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DEPARTMENT OF WATER AFFAIRS

Documentation of the January, 1978 floods in Pretoria and in the Crocodile River catchment

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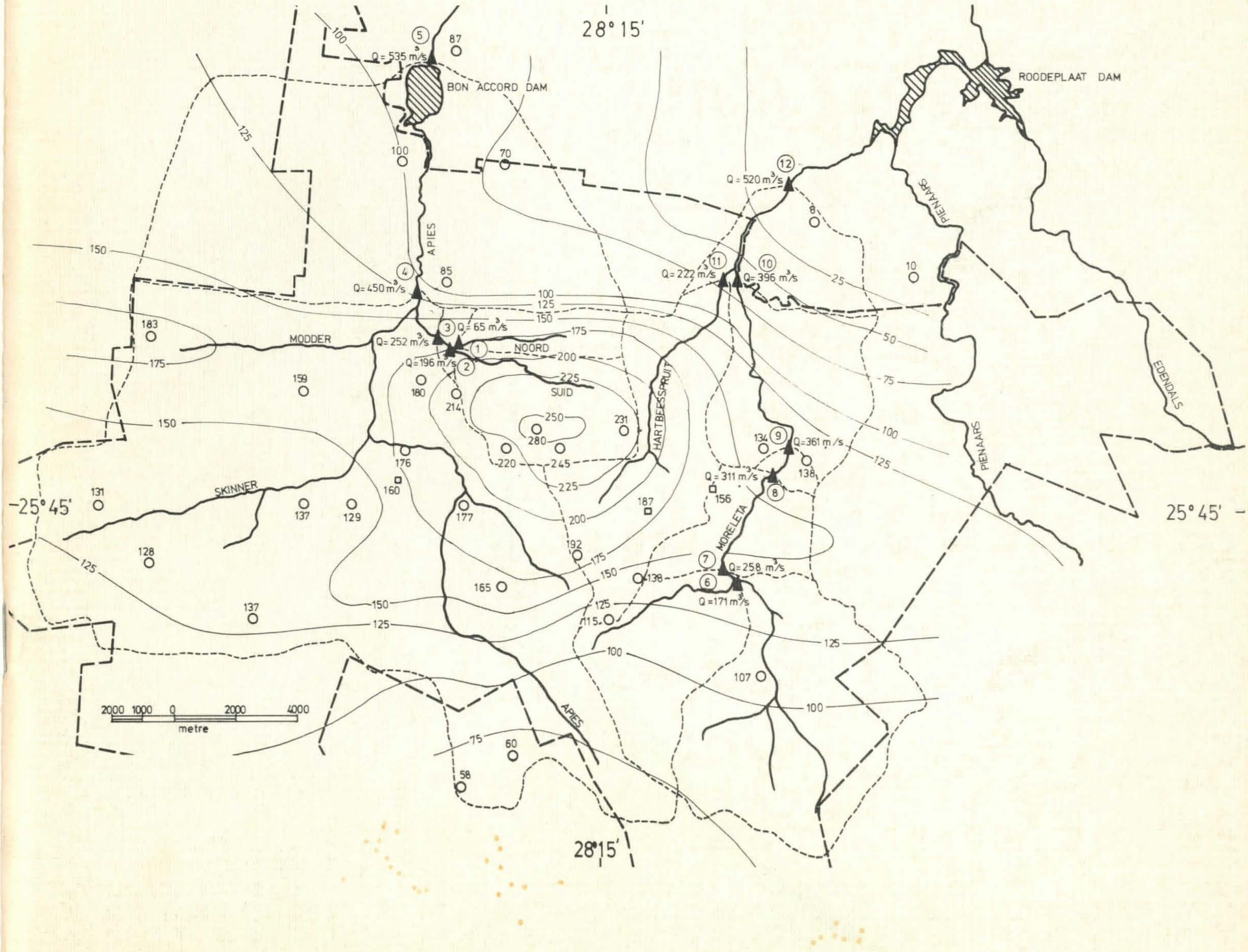

DEPARTMENT OF WATER AFFAIRS
Division of Hydrology

Technical Report No TR 