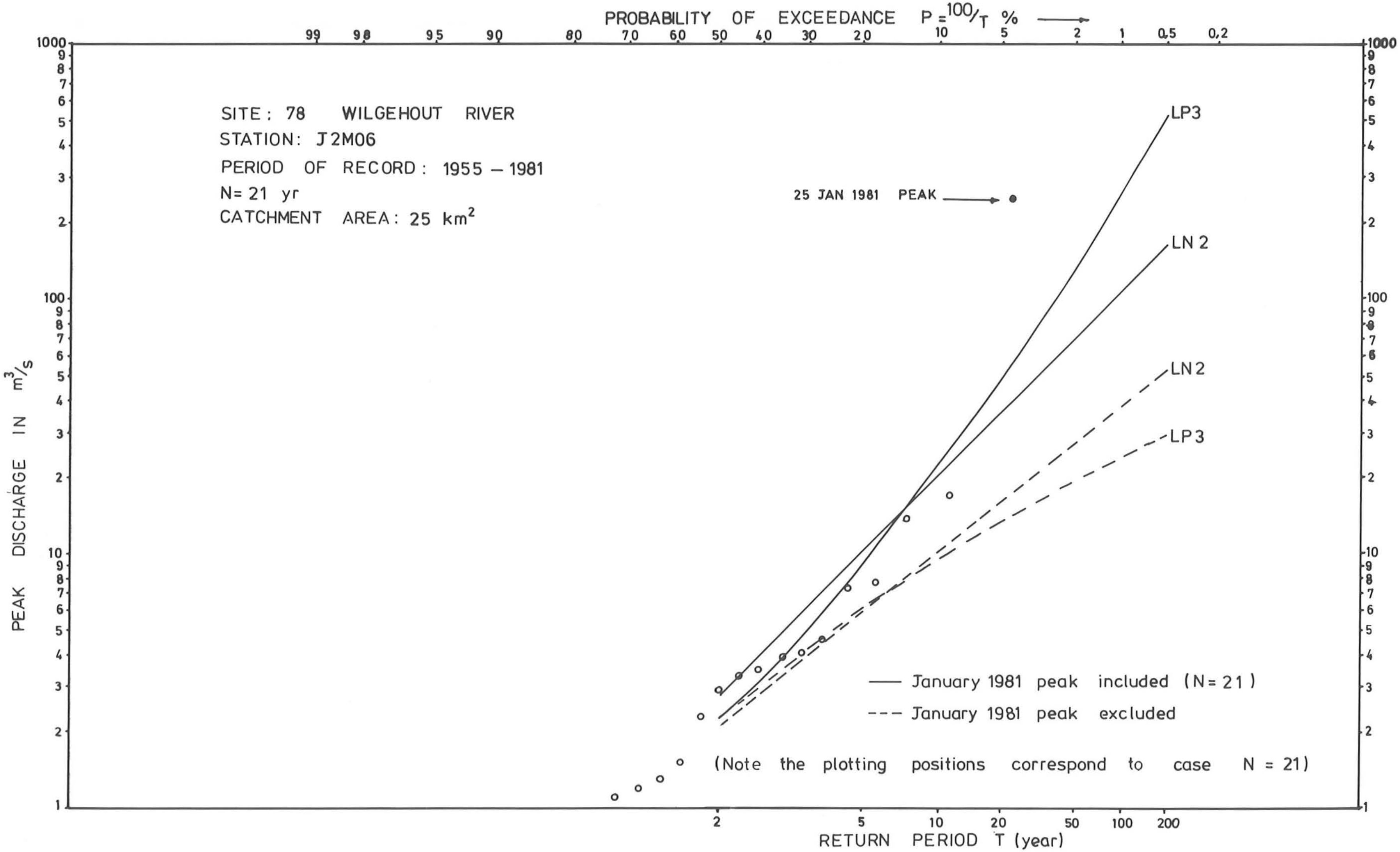


FIG. 11 d. FLOOD PEAK FREQUENCY DISTRIBUTIONS AT SITE 78

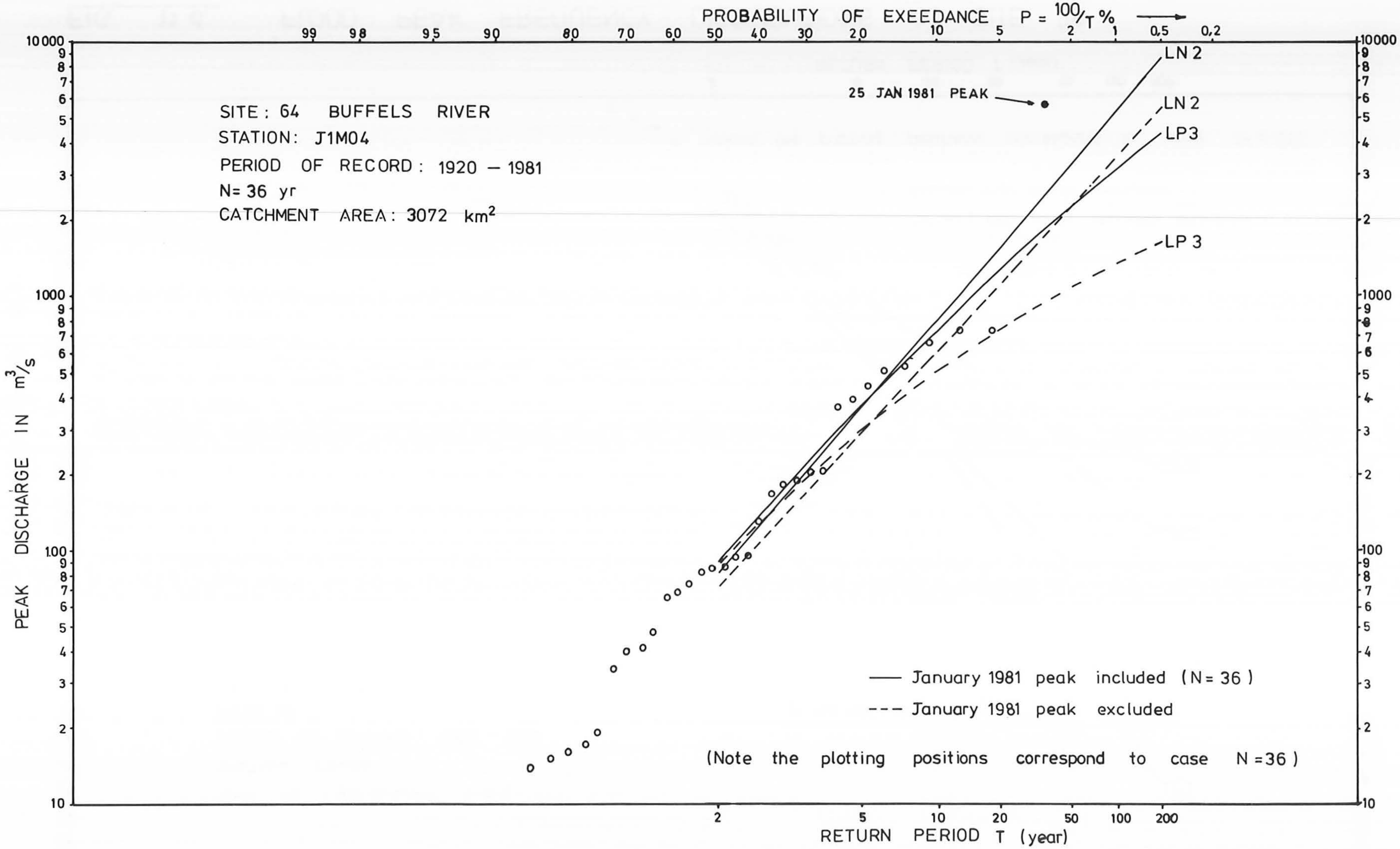


FIG. 11 c. FLOOD PEAK FREQUENCY DISTRIBUTIONS AT SITE 64

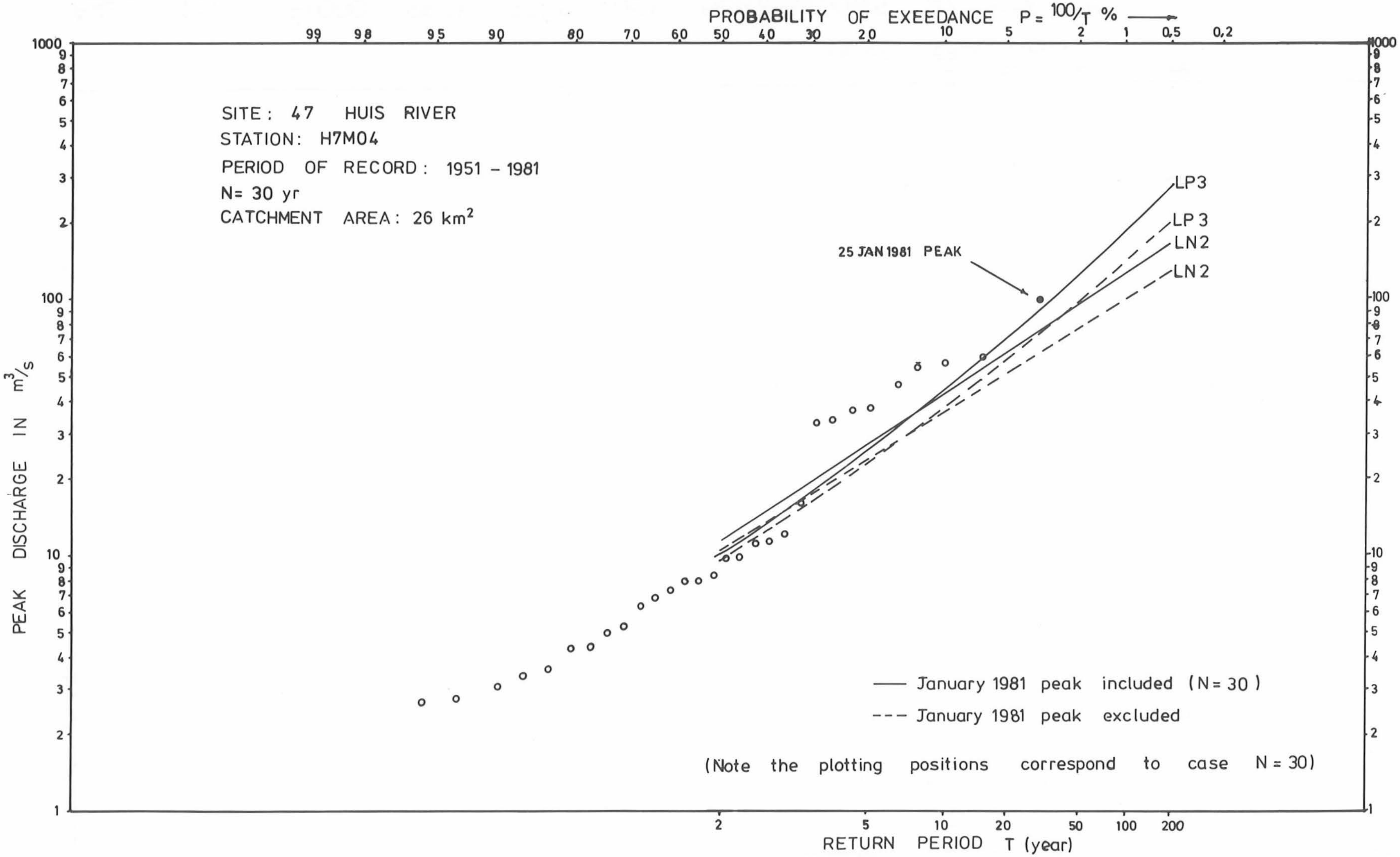


FIG. 11 b. FLOOD PEAK FREQUENCY DISTRIBUTIONS AT SITE 47

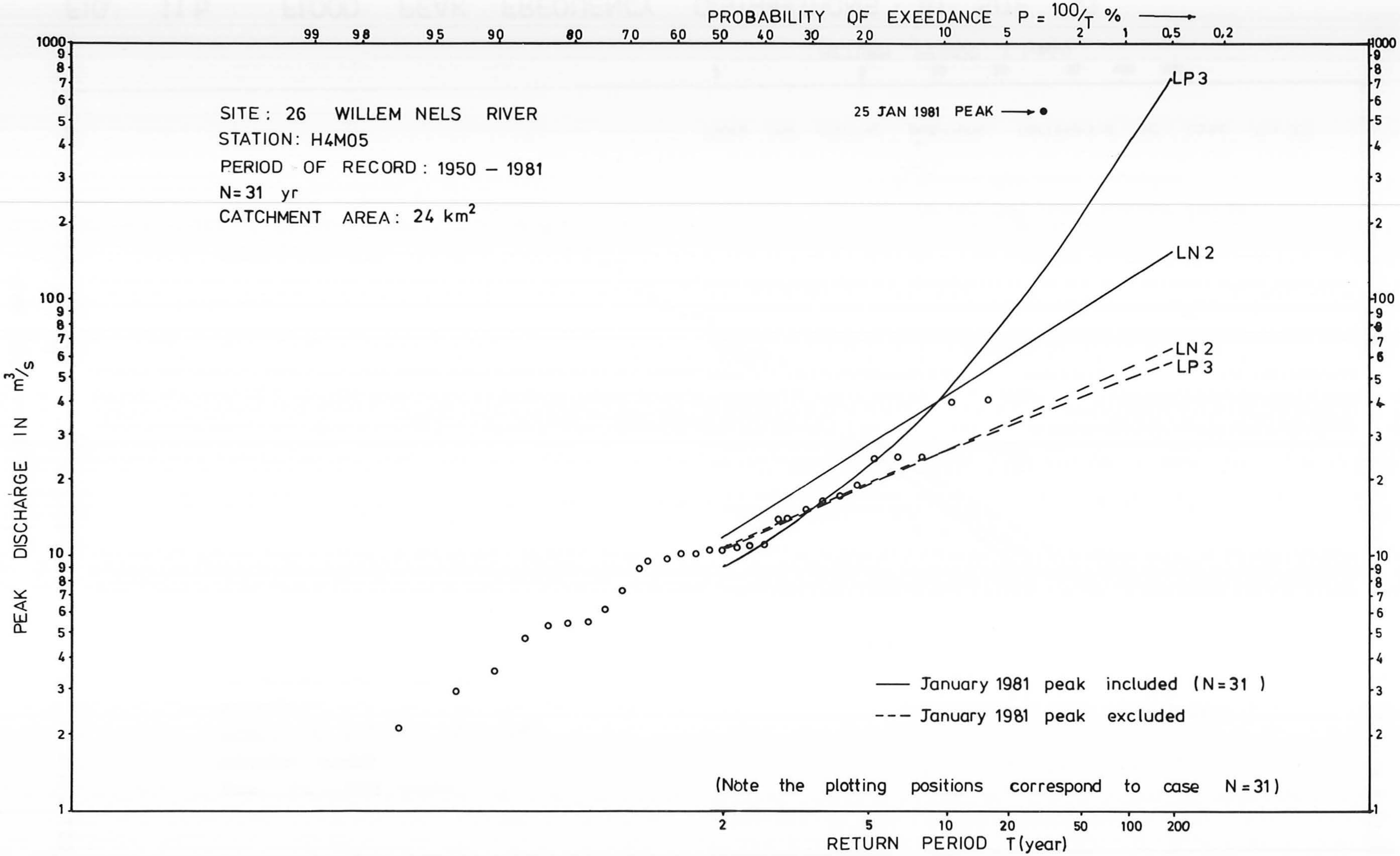
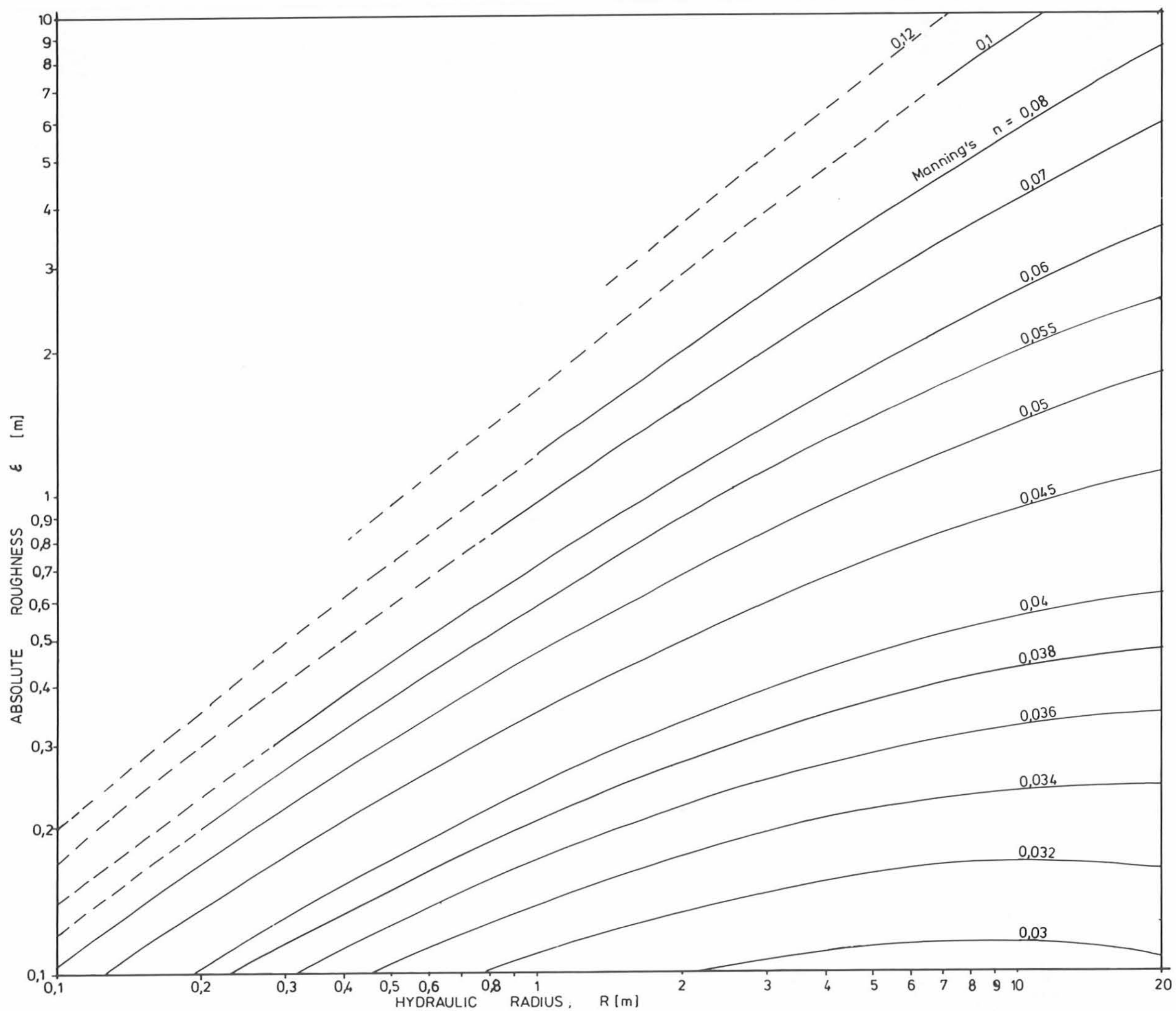


FIG. 11a. FLOOD PEAK FREQUENCY DISTRIBUTIONS AT SITE 26



MODIFIED COLEBROOK — WHITE FORMULA:

$$n = \frac{R^{\frac{1}{6}}}{18 \log \frac{6R}{\varepsilon}} \quad [s/m^{\frac{1}{3}}]$$

FIG. 10. ABSOLUTE ROUGHNESS VS MANNING'S n IN ROUGH NATURAL CHANNELS

[USING THE MODIFIED COLEBROOK — WHITE FORMULA]

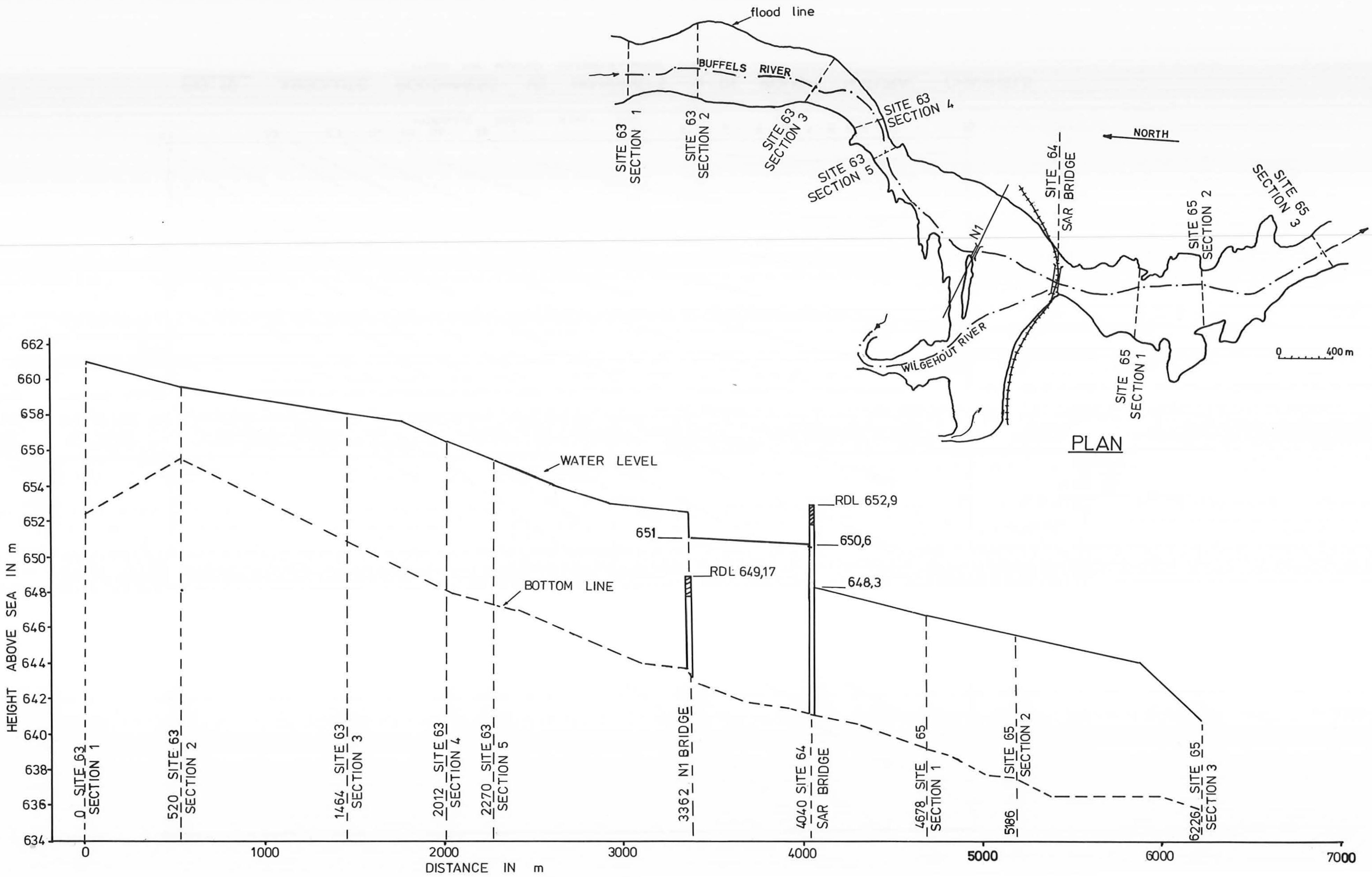


FIG. 9 c. FLOOD PROFILE AND PLAN OF BUFFELS RIVER IN AND AROUND LAINGSBURG

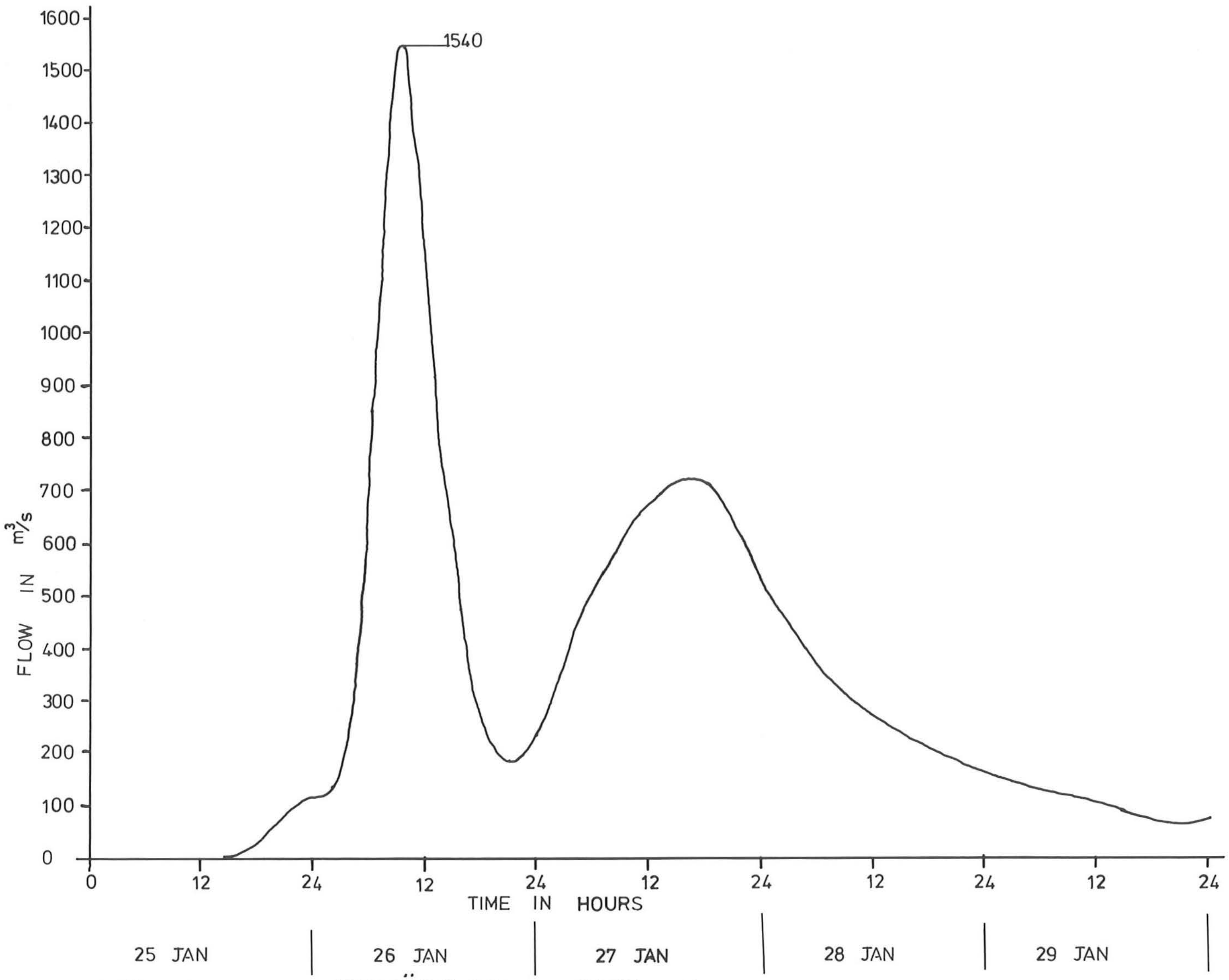


FIG. 14 c. BREËRIVER SITE 46

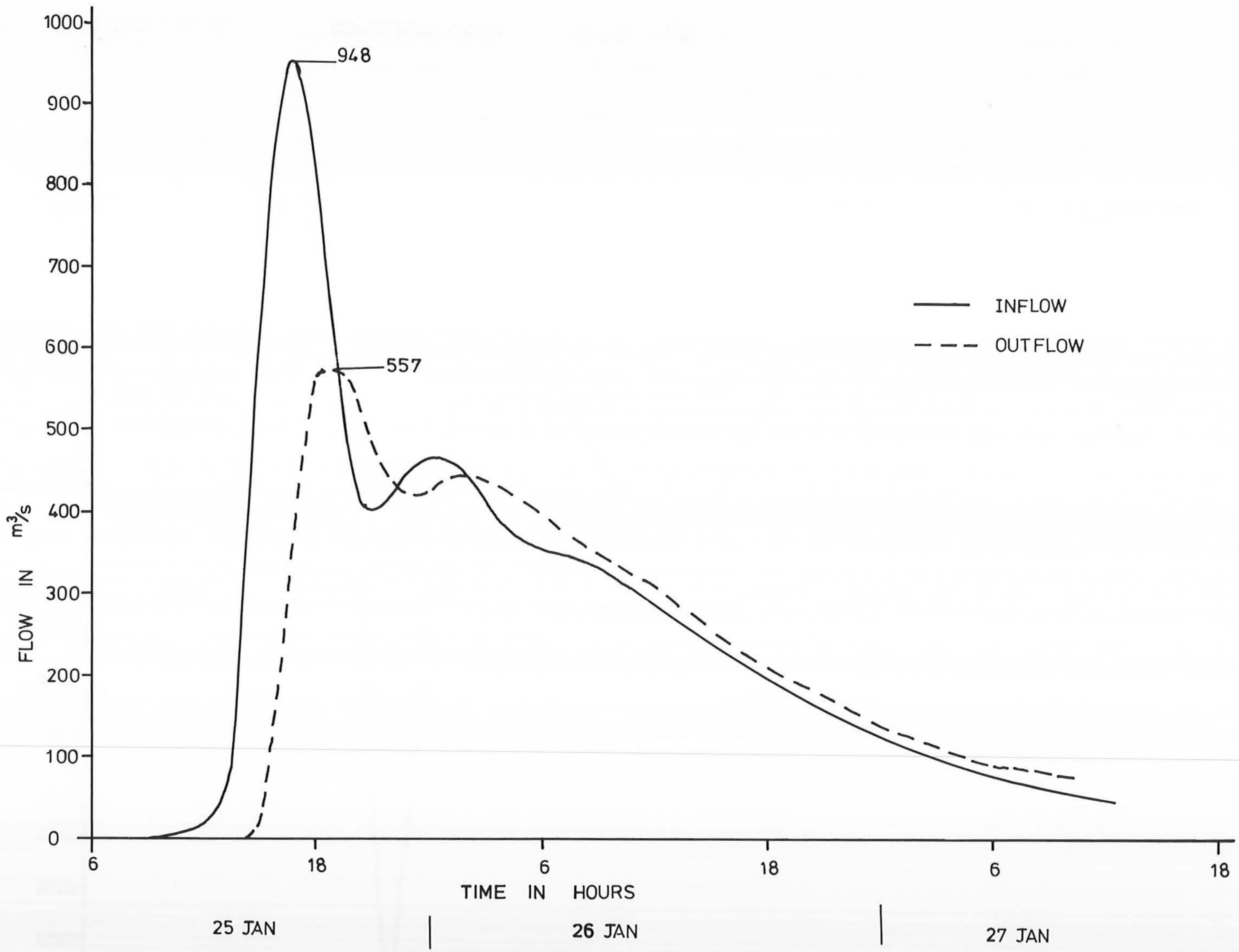


FIG. 14 d. KEEROMDAM SITE 18

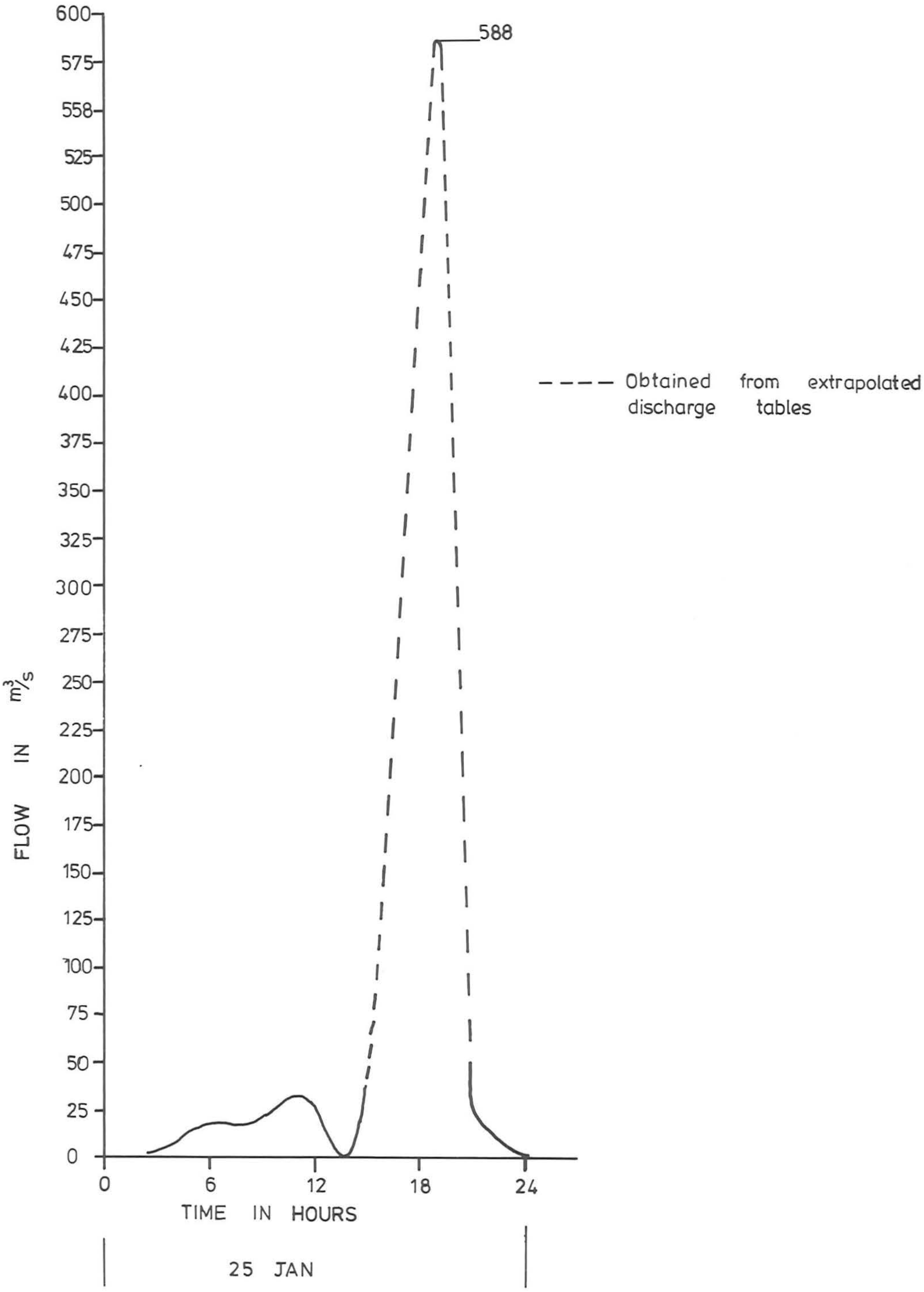


FIG. 14 e. WILLEM NELS RIVER SITE 27

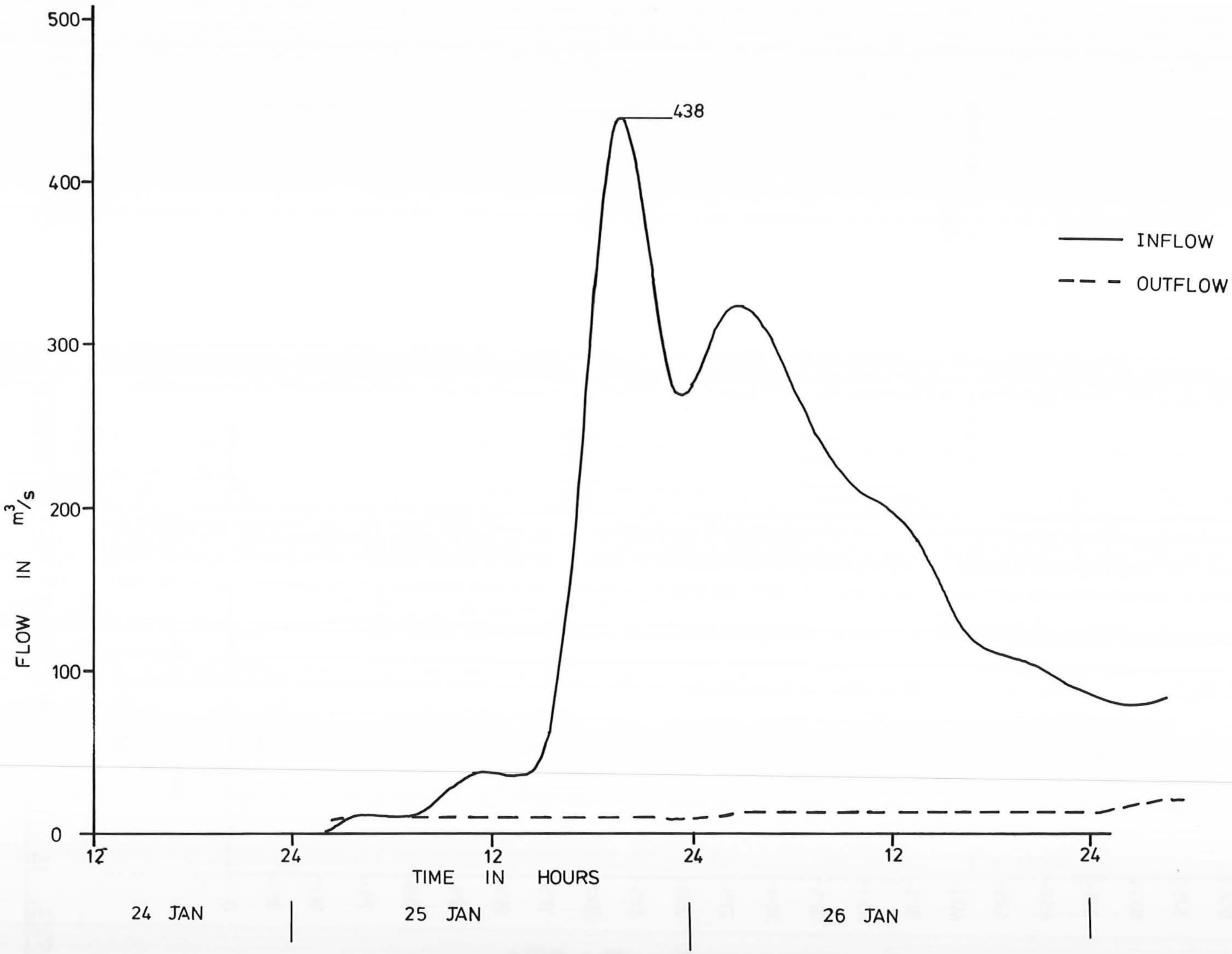


FIG. 14 f. THEEWATERSKLOOFDAM SITE 44

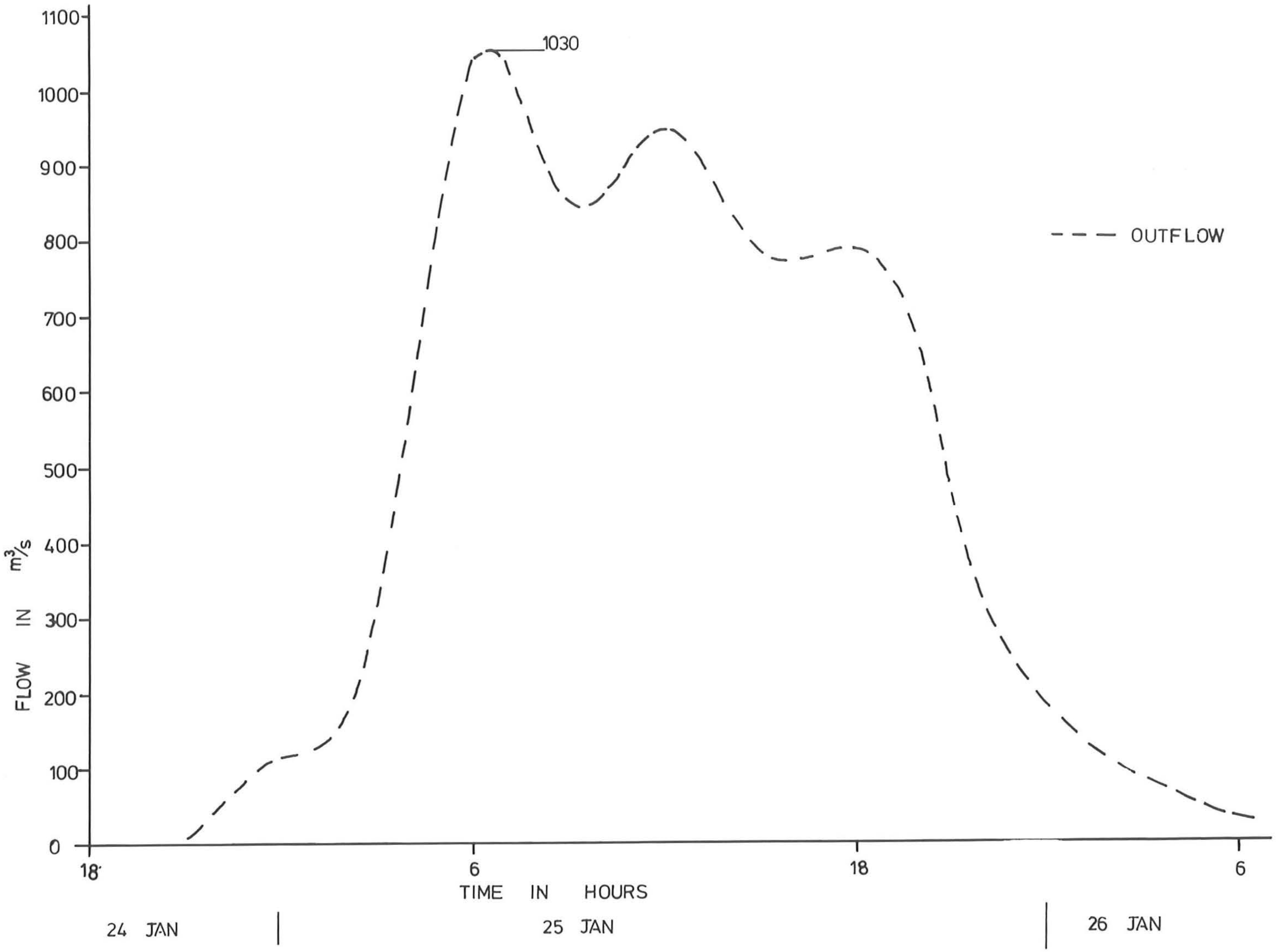


FIG. 14 g. PRINSRIVIERDAM SITE 56

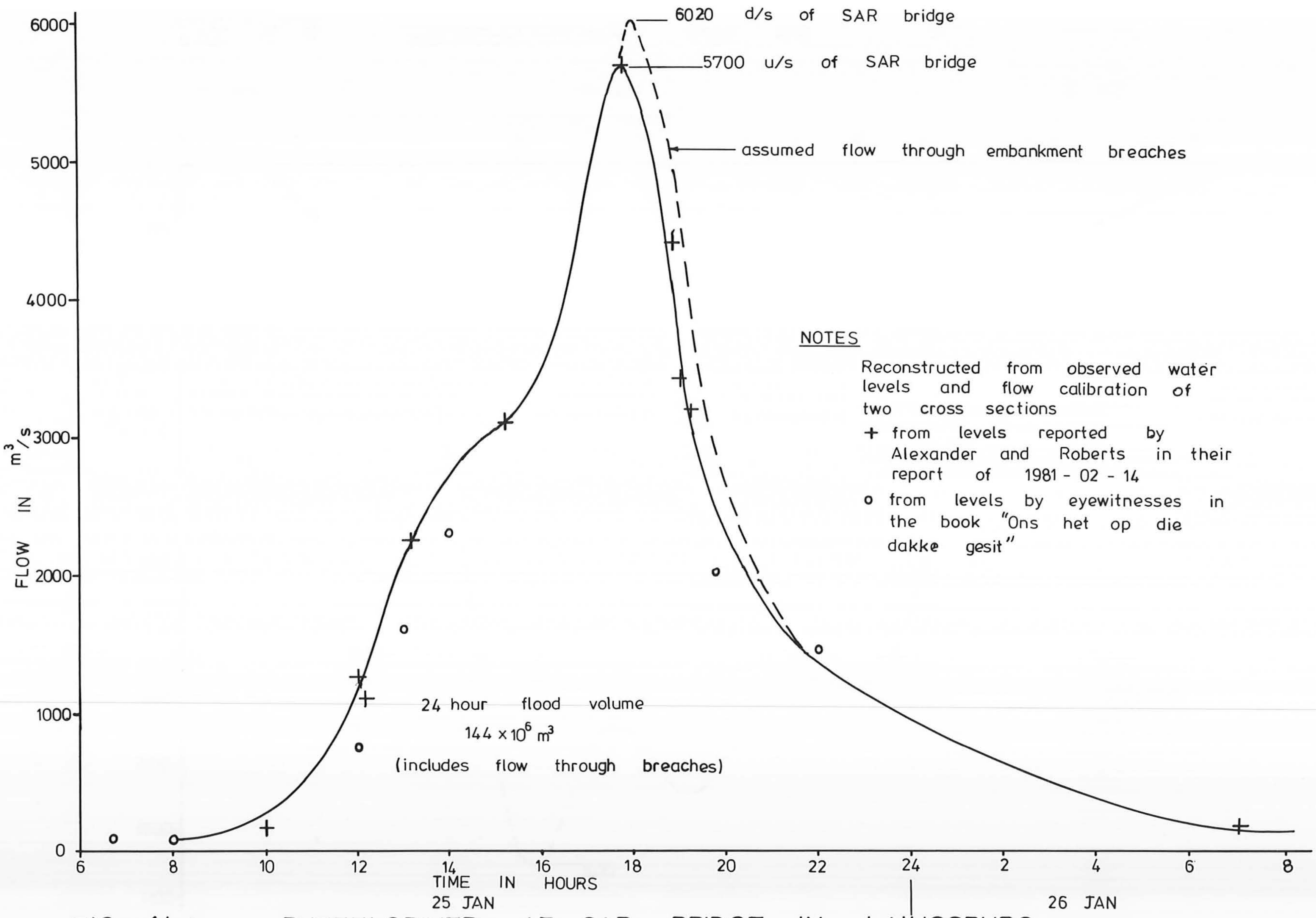


FIG. 14 h. BUFFELSRIVER AT SAR BRIDGE IN LAINGSBURG

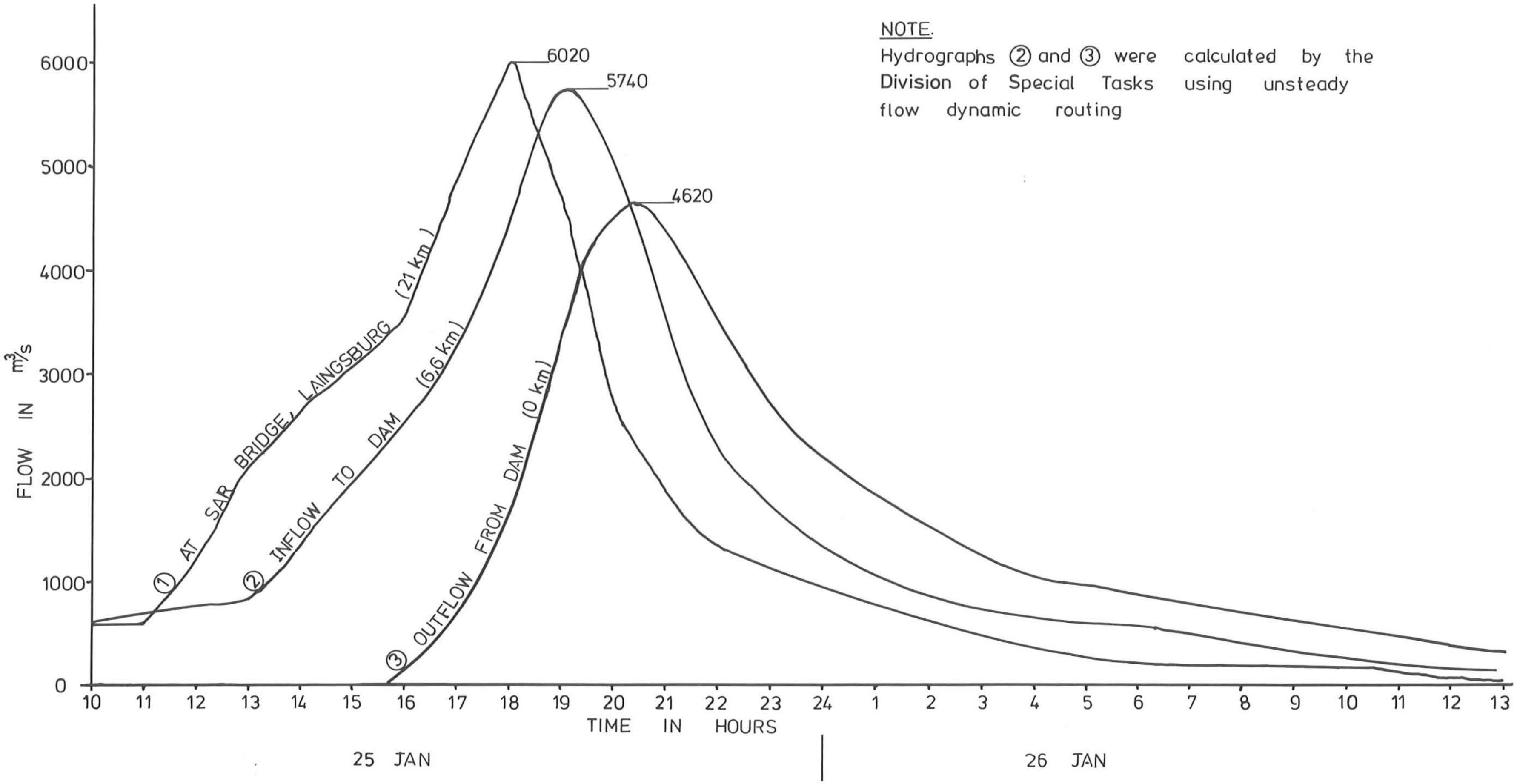


FIG. 14 i. BUFFELSRIVER AT LAINGSBURG AND IN THE FLORISKRAALDAM

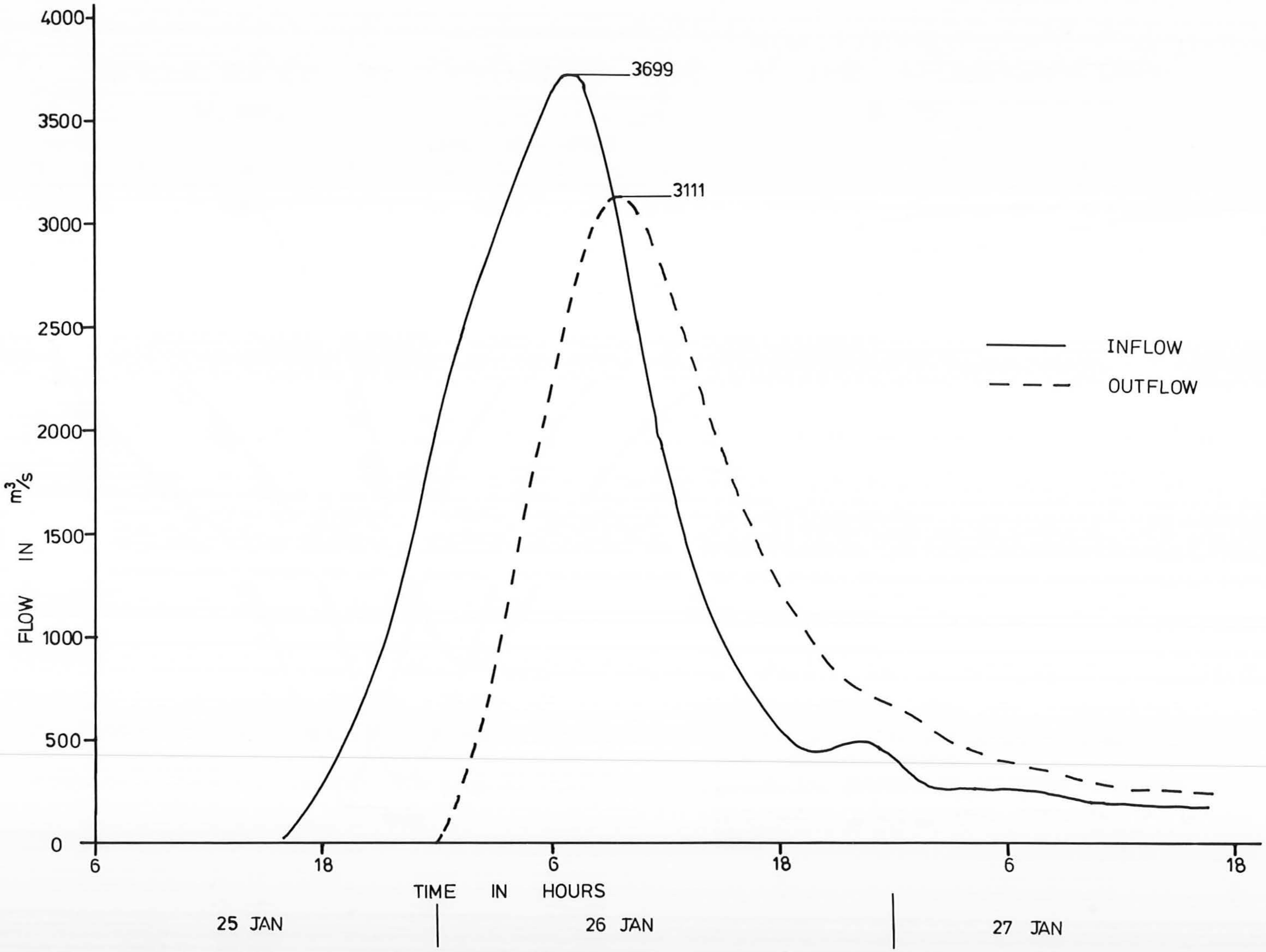


FIG. 14 j GAMKAPOORTDAM SITE 75

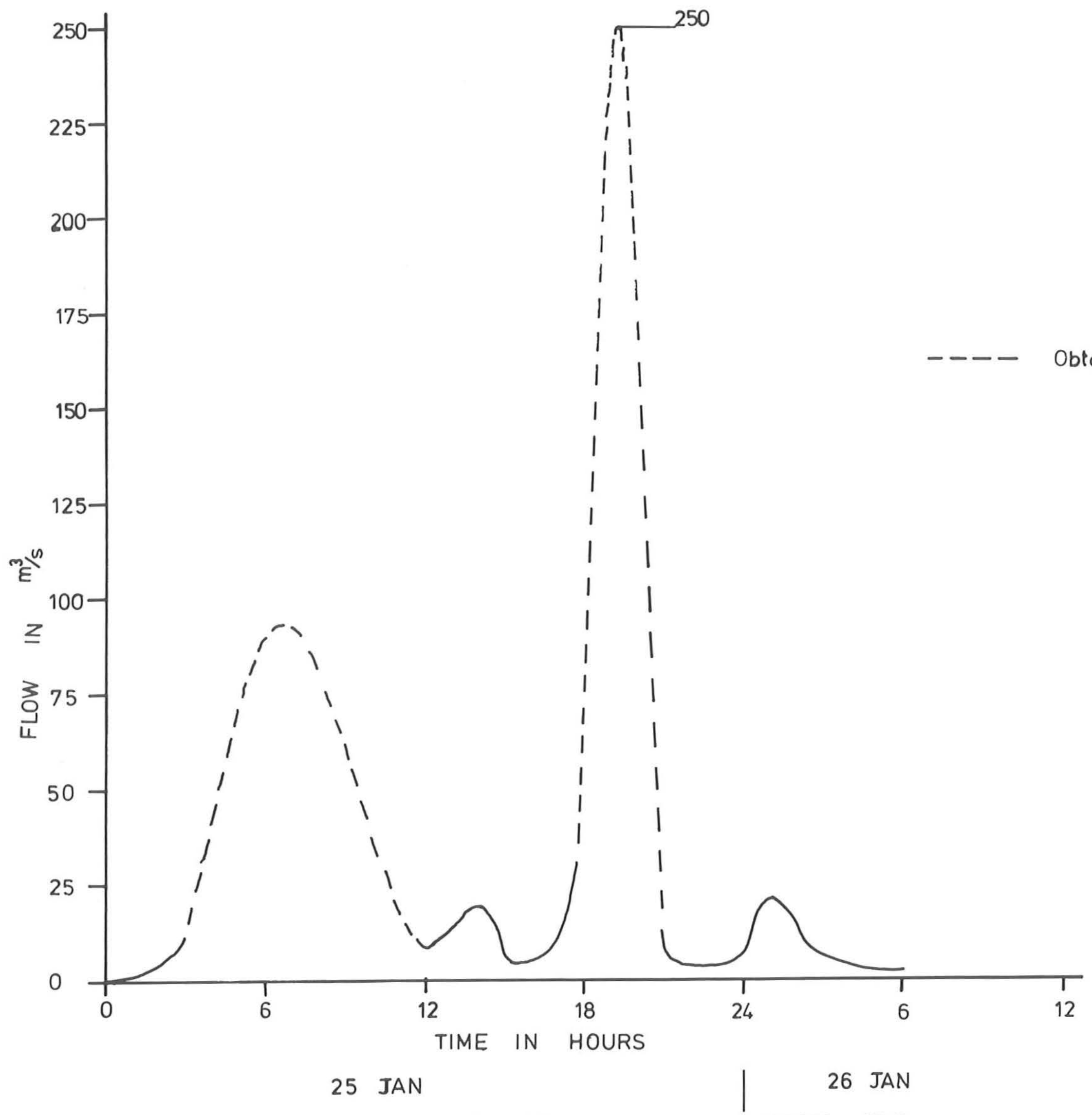


FIG. 14 k. WILGEHOUTRIVER SITE 78

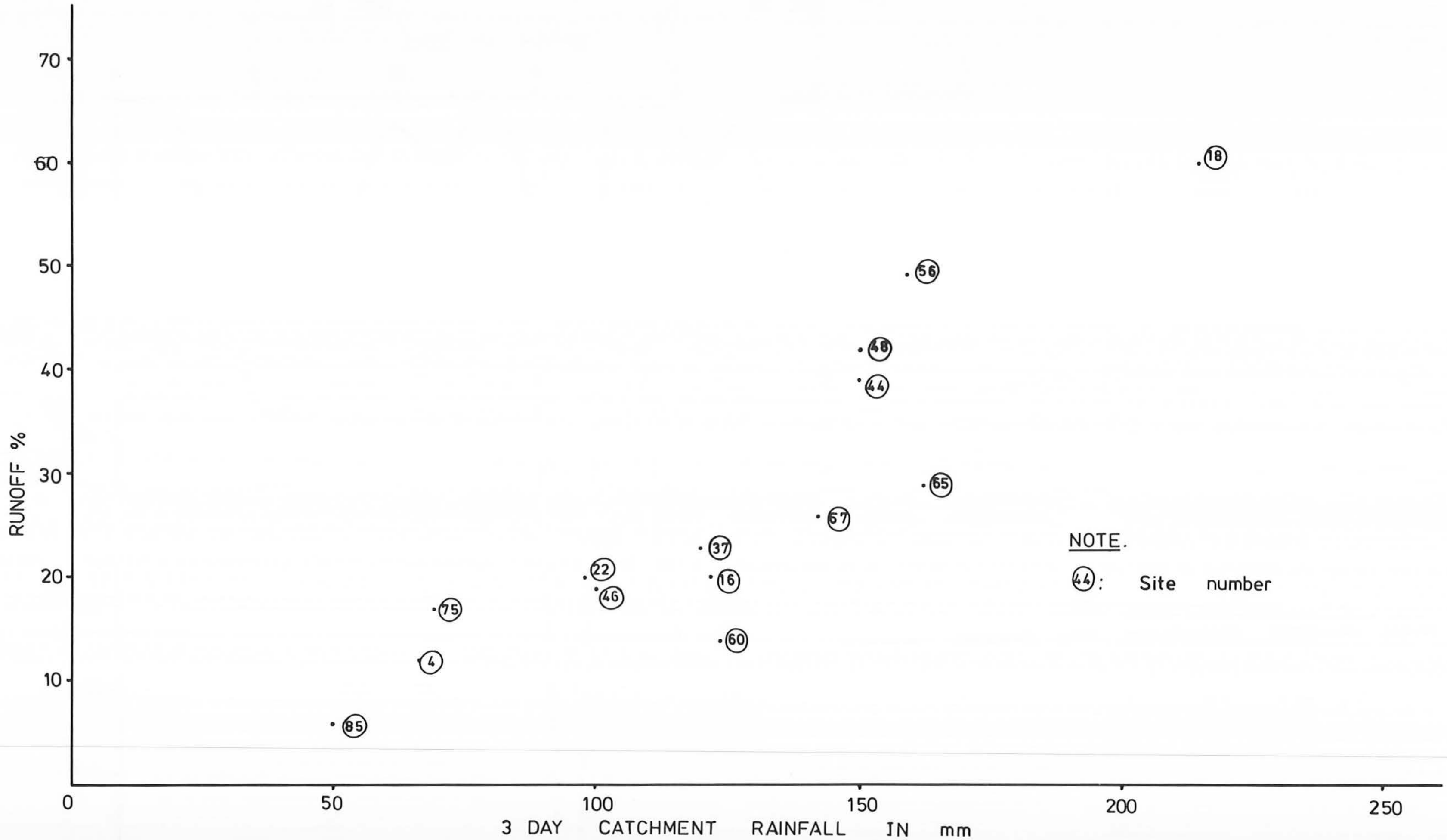


FIG. 15. RUNOFF % VS 3 DAY CATCHMENT RAINFALL IN AREAS > 200 km^2

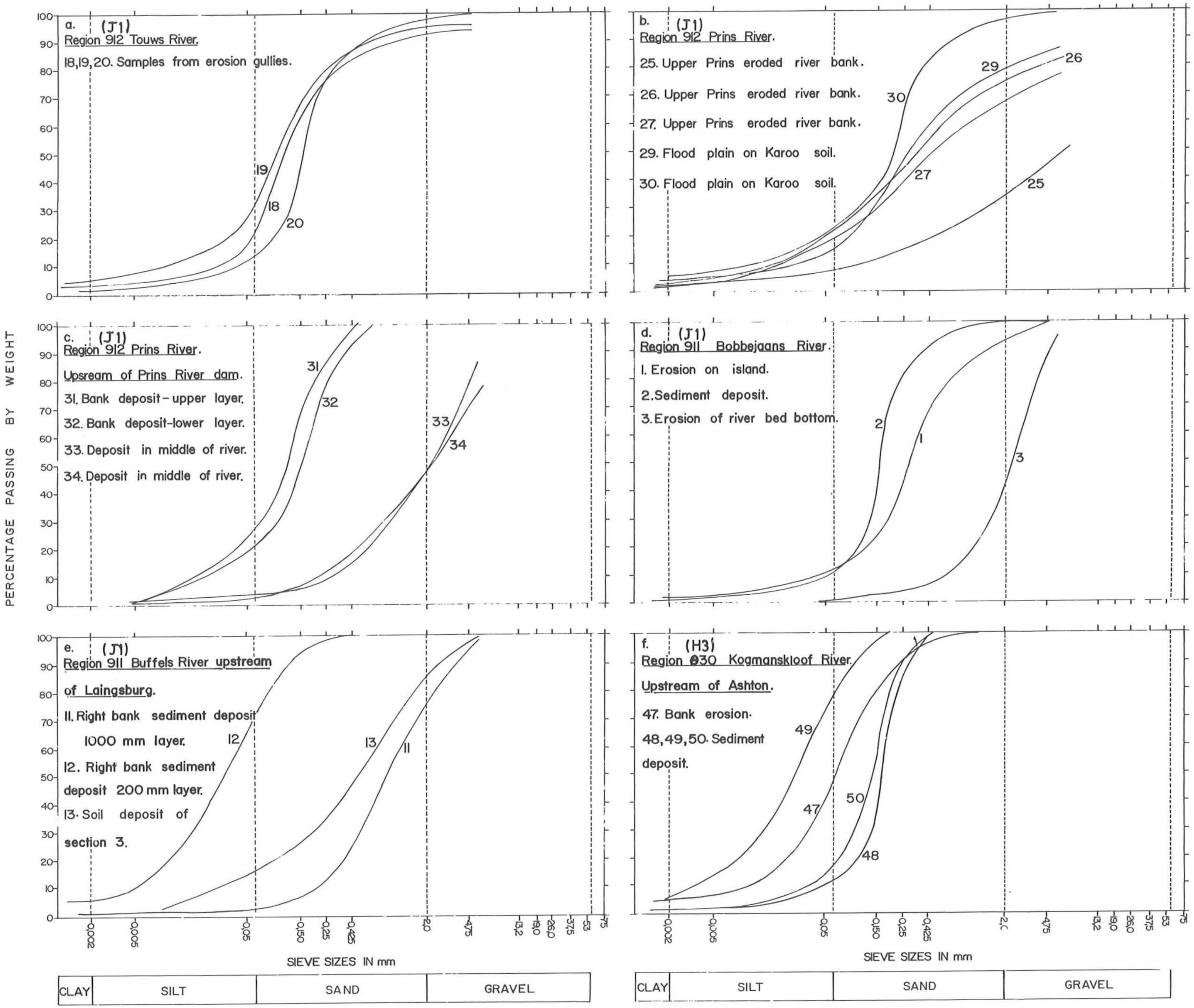


FIG. 16. SEDIMENT GRADING CURVES

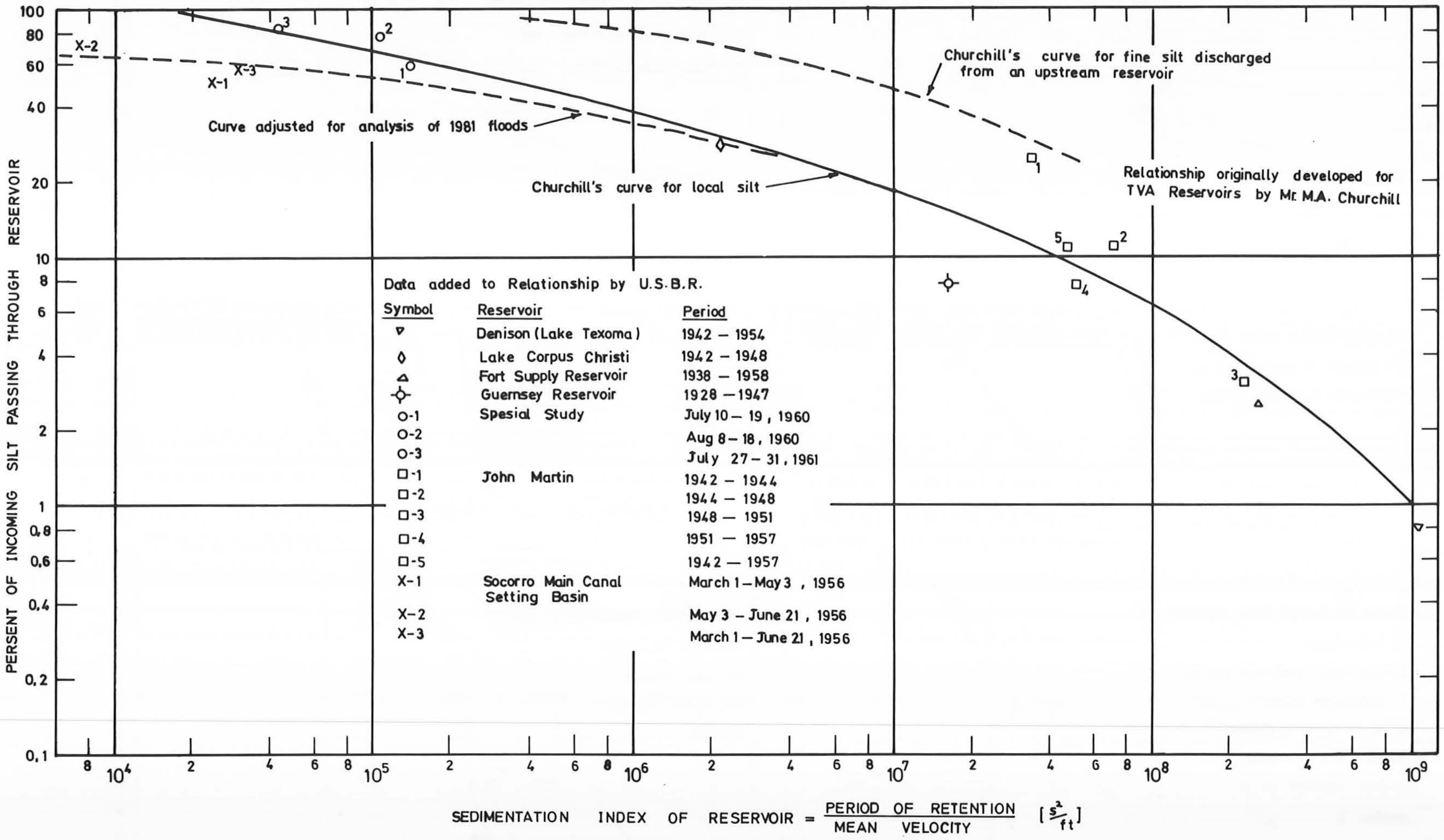


FIG. 17. RESERVOIR TRAP EFFICIENCY [FIG. H-5 IN REFERENCE C]

LIST OF PHOTOGRAPHS

24. Prins River : Channel erosion downstream of Prinsrivier Dam (site 56).

25. Touws River : Sediment deposits (site 59).

26. Touws River : Sediment deposits (site 59)

27. Laingsburg : Sediment deposits.

PHOTOGRAPHS IN APPENDIX II (FOR SLOPE-AREA CALCULATION EXAMPLE);

A1 Section 1L

A2 Section 1R

A3 Section 3L-M

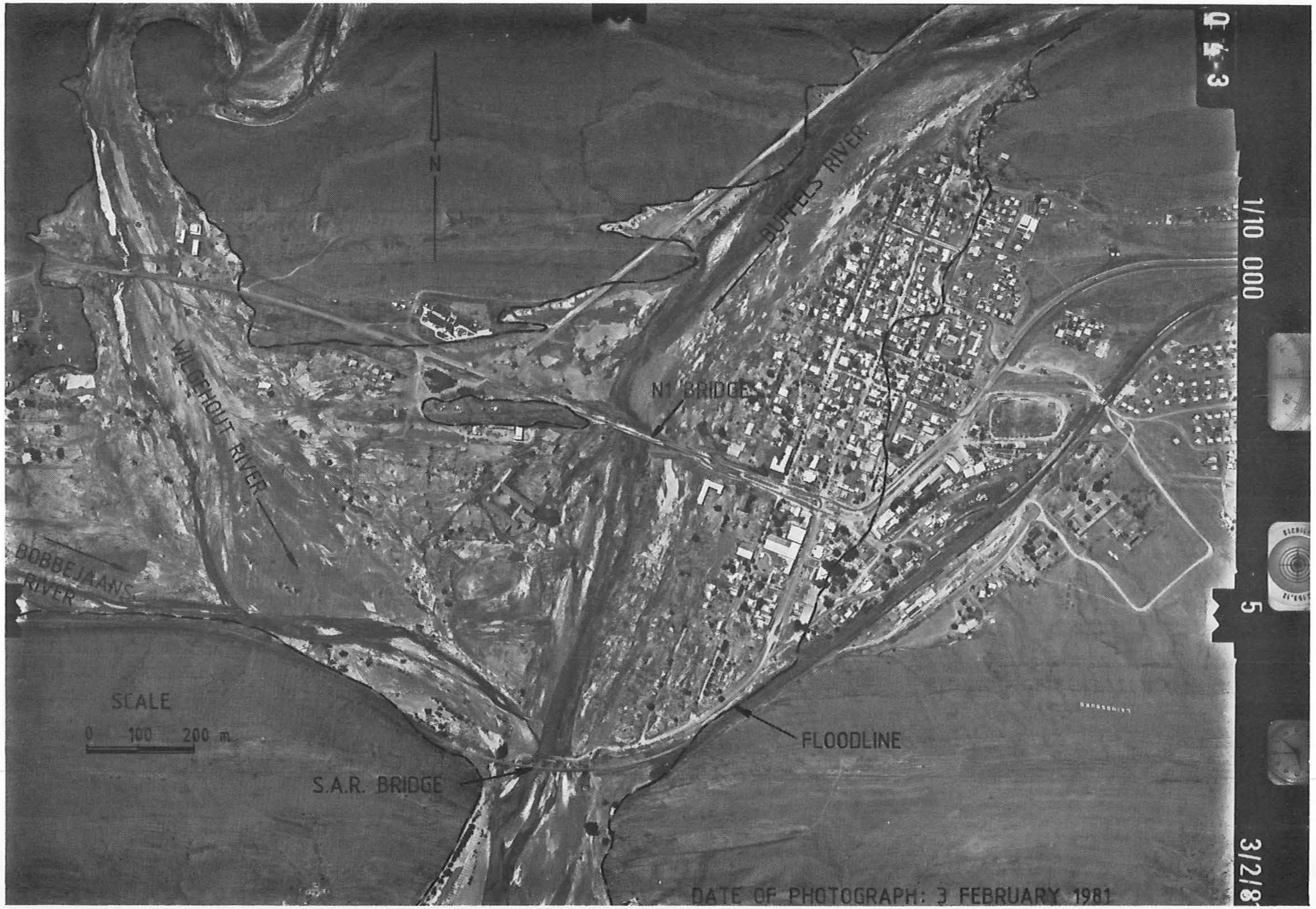
A4 Section 3R

A5 Section 4L-M

A6 Section 4R

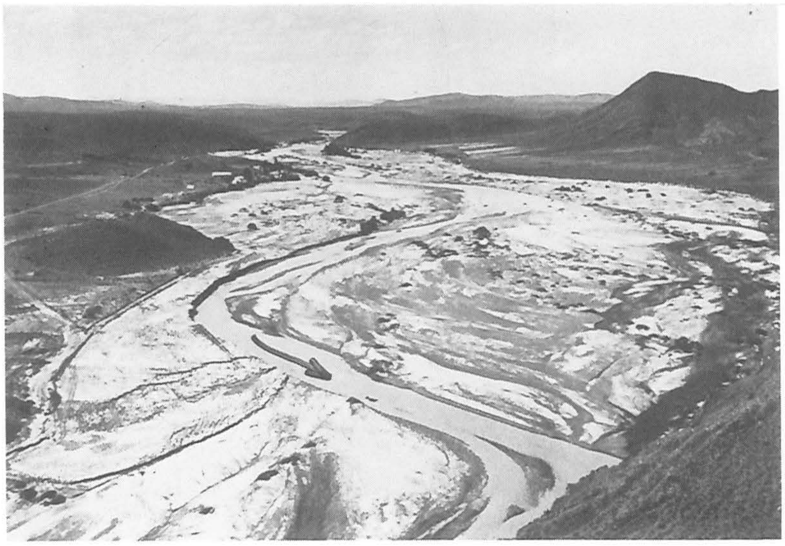
1. Laingsburg after the flood: general view from the south

2. Laingsburg after the flood: general view from the north



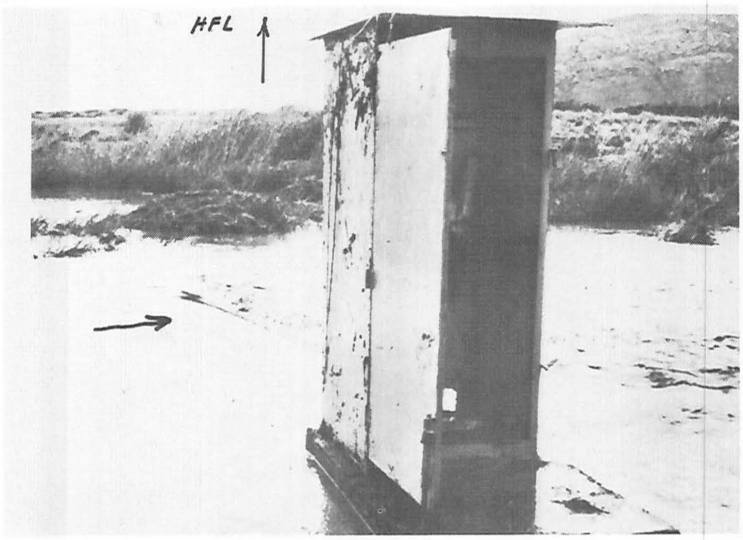
3. Laingsburg: aerial view after the flood

4. Laingsburg: area of complete destruction

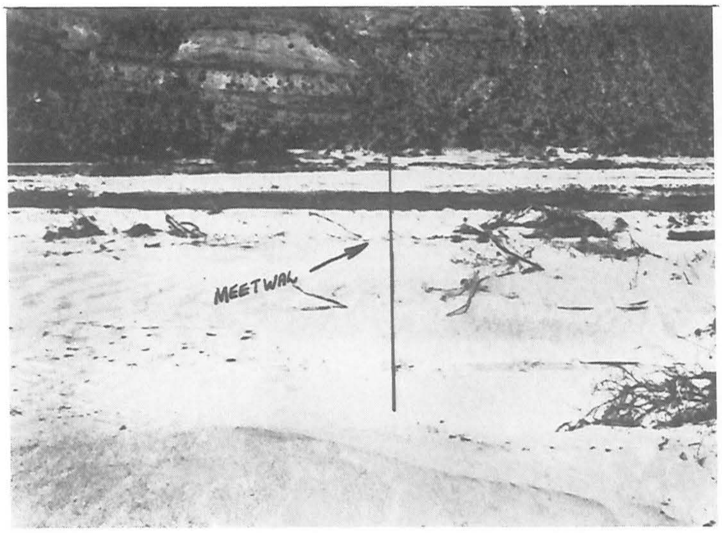


5. Buffels River: upstream view of slope-area reach (site 63)

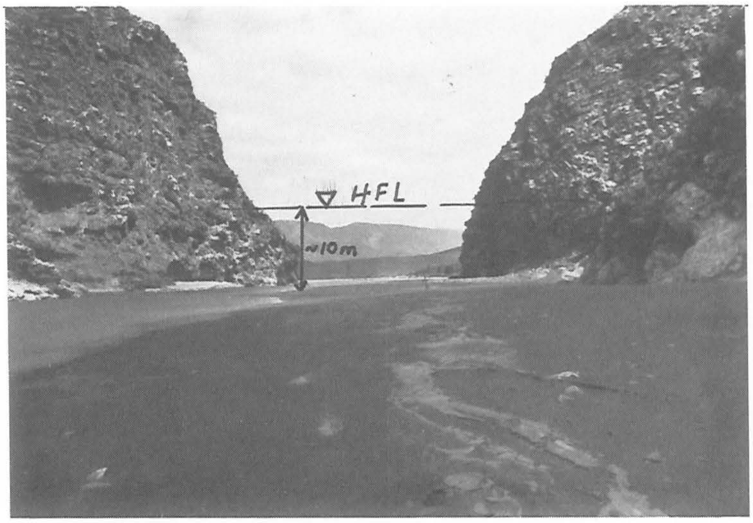
6. Willem Nels River: light damage to weir at gauging station H4M05 (site 26)



7. Nuy River: damage to recording hut at gauging station H4M04

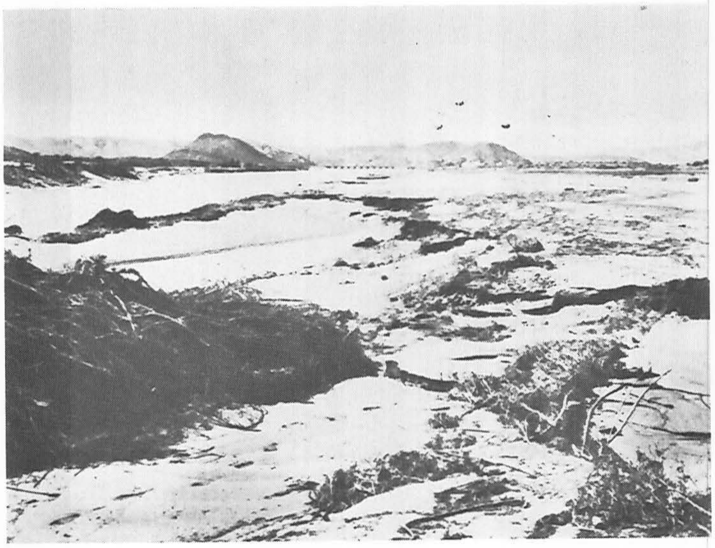


8. Touws River: site of washed away weir at gauging station J1M10

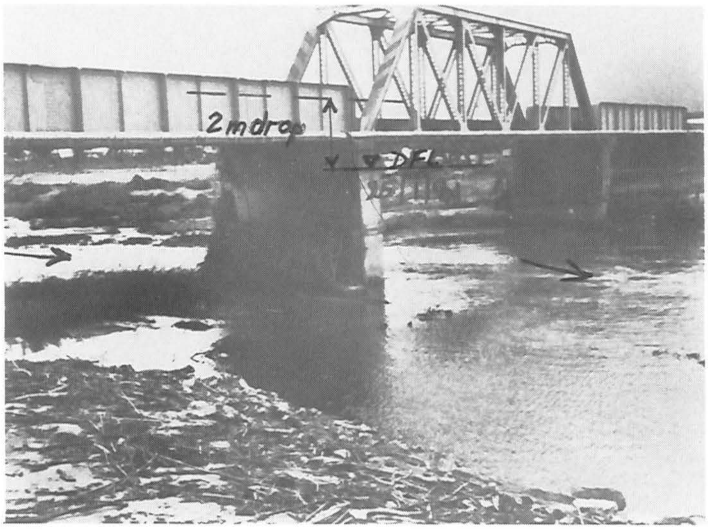


9. Buffels River: rough and steep river banks (site 69)

10. Buffels River: wide flood section (site 65)

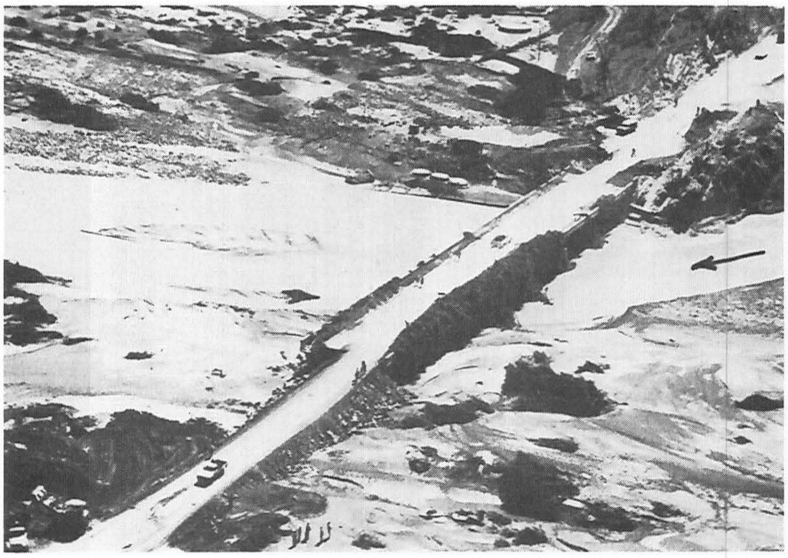


11. Touws River: erosion and deposits (site 59)



12. Kogmanskloof River: S.A.R. bridge at Ashton (site 36)

13. Gamka River: submerged road bridge during peak flow (site 76)

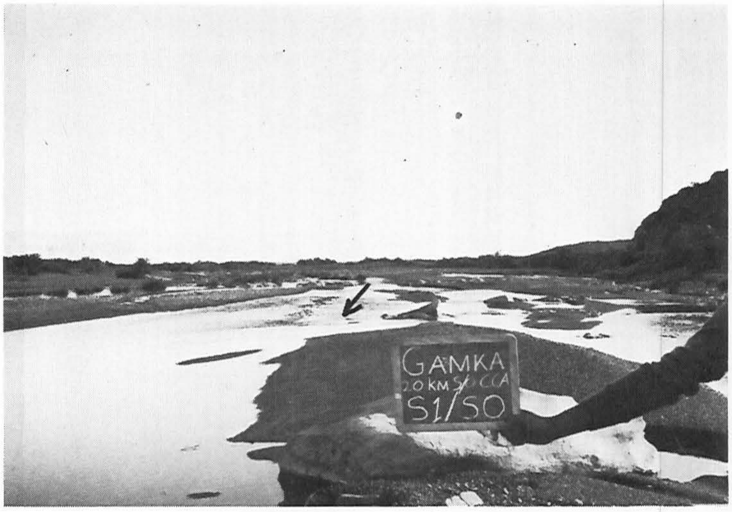


14. Buffels River: damaged N1 road bridge with debris at Laingsburg

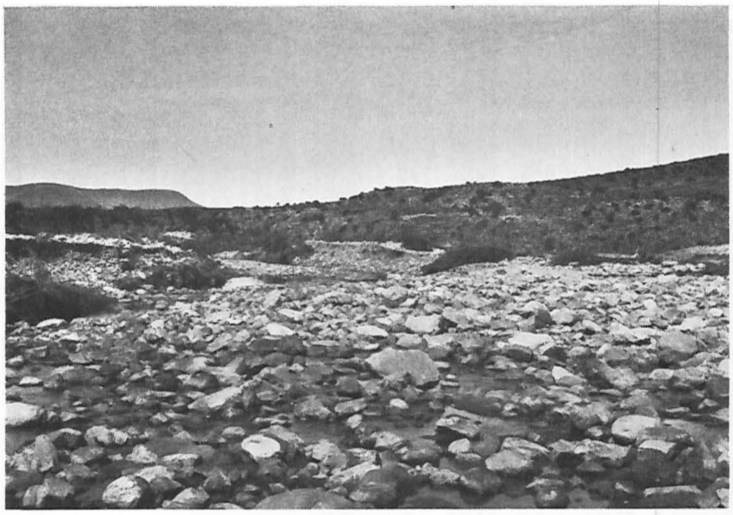
15. Buffels River: downstream view of Floriskraal Dam (site 67)

16. Geelbek River: smooth channel bottom, roughness ~0,2m (site 66)

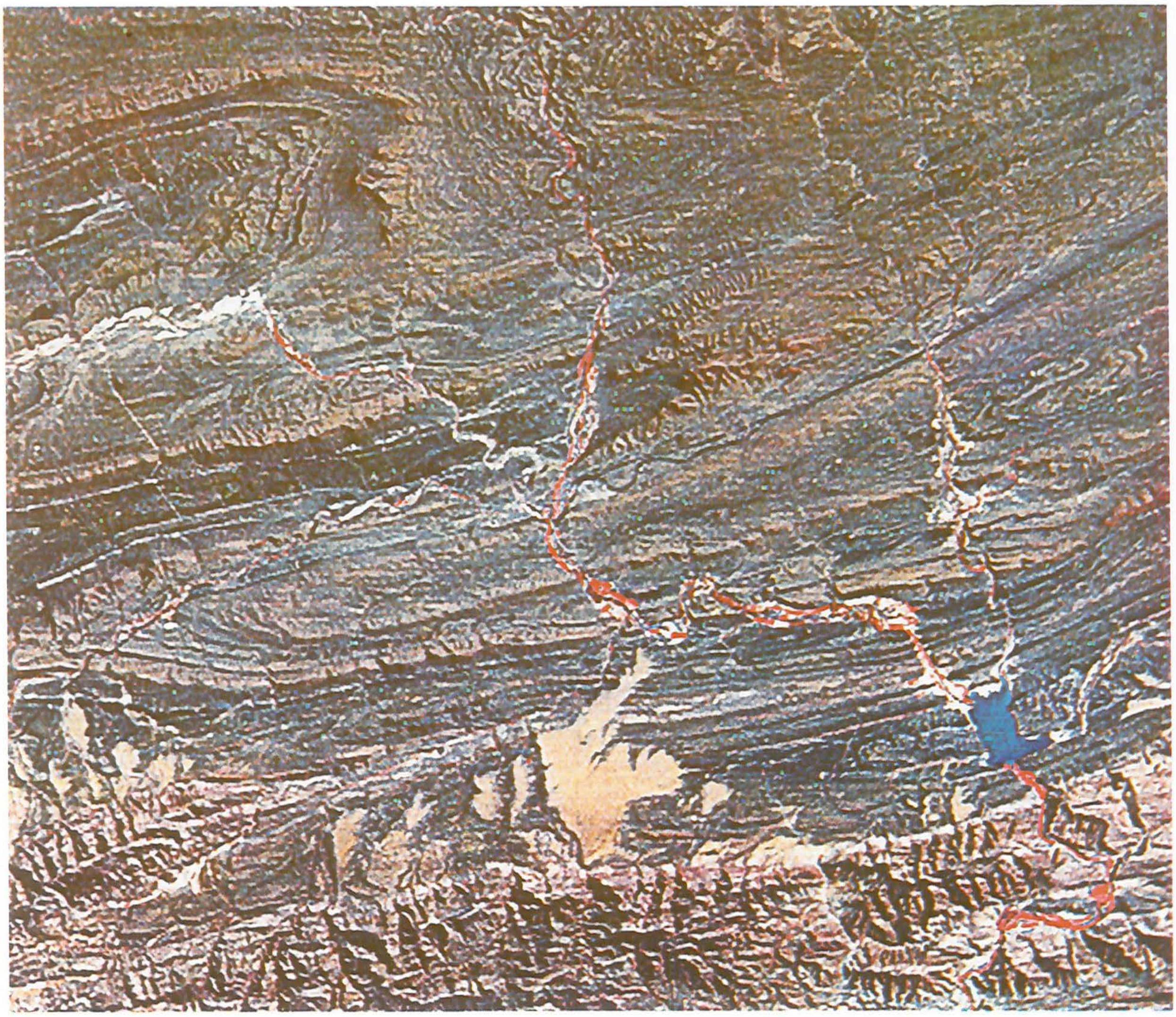
17. Joubert River: very rough channel, roughness ~ water depth (site 79)



18. Gamka River: roughness 0,4 m (site 74)

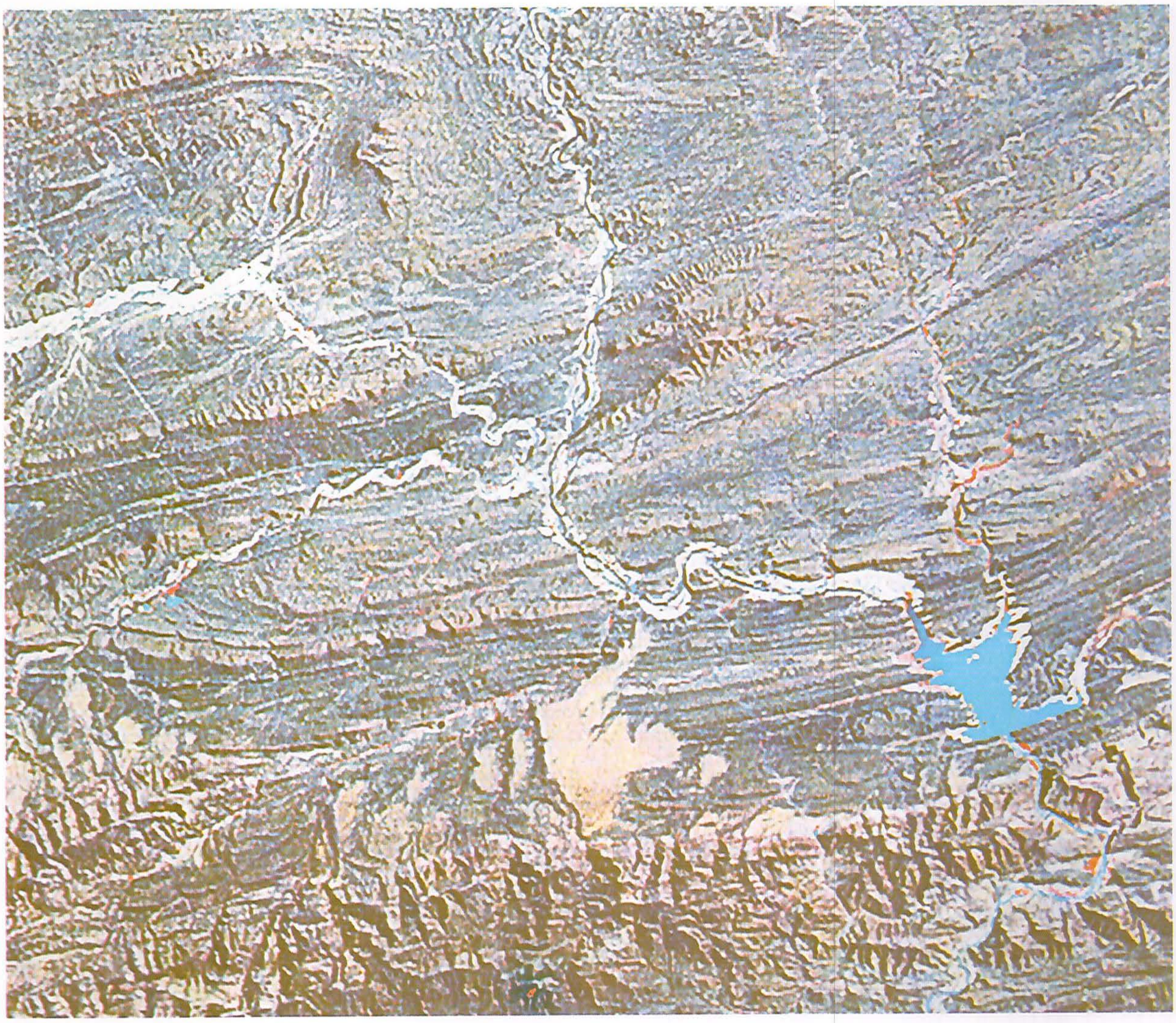


19. Touws River: roughness 0,5 m (site 55)



20. Buffels River catchment: Landsat image in November 1980

(The red colour along rivers indicates green irrigated zones. The Floriskraal Dam appears in the S.E. sector. The dark blue colour indicates relatively clear water.)

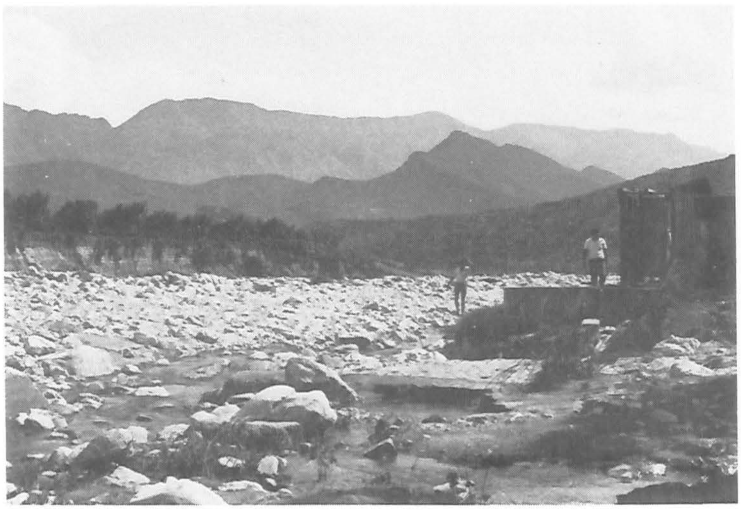


21. Buffels River catchment: Landsat image in February 1981

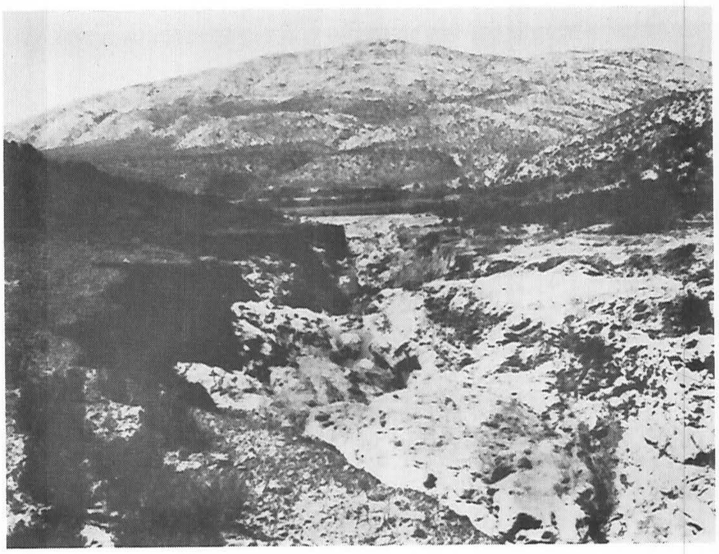
(Compared to the November image note the following changes:

(i) The irrigated zones along the Buffels River are wiped out.
(ii) The river channels are considerably enlarged.
(iii) The surface of Floriskraal Dam is much larger. The light blue colour indicates muddy water.)

22. Buffels River: sheet and channel erosion upstream of Laingsburg



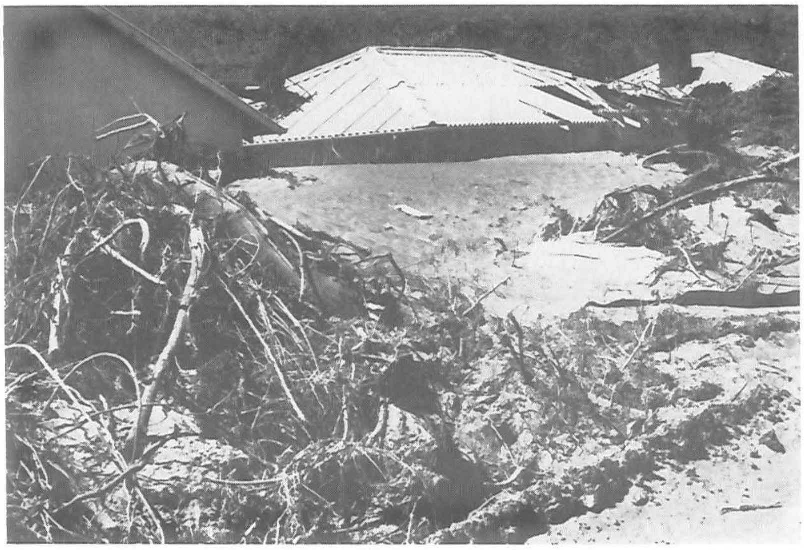
23. Baden River (near Montagu): bank erosion and boulder deposits



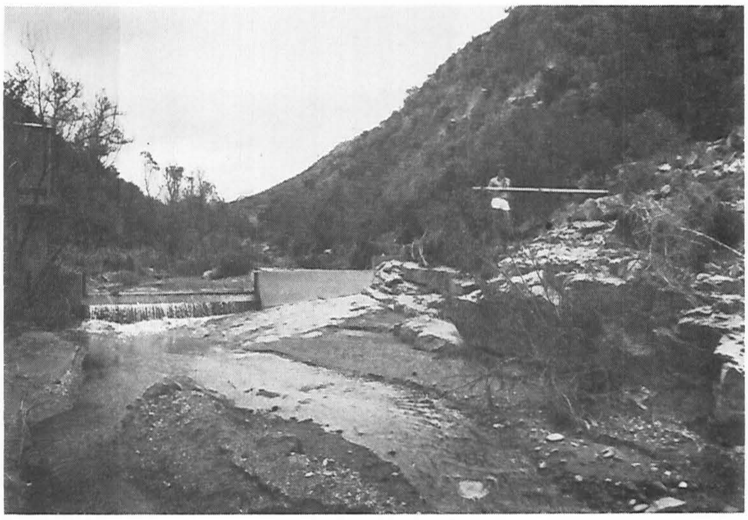
24. Prins River: channel erosion downstream of Prinsrivier Dam (site 56)

25. Touws River: sediment deposits (site 59)

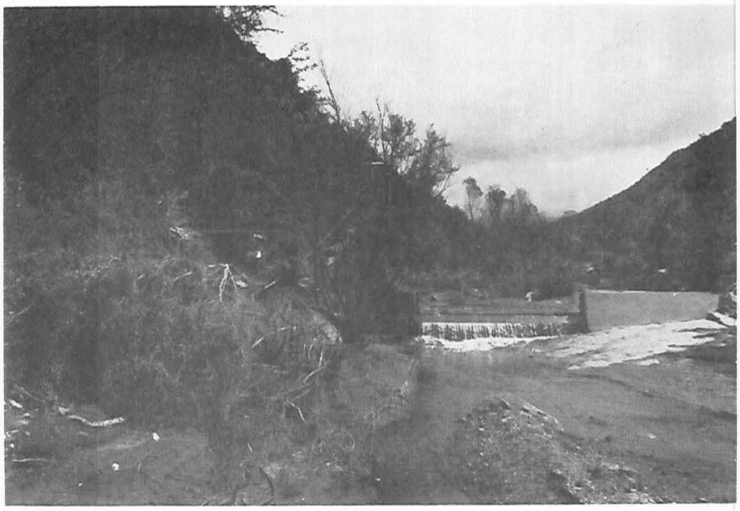
26. Touws River: sediment deposits (site 59)



27. Laingsburg: sediment deposits

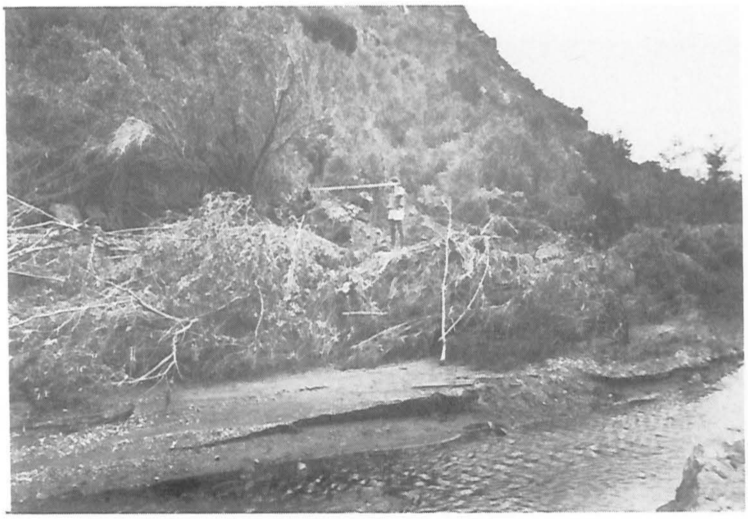


A1. Example for slope-area calculations: section X1, Left



A2. Example for slope-area calculations: section X1, Right

A3. Example for slope-area calculations: section X3, Left - Middle



A4. Example for slope-area calculations: section X3, Right

A5. Example for slope-area calculations: section X4, Left - Middle

A6. Example for slope-area calculations: section X4, Right

APPENDIX I SUMMARY OF RAINFALL AND FLOOD DATA IN 86 CATCHMENTS

SITE No	DRAINAGE REGION or STATION No	GEOGRAPHIC POSITION lat ° '	long ° '	RIVER	PLACE (farm, road, bridge, dam)	METHOD OF MEASUREMENT	CATCHMENT AREA A km²	FLOOD PEAK discharge m³/s	return period T_Q (yr)	Francou Rodier K	accuracy rating	time day hour	FLOOD VOLUME $10^6 m^3$	FH mm	RAINFALL OVER CATCHMENT 23Jan-26Jan(8am) P mm	return period Tp (yr)	RUN-OFF % $\frac{100FH}{P}$	REMARKS
(1)	(2)	(3)		(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
1	H1M13	33-21½	19-18	Koekedou	Persephone	G	53	12,2	<10	2,17	1	$25-21^{00}$	0,27	5,1	60	3	8,5	
2	H1M16	33-25¼	19-28¾	Rooikloof	Ben Etive	G	11	30	<10	3,49	1	$25-16^{00}$	0,68	62	85	6	72	
3	H1M14	33-25¾	19-24½	Vals	Ben Etive	G	18	116°	10-50	4,17	U	$25-15^{00}$	-	-	85	8	-	
4	H1M03	33-22¾	19-18¼	Breë	Ceres	G	657	148	<10	2,61	1	$25-18^{00}$	5,34	8,1	66	3	12	
5	H1M06	33-25¼	19-16	Breë	Witbrug	G	753	252	<10	2,98	2	$25-18^{00}$	-	-	75	4	-	
6	H1M07	33-34	19-09	Wit	Drosterskloof	G	84	115	<10	3,52	U	$25-16^{20}$	(4,65)	(55)	110	10	(50)	overestimated flood volume
7	H1	33-39	19-20¼	Breë	Bridge: N1 rd W of Worcester	B	1 267	520	±10	3,29	2	-	-	-	90	6	-	
8	H1M17	33-44	19-07	Elands	Hawequas Forest Reserve	G	61	111	±10	3,64	U	$25-20^{30}$	(3,26)	(53)	(65)	(2)	(81)	inaccurate flood volume and/or P
9	H1M18	33-43½	19-10¼	Molenaars	"	G	113	111	<10	3,35	1	$25-19^{00}$	3,6	32	70	2	46	
10	H1R02	33-50¼	19-15¼	Settynskloof	Settynskloof Dam	D	55	I 137	±10	3,83	1	$25-17^{00}$	4,9	89	110	3	81	
								Ø 10,2	-	-	1	$27-03^{00}$						
11	H1M20	33-33½	19-26	Hartbees	near Worcester	G	13	24	<10	3,29	1	$25-12^{00}$	1,0	78	110	30	71	
12	H2R02	33-22	19-34¾	Sanddrifskloof	Lakenvalley Dam	D	80	I 48	<10	2,91	1	$25-14^{00}$	3,0	38	107	16	35	
								Ø 0	-	-	-	-						
13	H2R01	33-26¼	19-34½	Sanddrifskloof	Roode Els Berg Dam	D	139	I 93	<10	3,53	1	$25-16^{00}$	8,7	63	110	15	57	net flood volume: $5,7 \times 10^6 m^3$
								Ø 60	-	-	1	$25-19^{30}$						
14	H2M04	33-29	19-31¾	Sanddrifskloof	Zanddrifts Kloof	G	175	(392)	10-50	(4,08)	U	$25-20^{00}$	(13,5)	(77)	115	16	(67)	overestimated floodpeak and volume
15	H2M06	33-34	19-30½	Hex	New Glen Heatlie	SA	700	230	<10	2,94	1	-	-	-	122	30	-	
16	H2M03	33-36	19-30½	Hex	De Wet	G	718	244	<10	2,98	1	$26-00^{00}$	18,0	25	122	30	20	net flood volume: $13,3 \times 10^6 m^3$
17	H4M07	33-38¼	19-48¾	Koo	Bridge: Montagu-Matroosberg rd	B	48	160°	10-50	3,99	U	25 after 16^{20}	-	-	215	180	-	
18	H4R02	33-35	19-42½	Nuy	Keerom Dam	D	377	I 948°	≥ 50	4,43	2	$25-17^{30}$	48,5	129	215	200	60	
								Ø 557	-	-	2	$25-19^{00}$						
19	H4	33-40½	19-34½	Nuy	Bridge: Worcester-Robertson rd.	B	525	442	10-50	3,65	2	-	-	-	200	≥200	-	
20	H4	33-42⅔	19-29⅔	Nuy	near Worcester	SA	573	418	≥ 10	3,55	2	-	-	-	190	≥ 200	-	
21	H4M13	33-51½	19-24½	Hoeks	Moddergat	SA	103	244°	10-50	3,97	2	-	-	-	160	18	-	
22	H4M14	33-46	19-32¾	Breë	Karroo	G	4 140	794	<10	2,93	1	$26-16^{00}$	80	19,2	98	10	20	net flood volume: $64 \times 10^6 m^3$
23	H4M12	33-57¼	19-35¼	Waterkloofspruit	Poesjenels River	G	14	47	≤10	3,69	1	$25-21^{00}$	1,2	84	130	60	64	
24	H4	33-52	19-43	Poesjenels	Bridge: Le Chasseur-Robertson rd	B	211	225	±10	3,57	1	-	-	-	110	35	-	
25	H4	33-46¾	19-46¾	Vink	Noree	SA	194	1 450°	±RMF	5,03	2	$25-19^{00}$	-	-	270	>200	-	
26	H4M05	33-45¾	19-52	Willem Nels	Langevalley	G	24	(551°)	±RMF	(5,08)	2	$25-19^{30}$	-	-	230	>200	-	overestimated peak
27	H4	33-47	19-51¾	Willem Nels	± 3km from Robertson	SA	32	588°	±RMF	5,03	2		(7,0)	(219)	240	>200	(91)	flood volume is rough approximation
28	H4R03	33-56½	19-47½	Konings	Klipberg Dam	D	54	I 17,3	<10	2,40	1	$25-14^{00}$	0,78	14,4	75	10	19	
								Ø 0,2	-	-	-	-						
29	H4M15	33-59½	19-49	Houtbaais	Rheebokskraal	G	25	11,8	<10	2,54	1	$25-15^{00}$	1,1	44	70	9	63	
30	H4M16	33-56	19-50¾	Keisers	Mc Gregor	G	117	36	<10	2,51	1	$25-19^{00}$	2,1	18,0	72	10	25	
31	H3R02	33-40¼	20-01	Pietersfontein	Pietersfontein Dam	D	116	Ø 576°	-	-	2	$25-17^{00}$	-	-	250	>200	-	peak inflow could have been considerably higher

NOTES: Col 6: G= river gauging station D= dam SA= slope-area B= bridge contraction
Col 8: ° = biggest on record or in living memory at site I = inflow to dam Ø = outflow from dam
Col 11: 1= error less than ± 10% 2 = error less than ± 30% U= unknown accuracy

(1)	(2)	(3)		(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
32	H3	33 - 41	20 - 02¼	Pietersfontein	2km d/s of Pietersfontein Dam	SA	120	673°	≥ 200	4,64	2	-	-	-	240	>200	-	
33	H3M05	33 - 42½	20 - 03½	Keisie	Keisies Doorns	G	78	120°	± 10	3,58	U	25-15^{30}	4,4	56	141	40	40	
34	H3	33 - 48¼	20 - 05½	Kogmanskloof	"Loftus" bridge: Montagu-Robertson rd	B	999	1 460°	± 50	4,33	2	-	-	-	150	65	-	
35	H3	33 - 49½	20 - 05¼	Kogmanskloof	Kogmanskloof	SA	1 001	1 330°	≤ 50	4,25	2	-	-	-	150	65	-	
36	H3	33 - 50¼	20 - 04¼	Kogmanskloof	SAR bridge at Ashton	B	1 031	1210°	≤ 50	4,15	2	25-20^{30}	-	-	150	65	-	
37	H5M04	33 - 53¾	20 - 00¾	Breë	Wolvendrift	G	6 690	(2 516)	10 - 50	(3,77)	2	26-02^{00}	(183)	(27)	120	35	(23)	overestimated flood peak and volume
38	H5M03	34 - 02½	19 - 58¾	Boesmans	Boschjesmans River	G	25	78	± 10	3,78	1	25-20^{00}	2,7	110	(75)	(9)	-	rainfall is too low
39	H6M08	34 - 03¾	19 - 04¼	Riviersonderend	Nuweberg Forest Reserve	G	38	110	≥ 10	3,83	2	25-10^{00}	-	-	98	6	-	
40	H6M11	34 - 05¾	19 - 07½	Waterkloof	Tydsgenoeg	G	11	6,6	< 10	2,56	U	26-00^{30}	-	-	105	3	-	
41	H6M07	33 - 56½	19 - 10¼	Du Toits	Purgatory Outspan	G	46	79	≤ 10	3,53	2	25-19^{00}	2,1	45	180	12	25	
42	H6R02	33 - 57	19 - 17	Elands	Elandskloof Dam	D	50	I 81 Ø 61	< 10 -	3,51 -	1 1	25-19^{00} 25-22^{00}	2,4	48	180	12	27	
43	H6M10	33 - 59	19 - 19¾	Waterkloof	CDN.Q7-26	G	15	53	± 10	3,73	2	25-20^{00}	1,0	69	190	35	36	
44	H6R01	34 - 04¾	19 - 17½	Riviersonderend	Theewaterskloof Dam	D	497	I 438 Ø 20	10 - 50 -	3,67 -	1 1	25-20^{00} -	29	59	150	10	39	
45	H6M05	34 - 01¾	19 - 33½	Baviaans	Genadendal	G	24	110°	10 - 50	4,02	2	25-20^{00}	-	-	160	140	-	
46	H7M06	34 - 04	20 - 24¼	Breë	SW.Q.20-19	G	9 842	1540	± 10	2,98	1	26-09^{00}	184	18,7	100	14	19	net flood volume: 136 $\times 10^6 m^3$
47	H7M04	33 - 54¾	20 - 42¾	Huis	Barrydale	G	28	100°	10 - 50	3,90	U	25-21^{30}	-	-	155	20	-	
48	H7R01	34 - 01¼	20 - 32	Buffelsjagt	Buffelsjagt Dam	D	601	I ≥ 297 Ø 297	< 10	3,24	1	25-09^{00} 25-10^{00}	38	63	150	14	42	
49	H8R01	33 - 59¾	20 - 57	Duivenhoks	Duivenhoks Dam	D	148	I ≥ 194 Ø 194	± 10	3,63	1	25-22^{30} 25-24^{00}	15,2	103	170	35	60	
50	H9R01	34 - 00¼	21 - 10	Korinte	Korinte-Vet Dam	D	37	I 76 Ø 42	< 10 -	3,60 -	1 1	25-07^{30} 25-13^{00}	4,0	108	157	20	69	
51	H9M02	34 - 00¾	21 - 12	Vet	The Camp	G	89	270°	10 - 50	4,10	2	26-12^{00}	-	-	163	25	-	
52	H9M05	34 - 05½	21 - 17⅔	Kafferkuils	Klein Palmietrivier	SA	228	358°	10 - 50	3,89	2	-	-	-	130	14	-	
53	J1M16	33 - 17¼	19 - 43¾	Smalblaar	Verlorenvalley	G	30	27	< 10	3,00	1	25-14^{00}	0,58	19,3	110	18	18	
54	J1M15	33 - 21¼	19 - 43¼	Bok	Lot B	G	8,8	8,6	< 10	2,82	1	25-14^{00}	0,17	19,3	115	20	17	
55	J1	33 - 29	20 - 19¼	Touws	Bloutoring	SA	2 195	1 670°	≤ 50	4,04	2	-	-	-	163	> 200	-	
56	J1R01	33 - 31	20 - 45½	Prins	Prinsrivier Dam	D	757	I ≥ 1 030° Ø 1 030	≤ 50	4,17	2	25-09^{00} 25-12^{00}	59	78	159	> 200	49	
57	J1	33 - 32	20 - 15½	Prins	Kruitfontein	SA	881	840°	10 - 50	3,92	2	-	-	-	149	> 200	-	
58	J1R02	33 - 42½	20 - 36	Brak	Bellair Dam	D	558	I (68)° Ø 0	- -	- -	U U	25-23^{00} -	(3,8)	(6,8)	(185)	(>200)	(4)	unrealistically low flood peak and volume, rainfall is probably too high
59	J1	33 - 42½	21 - 10	Touws	Ockertskraal	SA	5 803	3 650°	50 - 200	4,25	2	-	-	-	166	> 200	-	
60	J1R04	33 - 49¾	21 - 08	Brand	Miertjeskraal Dam	D	251	I ≥ 104 Ø 104	< 10	2,89	2	25-23^{00} 26-01^{00}	4,2	16,7	124	18	14	
61	J1	33 - 11¾	20 - 45⅓	Bobbejaans	Baviaanskrants	SA	217	375°	10 - 50	3,95	2	-	-	-	123	≥ 200	-	
62	J1	33 - 09	20 - 47	Wilgehout	Zoutekloof	SA	361	1 230°	≥ 200	4,65	2	-	-	-	164	> 200	-	
63	J1	33 - 11	20 - 51¾	Buffels	± 2 km u/s of Laingsburg	SA	2 375	4 350°	> 200	4,89	2	-	-	-	165	> 200	-	
64	J1	33 - 12	20 - 51	Buffels	SAR bridge at Laingsburg	B	3 070	5 680°	± RMF	5,02	1	25-17^{45}	-	-	162	> 200	-	
65	J1	33 - 12½	20 - 51⅓	Buffels	2km d/s of Laingsburg	SA	3 072	6 020°	± RMF	(5,08)	1	25-18^{00}	144	47	162	> 200	29	embankments of S.A.R. bridge collapsed during peak flow
66	J1	33 - 10¾	20 - 59⅓	Geelbek	d/s of N1 and SAR bridges	SA	338	132	< 10	2,91	2	-	-	-	72	20	-	

(1)	(2)	(3)		(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
67	J1R03	33 - 17 1/2	20 - 59 1/2	Buffels	Floriskraal Dam	D	4 001	I 5 740°	> 200	4,90	1	$25-19^{00}$	145	36	142	> 200	26	flood peak and volume calculated by Division of Special Tasks
								Ø 4 620	-	-	2	$25-20^{15}$						
68	J1	33 - 18 1/2	21 - 00	Buffels	2km d/s of Floriskraal Dam	SA	4 005	3 630°	≤ 200	4,45	2	-	-	-	142	> 200	-	flood peak is probably underestimated
69	J1M11	33 - 27 3/4	20 - 50 1/4	Buffels	Slang Gat	SA	4 700	4 680°	> 200	4,62	2	$25-21^{00}$	-	-	133	170	-	
70	J1M12	33 - 39	21 - 10 1/2	Groot(=Buffels)	Baviaans Krans	SA	5 565	5 250°	> 200	4,64	2	-	-	-	134	170	-	
71	J1	33 - 44 1/2	21 - 24 1/2	Groot	Conradie	SA	12 466	11 000°	± RMF	4,98	2	$26-02^{30}$	-	-	148	> 200	-	flood peak is probably overestimated
72	J2	32 - 31 3/4	21 - 14	Dwyka	Rietvalley	D	74	Ø 320°	≤ 50	4,30	U	-	-	-	125	50	-	surveyed by Vorster, v.d. Westhuizen, consulting engineers
73	J2	33 - 07 1/4	21 - 35 1/4	Dwyka	±5km d/s of N1 bridge	SA	3 073	930	10 - 50	3,28	2	-	-	-	107	30	-	
74	J2	33 - 01	21 - 59 3/4	Gamka	Kweekkraal	SA	9 350	2 239	10 - 50	3,42	2	-	-	-	71	12	-	
75	J2R06	33 - 18 1/2	21 - 38	Gamka	Gamkapoort Dam	D	17 076	I 3 699	10 - 50	3,55	1	$26-07^{00}$	206	12,1	69	12	17	net flood volume: $199 \times 10^6 m^3$
								Ø 3 111	-	-	1	$26-10^{00}$						
76	J2	33 - 29 3/4	21 - 37 3/4	Gamka	Bridge 4392: Ladismith-Calitzdorp rd	B	17 396	3 510	10 - 50	3,47	2	-	-	-	70	14	-	
77	J2M05	33 - 29 1/2	21 - 28 1/2	Huis	Zoar	SA	253	236°	± 10	3,52	2	-	-	-	175	> 200	-	
78	J2M06	33 - 29 1/2	21 - 29 1/2	Wilgehout	Opsoek	G	25	250°	≥ 50	4,54	2	$25-18^{30}$	3,7	14 8	175	> 200	84	
79	J2M07	33 - 29 1/2	31 - 30 3/4	Joubert	Opsoek	SA	25	185°	10 - 50	4,35	2	-	-	-	175	> 200	-	
80	J2	33 - 30 1/2	21 - 36 1/4	Huis	B4941: Calitzdorp-Ladismith rd	B	390	490°	10 - 50	3,88	U	-	-	-	170	> 200	-	
81	J2R01	33 - 29 1/4	21 - 42 1/4	Nels	Calitzdorp Dam	D	170	I 127	< 10	3,25	2	$25-20^{00}$	6,3	37	130	70	29	
								Ø 72	-	-	2	$25-24^{00}$						
82	J3M14	33 - 25 1/4	22 - 14 1/2	Grobbelaars	Kombuys	G	151	52	< 10	2,64	2	$25-15^{00}$	2,6	17,2	110	15	16	
83	J3M11	33 -.39 1/2	21 - 46 1/2	Olifants	Warm Water	SA	10 927	105	< 10	0	U	-	-	-	49	2	-	
84	J4M02	33 - 59	21 - 39	Gouritz	Mullershoop	SA	43 451	11 400°	≤ 200	4,22	2	-	-	-	91	30	-	flood peak is probably underestimated
85	E2M02	32 - 30 1/4	19 - 32 1/4	Doring	Aspoort	G	6 903	509	< 10	2,09	1	$26-17^{00}$	20,6	3,0	50	-	6	
86	E2	32 - 16 3/4	19 - 30 3/4	Doring	Elandsvlei	SA	14 334	660	< 10	1,73	1	-	-	-	50	-	-	

APPENDIX II : APPLICATION OF THE SLOPE-AREA METHOD

1. Required conditions in a good slope-area reach

- accessibility by road
- fairly straight, stable and clean main channel
- no deep pools, rapids, islands and sharp curves
- no bridges, weirs or ther obstructions within or immediately downstream of the reach
- simple cross-section is preferable to cross-sections consisting of main channel and flood plains (combined cross-section)
- flood marks are easily recognizable
- length of reach is at least six times the flood width
- number of cross-sections : Minimum three, preferably four to five.

2. Field survey

Consult references (7,8).

3. Hydraulic calculations

3.1 Chézy equation (apply at each cross-section)

In simple cross-sections : $Q_{xx} = CAR^{\frac{1}{2}}S^{\frac{1}{2}} = KS^{\frac{1}{2}}$ (m^3/s)

In combined cross-sections : $Q_x = K_1S^{\frac{1}{2}} + K_2S^{\frac{1}{2}} = S^{\frac{1}{2}}\ \Sigma K$ (m^3/s)

A	=	net area of cross-section	(m^2)
R	=	hydraulic radius = $\frac{A}{P}$	(m)
P	=	wet perimeter	(m)
S	=	longitudinal slope	(m/m)
C	=	Chézy's resistance factor	$(m^{\frac{1}{2}}/s)$
K	=	conveyance = $C\ A\ R^{\frac{1}{2}}$	(m^3/s)

Mean peak discharge in a reach wher N cross-sections were surveyed :

$$\overline{Q}_{(CH)} = \frac{\sum^{N} Q_i}{N}$$

Anomalous Q_i values should be excluded from the calculation of $\overline{Q}$.

Practical rules for the execution of the calculations :

1. Elements of cross-sections (Figure II.1)

Subsection : Is a division of a cross-section made according to cross-section geometry.

Subdivision : Is a division of a subsection made according to roughness.

Simple section (xx) : Is a cross-section consisting of only one subsection.

Combined section (x) : Is a cross-section consisting of more than one subsection (main channel and flood plains). The maximum number of subsections is usually three.

2. Reduction of cross-sectional area because of vegetation growth

Vegetation not only increases roughness, but it literally occupies a portion of the cross-section. In the calculations use always net cross-sectional areas i.e. :

$$A = r A_o$$

A = net cross-section
r = areal reduction factor : $0,5 \leq r \leq 1,0$
A_o = gross cross-section.

Recommended values for r :

	VEGETATION GROWTH	r
1	no growth, isolated trees or bushes, shrubs, low grass	1,0
2	thin bush and forest, tall grass, mealies	0,9
3	medium to dense bush, forest or plantation	0,75
4	very dense bush, forest (jungle)	0,5

Note : Interpolate according to local conditions.

3. Wet perimeter (P)

In main channels flanked by flood plains P is the sum of the channel bed perimeter proper and the water depths at the verticals which divide main channel and flood plains. (Figure II.2b)

4. Hydraulic radius (R)

It should always be calculated from net cross-sectional areas.

5. Chézy's flow resistance factor (C)

$$C = 18 \log \frac{6R}{\varepsilon}$$

ε = absolute roughness (m)

Characteristic ε values :

	CROSS-SECTION	ε
1	smooth, sandy simple natural channel	smaller of 0,15R or 0,20 m
2	natural channels and flood plains of fairly uniform cross-section and roughness	Ø of dominant obstacles
3	natural channels of non-uniform cross-section and roughness	1 to 2 x Ø -"- -"-
4	(dense) vegetation bent down by current : Cultivated plants, tall grass, bushes	0,25 to 0,5 times height, but always less than water depth
5	trees and bushes higher than water depth	$\varepsilon_{max} \sim$ water depth

Note : Interpolate between categories if necessary (for example in case of isolated obstacles, thin bushes).

C should be calculated for each subdivision. The results should be weighted according to respective subdivision areas in order to obtain a representative C value for the subsection (Figure II.2a and calculation forms of example).

6. Formulae of secondary hydraulic parameters

- energy coefficient α :

  simple section : $\alpha_{xx} = 1 + \frac{225}{C^2}$

  Combined section : $\alpha_x = \frac{\Sigma(\alpha_{xx} \; K_{xx}^{\;3}/A_{xx}^{\;2})}{K_x^{\;3}/\,A_x^{\;2}}$

- hydraulic depth : $h_m = \frac{A}{T}$ (m)

  T = top width (m)

- Froude number :

$$F = \frac{V\sqrt{\alpha}}{\sqrt{gh_m}}$$

if $F < 1$ the flow is subcritical

$F \sim 1$ the flow is critical

$F > 1$ the flow is supercritical

3.2 Slope-area equation (apply for each sub-reach between two successive cross-sections)

$$Q_{1,2} = K_2 \sqrt{\frac{Z_1 - Z_2}{\frac{K_2}{K_1} L + \frac{K_2^2}{2gA_2} k^* \left(\alpha_2 - \alpha_1 \left[\frac{A_2}{A_1} \right]^2 \right)}}$$

where index 1 refers to the more upstream section
index 2 refers to the more downstream section.

L = distance between the two sections (m)

k^* = eddy loss coefficient. It stands for energy loss in channel contractions and expansions.

Recommended k^* values

REACH	k^*
uniform	1,0
gradually contracting	0,9 to 1,0
gradually expanding	0,7 to 0,9
sudden contraction or expansion	0,5

In a reach where N cross-sections were surveyed the number of sub-reaches is N - 1 and the mean discharge is

$$\overline{Q}_{(SA)} = \frac{\sum^{N-1} Q_i}{N - 1}$$

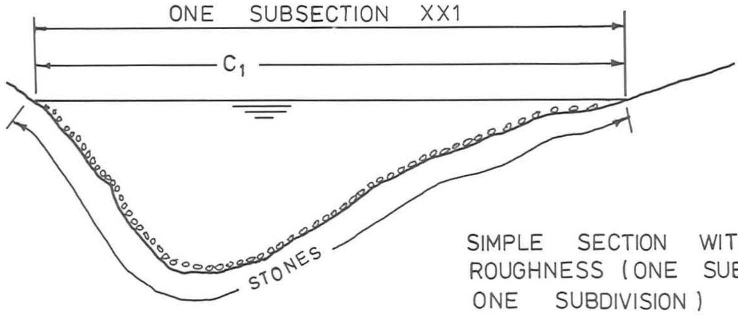
Anomalous Q_i values should be excluded from the calculation of $\overline{Q}$.

3.3 Representatative peak discharge in a slope-area reach

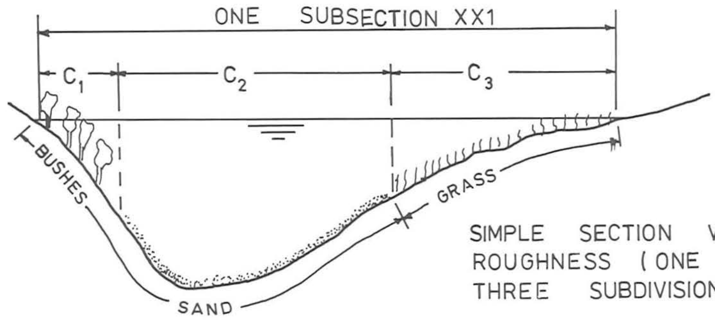
Consult Chapter 4.1.1, paragraph 6.

4. EXAMPLE

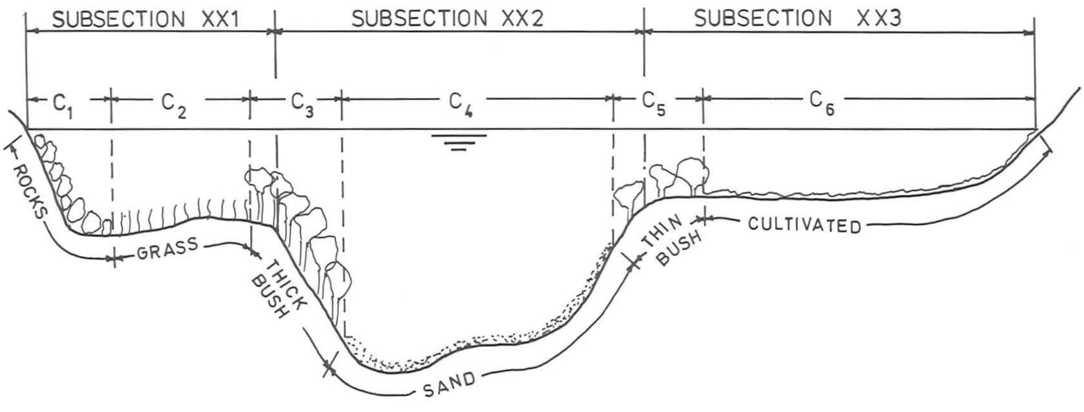
The survey carried out in the Joubert River (site 79) was chosen to illustrate the application of the slope-area method for simple cross-sections with variable roughness. For the sake of instructiveness the original data were in cases slightly modified. The text, figures, standard calculation forms and photographs A1-6 of Appendix II make the example self explanatory. Note that cross-section X2 is shown in photograph No. 17 of the report.



SIMPLE SECTION WITH UNIFORM ROUGHNESS (ONE SUBSECTION, ONE SUBDIVISION)

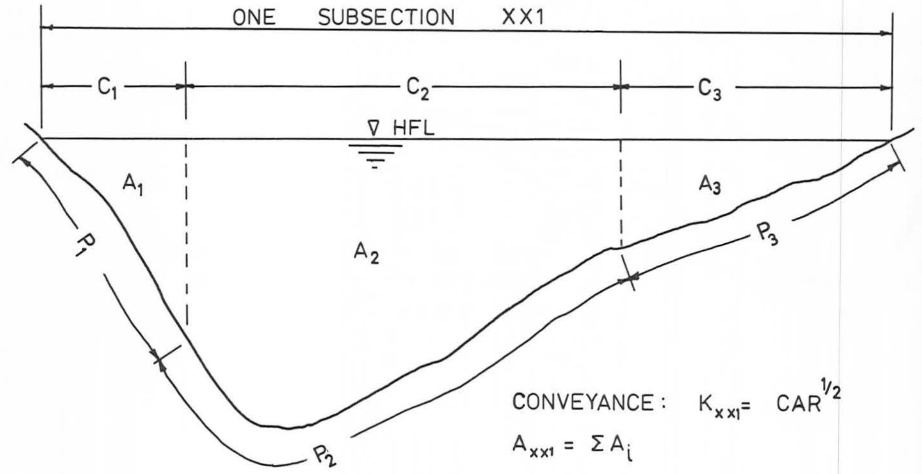


SIMPLE SECTION WITH NON - UNIFORM ROUGHNESS (ONE SUBSECTION, THREE SUBDIVISIONS)



COMBINED SECTION WITH NON - UNIFORM ROUGHNESS (THREE SUBSECTIONS, SIX SUBDIVISIONS)

FIG. II 1. ELEMENTS OF CROSS SECTIONS



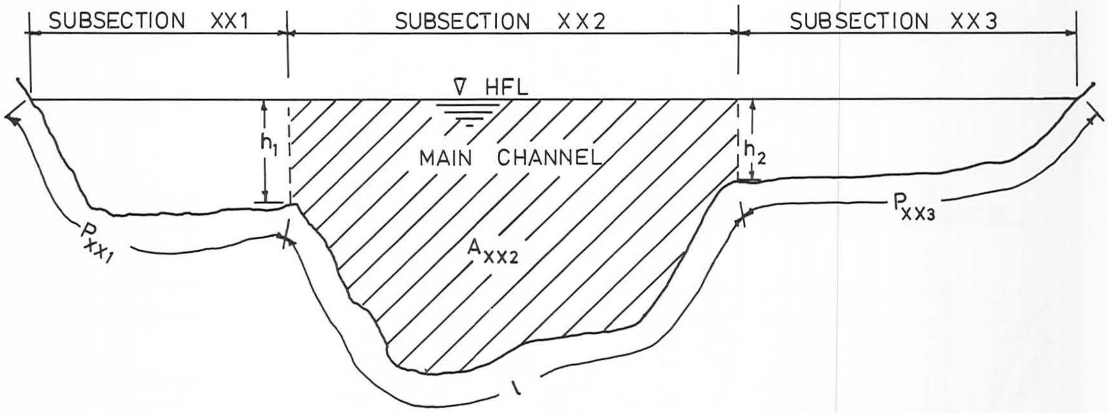
CONVEYANCE: $K_{xx1} = CAR^{1/2}$

$A_{xx1} = \Sigma A_i$

$R_{xx1} = \dfrac{\Sigma A_i}{\Sigma P_i}$

$C_{xx1} = C_1 \dfrac{A_1}{A_{xx1}} + C_2 \dfrac{A_2}{A_{xx1}} + C_3 \dfrac{A_3}{A_{xx1}}$

(a) IN SIMPLE SECTION



PERIMETER IN MAIN CHANNEL: $P_{XX2} = l + h_1 + h_2$

HYDRAULIC RADIUS IN MAIN CHANNEL: $R_{XX2} = \dfrac{A_{XX2}}{P_{XX2}}$

CONVEYANCE: $K = K_{XX1} + K_{XX2} + K_{XX3} = \Sigma K_{XX}$

(b) IN COMBINED SECTION

FIG II 2. CALCULATION OF CONVEYANCE

FLOOD OBSERVATIONS (1) SLOPE-AREA SURVEY

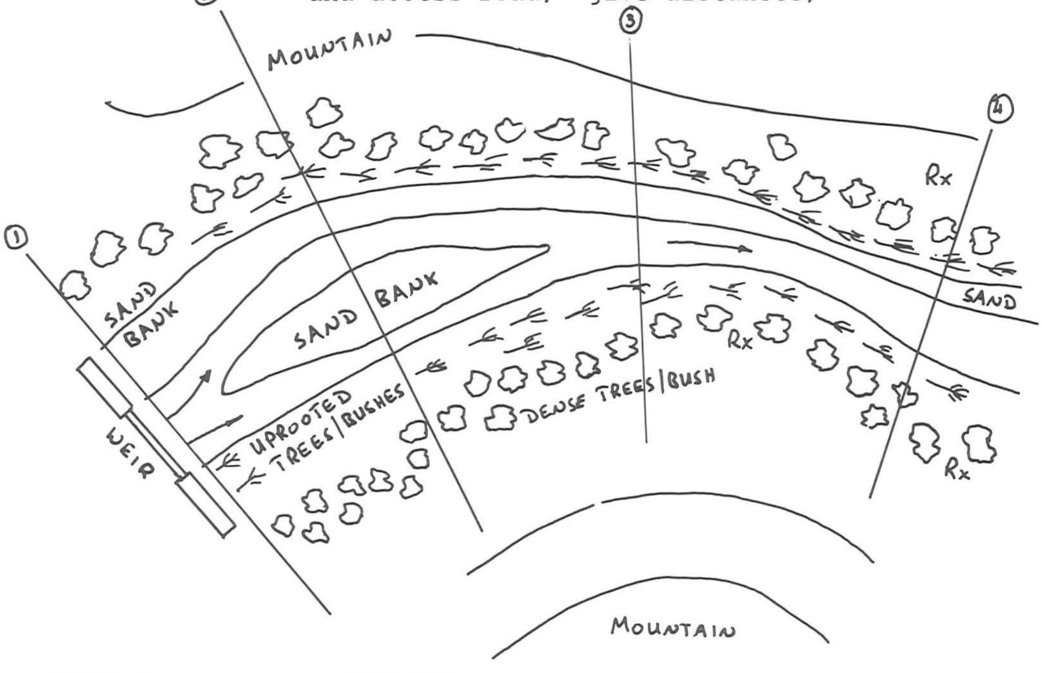
RIVER: JOUBERT
DRAINAGE REGION No.: 925
PLACE (name of farm): J2M07 (OPSOEK)
GEOGRAPHIC POSITION: LATITUDE: 33°29½' LONGITUDE: 21°30¾'
NEAREST TOWN: LADISMITH
DISTANCE TO TOWN: 25 km
DATE OF FLOOD PEAK: 26/1/81
DATE OF SURVEY: 26/2/81
SURVEY DONE BY: P. ELLIS

TOTAL LENGTH OF REACH: ....167. m
NUMBER OF CROSS SECTIONS: 4
AVERAGE TOP WIDTH OF CROSS SECTIONS: ...20,4.. m
NUMBER OF FLOOD MARKS: LEFT BANK: 8 RIGHT BANK: 8
BRIEF DESCRIPTION OF FLOOD MARKS (kind, quality):
KIND: FINE GRASS SEEDS + MUD PARTICLES ON PLANTS + ROCKS

QUALITY: FAIRLY GOOD

NUMBER OF PHOTOGRAPHS: 7
RIVER CHARACTERISTICS IN REACH (underline what applies):
(1) HORIZONTAL PLAN: straight, mild curve, sharp curve
(2) MAIN CHANNEL: material: rock, boulders, stones, sand, clay
condition: clean, some growth, heavy growth
(3) FLOOD PLAINS: bare, grass, cultivated, thin bush, dense bush
(4) OBSTACLES IN REACH (describe):
WEIR: 13m U/S OF SECTION 1

SKETCH OF REACH (show in plan floodmarks, cross-sections, obstacles and access road; give distances)



LIST OF ENCLOSED DOCUMENTS:

DO YOU WISH (a) assistance to complete survey?
(b) copy of flood peak calculations to be sent back to you?

IMPORTANT: GIVE ANY OTHER RELEVANT INFORMATION ON BACK-SIDE (as storm rainfall, damage etc.)

FIG. II 3 EXAMPLE. SITE OBSERVATIONS AND SKETCH

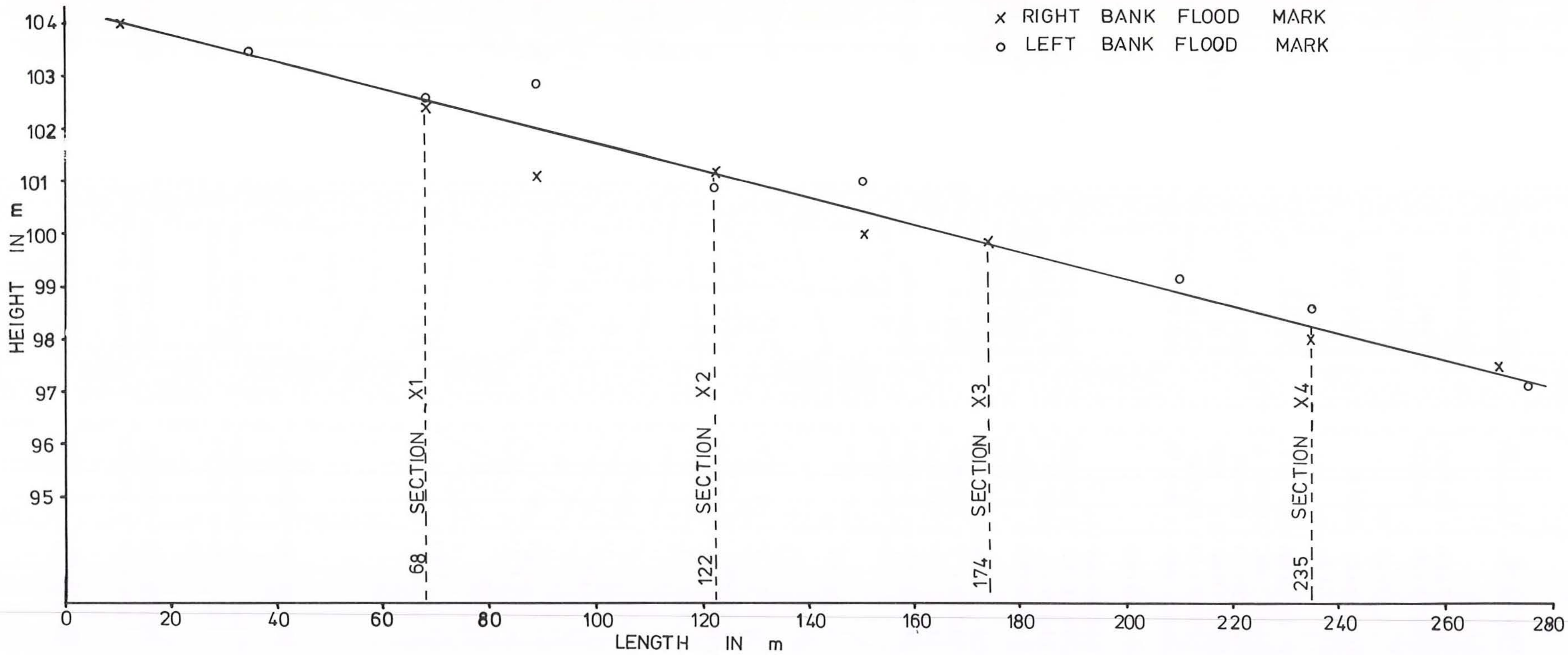


FIG. II 4. EXAMPLE. LONGITUDINAL FLOOD PROFILE

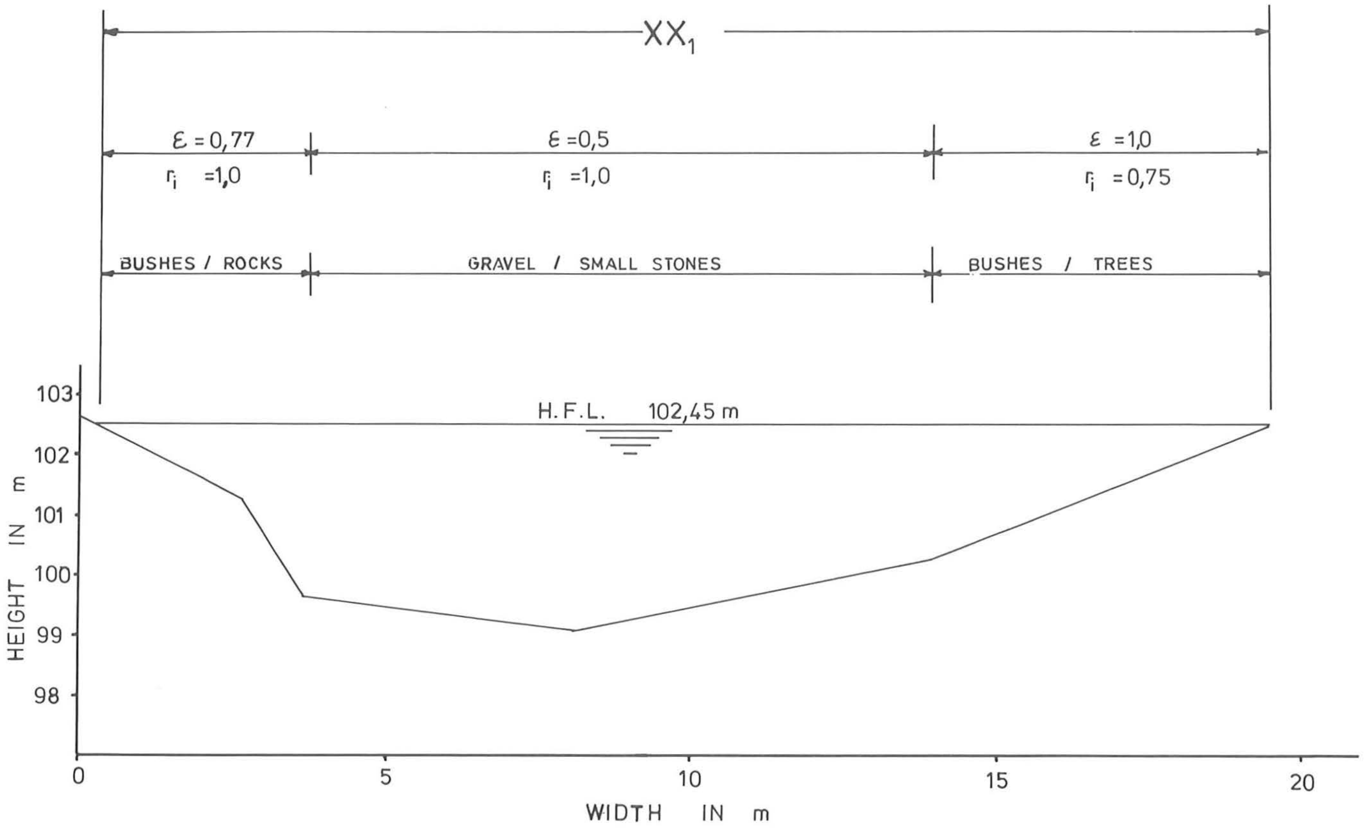


FIG. II 5. EXAMPLE. CROSS - SECTION X1

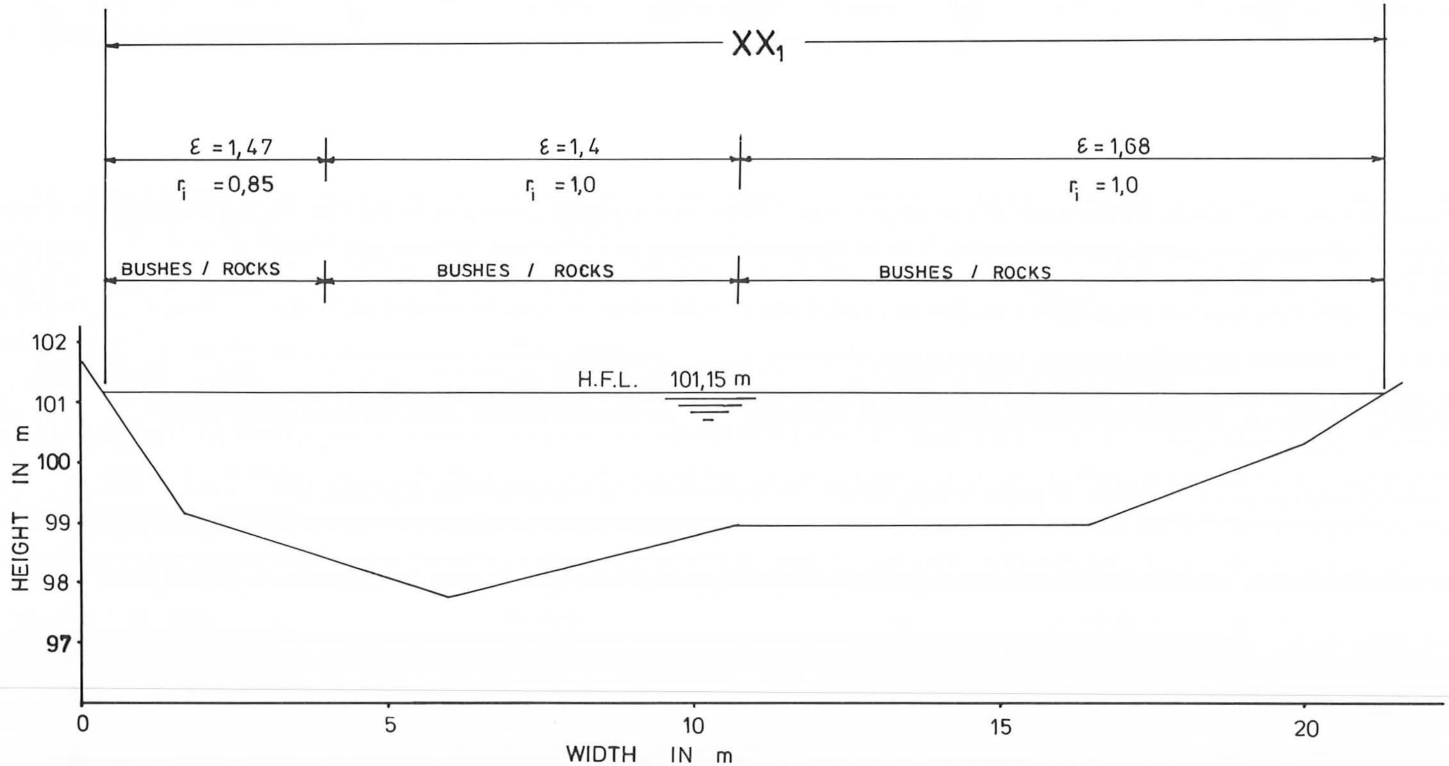


FIG. II 6. EXAMPLE. CROSS - SECTION X2

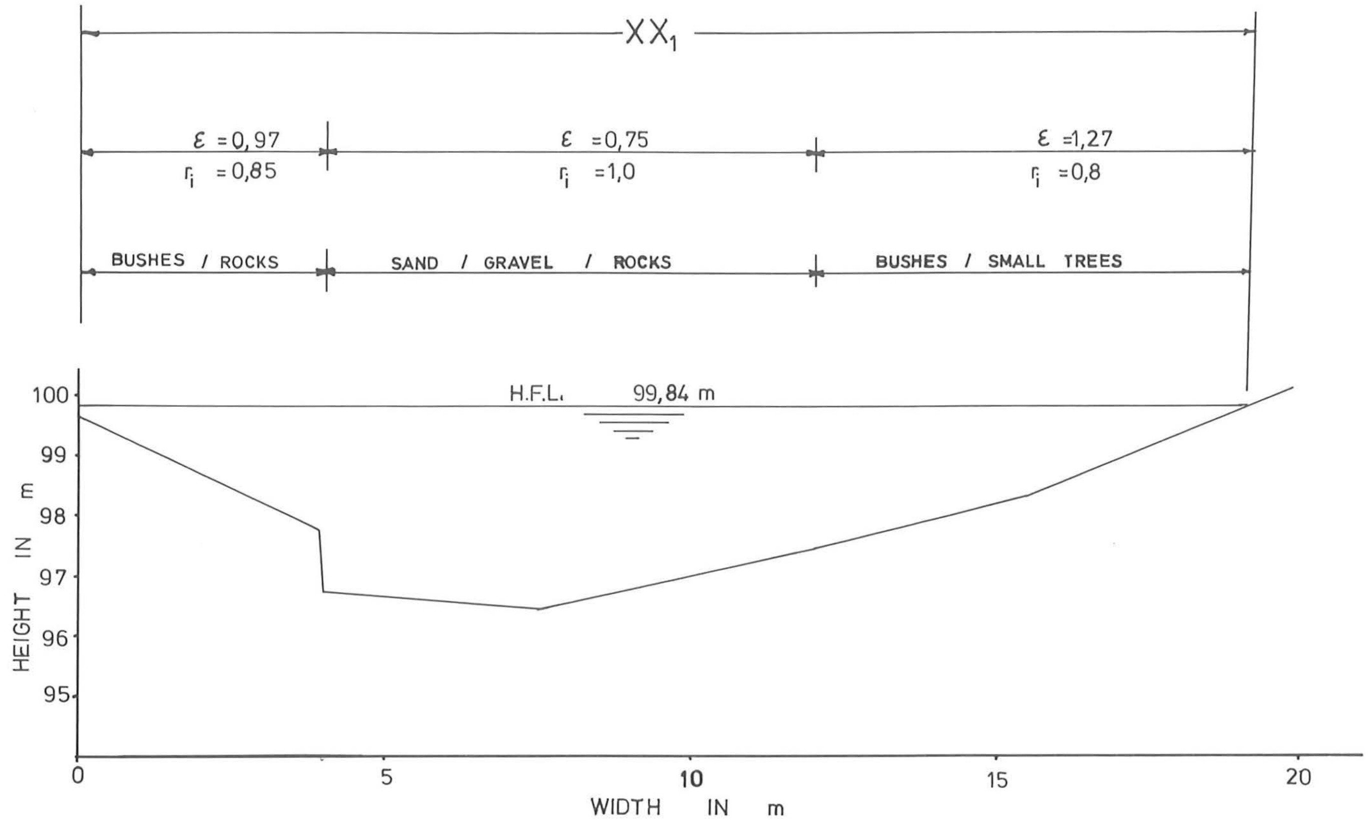


FIG. II 7. EXAMPLE. CROSS - SECTION X3

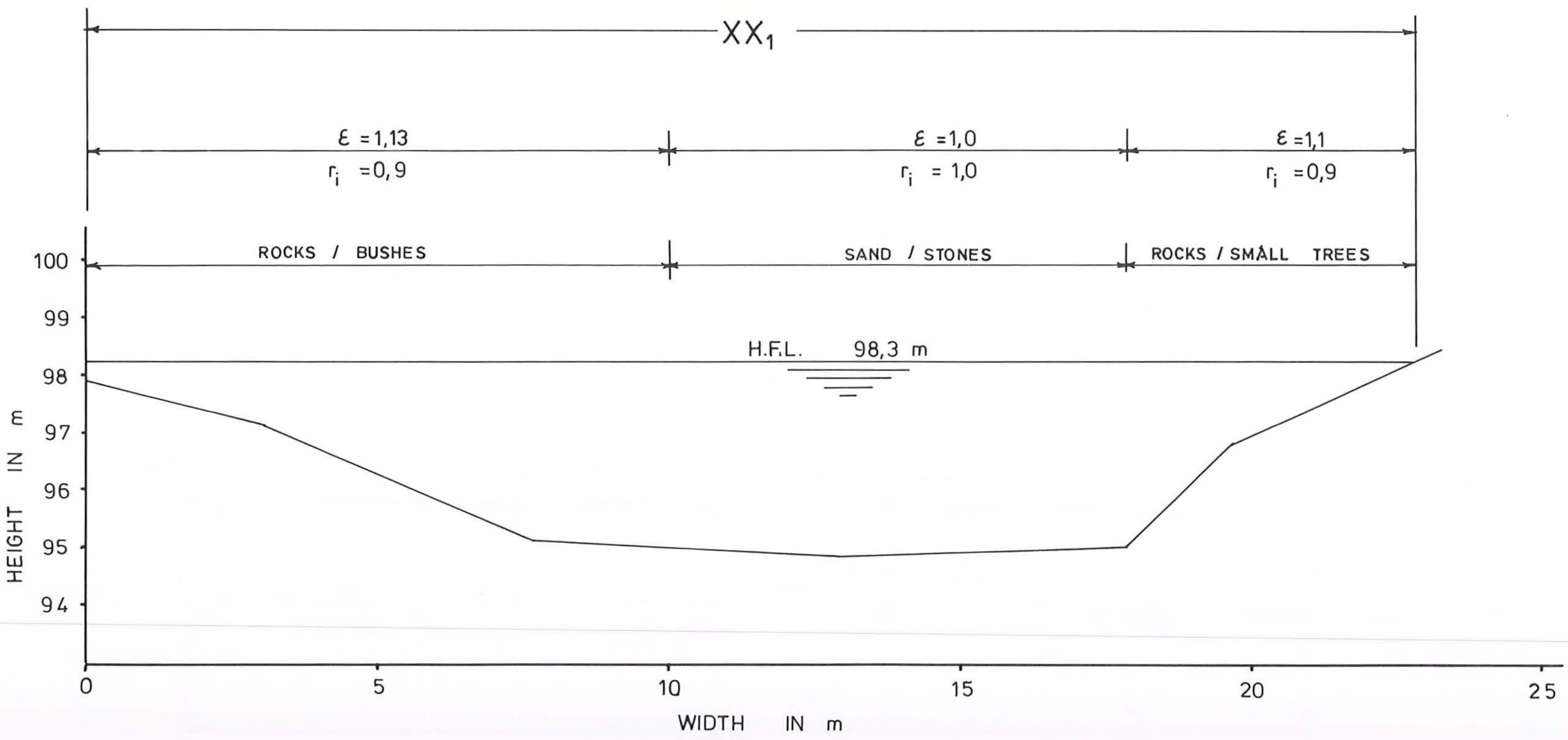


FIG. II 8. EXAMPLE. CROSS - SECTION X 4

LATITUDE: 33° 29½' LONGITUDE: 21° 30¾'	CATCHMENT AREA: (km^2) 25	RIVER: JOUBERT SITE: J2M07

DRAINAGE REGION: 925

CALCULATION OF PEAK DISCHARGE AT SITE

SECTION AND SUBREACH	Z m	ΔZ m	ΔL m	S	T m	A m^2	K m^3/s	α	k*	Q m^3/s CHÉZY	Q m^3/s SA EQUATION
1	102,45			0,025	19,1	38	1261	1,37		199	
1 - 2		1,30	54						0,9		173
2	101,15			0,025	21,2	44,4	990	1,87		157	
2 - 3		1,31	52						1,0		140
3	99,84			0,025	19,3	36,2	956	1,54		151	
3 - 4		1,54	61						0,9		231
4	98,30			0,025	22,8	48,6	1260	1,66		199	
4 - 5											
5											
		Σ ΔZ 4,15	Σ ΔL 167				MEAN PEAK DISCHARGE			177	181

REPRESENTATIVE PEAK DISCHARGE Q = 179 m^3/s

NOTES:

CALCULATED BY	D.B. DU PLESSIS	DATE: 4/8/81
CHECKED BY	W.G. v.d. WESTHUIZEN	DATE: 14/8/81

RIVER: JOUBERT	FL: 102,45	CHÉZY - EQUATION
SITE: J2M07	SLOPE: 0,025	SECTION NO.: 1

CALCULATION OF SUBSECTION PARAMETERS

SUB SECTION XX	DIV i	P_i	A_i	r_i	$A_i = r_i \cdot A_i$	$R_i = A_i/P_i$	ε_i	C_i	$m_i = A_i/\Sigma A_i$	$m_i \cdot C_i$	$K = C \cdot A \cdot R^{\frac{1}{2}}$
1	1	4,49	3,44	1	3,44	0,77	0,77	14,0	0,091	1,27	
	2	10,5	30,1	1	30,1	2,87	0,5	27,7	0,792	21,9	
	3	5,91	5,97	0,75	4,48	0,76	1,0	11,9	0,118	1,40	
	4										
	5										
		$\Sigma P_i =$ 20,9			$\Sigma A_i =$ 38,0	$\Sigma A_i/\Sigma P_i =$ 1,82			1,00	$C = \Sigma(m_i \cdot C_i)$ 24,6	1261
2	1										
	2										
	3										
	4										
	5										
		$\Sigma P_i =$			$\Sigma A_i =$	$\Sigma A_i/\Sigma P_i =$			1,00	$C = \Sigma(m_i \cdot C_i)$	
3	1										
	2										
	3										
	4										
	5										
		$\Sigma P_i =$			$\Sigma A_i =$	$\Sigma A_i/\Sigma P_i =$			1,00	$C = \Sigma(m_i \cdot C_i)$	
4	1										
	2										
	3										
	4										
	5										
		$\Sigma P_i =$			$\Sigma A_i =$	$\Sigma A_i/\Sigma P_i =$			1,00	$C = \Sigma(m_i \cdot C_i)$	

CALCULATION OF SECTION PARAMETERS

SUB-SECTION	PRIMARY						SECONDARY			
XX	A	R	C	K	Q	$v = Q/A$	α	T	h_m	F
SECTION	Σ 38			Σ 1261	Σ 199	$\Sigma Q/\Sigma A =$ 5,24	1,37	Σ 19,1	Σ 1,99	1,39

RIVER: JOUBERT	FL: 101,15	CHÉZY - EQUATION
SITE: J2M07	SLOPE: 0,025	SECTION NO.: 2

CALCULATION OF SUBSECTION PARAMETERS

SUB SECTION XX	DIV i	P_i	A_i	r_i	$A_i = r_i \cdot A_i$	$R = A_i/P_i$	ε_i	C_i	$m_i = A_i/\Sigma A_i$	$m_i \cdot C_i$	$K = C \cdot A \cdot R^{\frac{1}{2}}$
1	1	5,01	7,37	0,85	6,26	1,25	1,47	12,7	0,141	1,79	
	2	7,06	19,5	1	19,5	2,76	1,40	19,3	0,439	8,47	
	3	11,1	18,6	1	18,6	1,68	1,68	14,0	0,419	5,87	
	4										
	5										
		$\Sigma P_i =$ 23,2			$\Sigma A_i =$ 44,4	$\Sigma A_i/\Sigma P_i =$ 1,91			1,00	$C = \Sigma(m_i \cdot C_i)$ 16,13	990
2	1										
	2										
	3										
	4										
	5										
		$\Sigma P_i =$			$\Sigma A_i =$	$\Sigma A_i/\Sigma P_i =$			1,00	$C = \Sigma(m_i \cdot C_i)$	
3	1										
	2										
	3										
	4										
	5										
		$\Sigma P_i =$			$\Sigma A_i =$	$\Sigma A_i/\Sigma P_i =$			1,00	$C = \Sigma(m_i \cdot C_i)$	
4	1										
	2										
	3										
	4										
	5										
		$\Sigma P_i =$			$\Sigma A_i =$	$\Sigma A_i/\Sigma P_i =$			1,00	$C = \Sigma(m_i \cdot C_i)$	

CALCULATION OF SECTION PARAMETERS

SUB-SECTION	PRIMARY						SECONDARY			
XX	A	R	C	K	Q	$v = Q/A$	α	T	h_m	F
SECTION	Σ 44,4			Σ 990	Σ 157	$\Sigma Q/\Sigma A =$ 3,54	1,87	Σ 21,2	Σ 2,09	1,07

APPENDIX III : PRINCIPLES OF FLOOD PEAK CALCULATION AT BRIDGES

1. Required conditions at a good bridge site

- Fairly straight, stable and clean main channel.
- Road (railway) crossing at right angles to flow direction.
- At least 0,20 m drop in water level.
- Less than 20% reduction of bridge opening by floating trees and debris.
- Water level at downstream side is not influenced by obstacles further downstream.

2. Field survey

Consult references (7, 8, 12) and Chapter 3.2 of this report.

3. Flow types

Figure III.1 is a general sketch of a bridge contraction showing essential hydraulic terms. Note the following :-

Approach section (1) : Backwater $h^* = h_1 - h_N$ is maximum. This section should be situated as near as possible to the upstream side of bridge.

Contracted section (B) : In case of free-surface flow (Type Ia-b in Figure III.2) it is the minimum wet cross-section between abutments and is situated at the down= stream side of bridge. In case of orifice flow or pipe flow (Type II-IV in Figure III.2) it is situated at the upstream side of bridge.

Normal section (N) : A typical flood section in the reach where the disturbing effect of the bridge is not felt. It is situated approximately at distance 'b' downstream of the bridge.

Figure III.2 shows the four main flow types encountered at bridges during floods. These can be easily identified from a good field survey. The following table summarizes some of the essential flow characteristics of the four types.

TYPE	HYDRAULIC DESCRIPTION	STATE OF UNDISTURBED FLOW (1)	SPECIFIC ENERGY OF UNDISTURBED FLOW(1) $E_N^{(2)}$	DEPTH IN SECTION 'B'
Ia	free surface flow	subcritical, $F_N < 1$	greater than critical specific energy in section B : $\frac{E_N}{E_{CB}} > 1$	$h_B > h_{CB}$
Ib	"	subcritical or supercritical, $F_N \gtrless 1$	less than critical specific energy in section B : $\frac{E_N}{E_{CB}} < 1$	$h_B < h_{CB}$
II	orifice flow	upstream bridge soffit is in contact with water		$h_B = D$
III	pipe flow	upstream and downstream soffit is in contact with water		$h_B = D$
IV	pipe and weir flow	as III plus flow over road		$h_B = D$

Notes : (1) Refers only to types Ia, Ib.

(2) Specific energy $E = h + V^2/2g$ (another symbol for specific energy is H, used later in the broad crested weir formula).

4. Methods of calculation

Type I : Free surface flow

(1) U.S. Geological Survey formula

$$Q = 4{,}43\ C_a\ A_B \sqrt{\frac{\Delta h}{1 - \alpha_1\ C_a^2 \left(\frac{A_B}{A_1}\right)^2 + 2g\ C_a^2 \left(\frac{A_B}{K_B}\right)^2 \left(1 + L\frac{K_B}{K_1}\right)}}$$

C_a = discharge coefficient with a maximum value of 1,0. For details refer to (7, 11).

Suffixes '1' and 'B' refer to the approach and contracted sections.

Valid : Only for type Ia flow, thus when the flow is subcritical in section B. It is important to check the validity of above condition once Q has been calculated.

A quick way of checking is shown in paragraph (4).

(2) D'Aubuisson formula

$$Q = \mu_m A_B \sqrt{2g \Delta h + V_1^2}$$

μ_m = discharge coefficient depending on contraction ration 'm' (Figure III.1). Corresponding m and μ_m values :

m	0,1	0,2	0,3	0,4	0,5	0,6	0,9
μ_m	0,97	0,94	0,91	0,89	0,86	0,845	0,84

V_1 = mean velocity in section 1 = Q/A_1. Determine it by iteration starting with $V_1 = 0$. Number of iterations : 2 to 3.

Valid : For both Ia and Ib types. In case of Type Ia it is less accurate than method (1).

(3) Broad crested weir formula

$$Q = \mu_m \mu_s C_w b_o H_1^{3/2}$$

μ_s = discharge coefficient depending on submergence ratio h_B/h_1. From Figure 24 of reference (12) :

h_B/h_1	<0,85	0,88	0,90	0,92	0,94	0,95	0,96	0,97
μ_s	∿1,00	0,96	0,93	0,89	0,82	0,78	0,72	0,65

C_w = Weir coefficient ∿ 1,70

b_o = net width between abutments

H_1 = $h_1 + \alpha 1 \dfrac{V_1^2}{2g}$ = specific energy in approach section. Determine V_1 by iteration starting with $V_1 = 0$. Number of iterations : 2 to 3.

Constricted sections having uneven bottom should be subdivided into parts with approximately constant water depths.

Valid : For Type Ib flow, thus when the flow is critical or supercritical in section B. Check validity of above conditions as shown in following paragraph (4).

(4) Checking of state of flow in the contracted section (B) for the calculated peak discharge

General equation of critical flow condition :

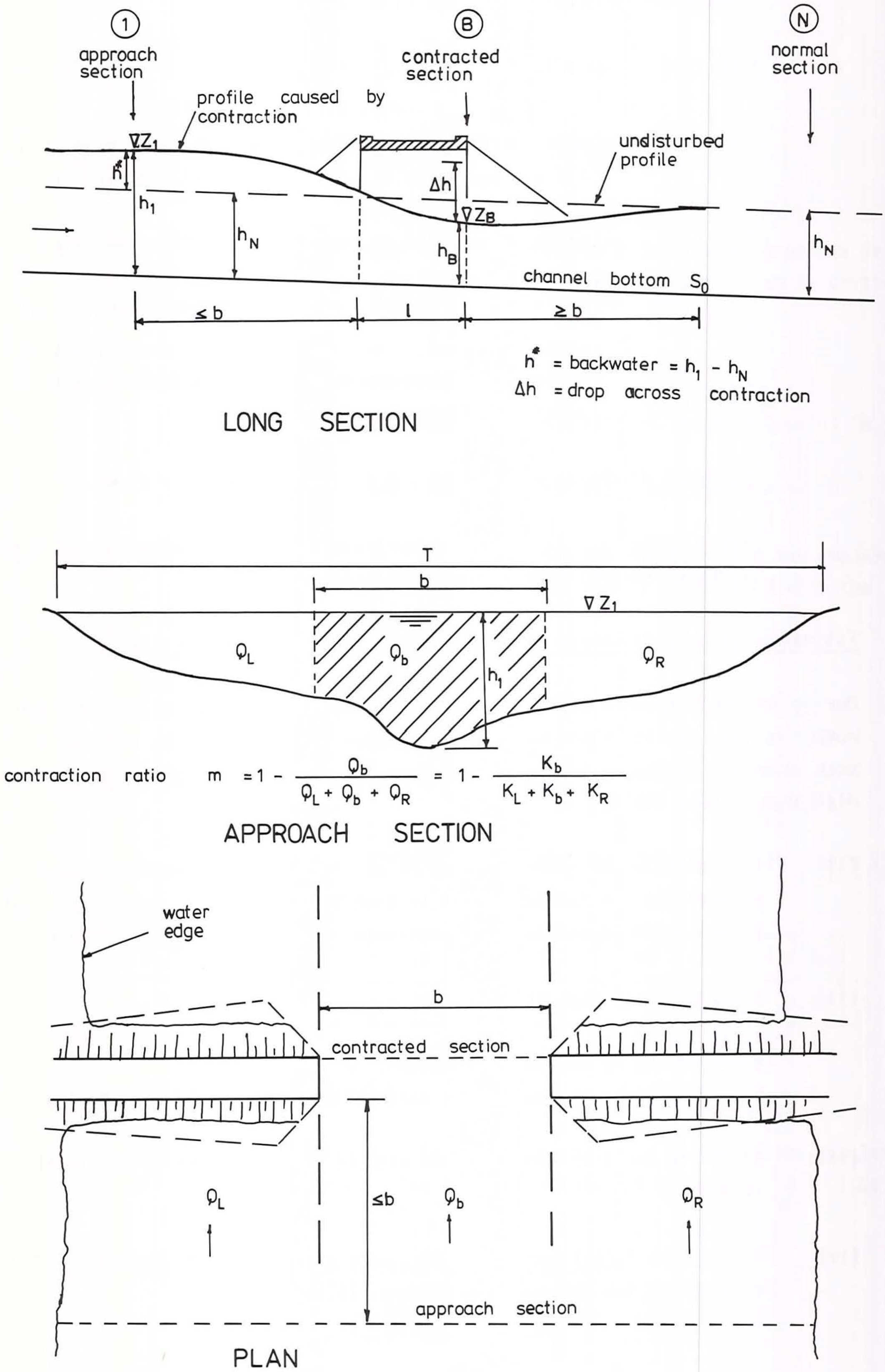
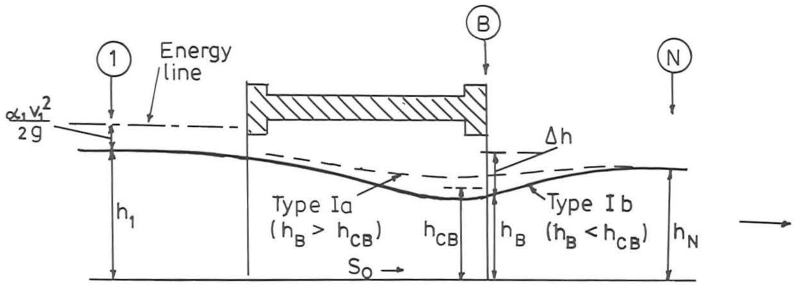
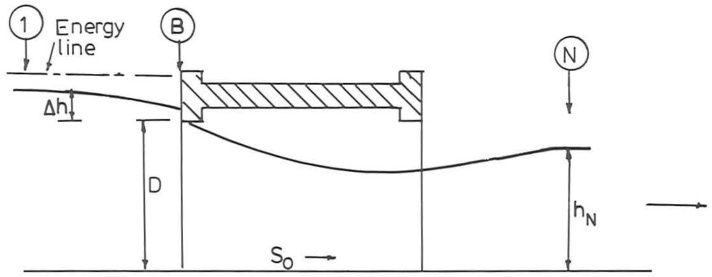


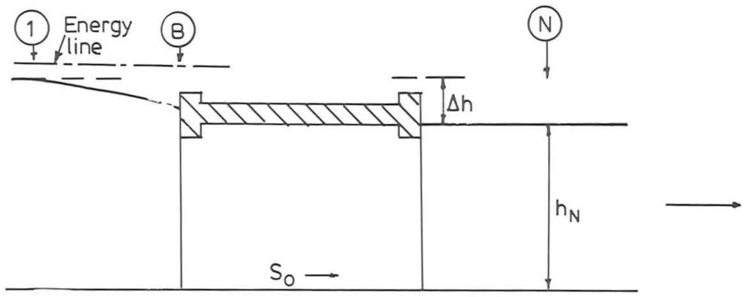
FIG. III 1. SKETCH OF A BRIDGE CONTRACTION



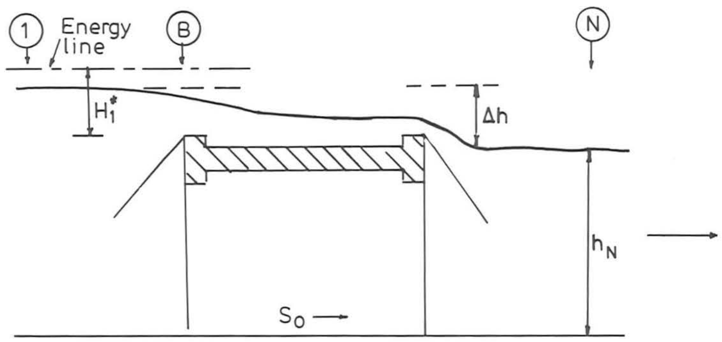
TYPE I FREE SURFACE FLOW



TYPE II. ORIFICE FLOW



TYPE III. PIPE FLOW



TYPE IV. PIPE AND WEIR FLOW
(over road + under bridge)

FIG. III 2. FLOW TYPES AT BRIDGES

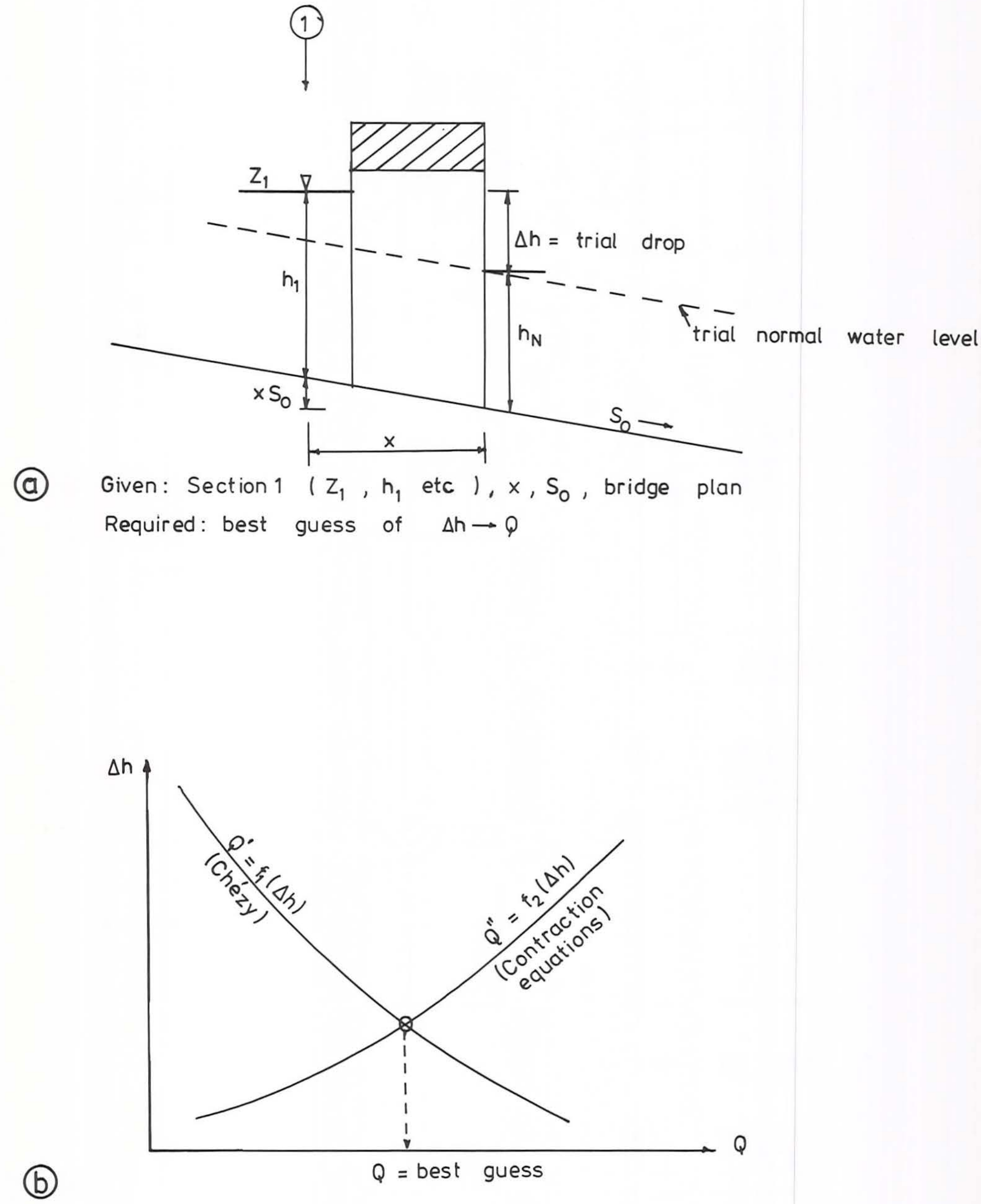


FIG. III 3. ESTIMATION OF Q AND Δh AT BRIDGES FROM UPSTREAM WATER LEVEL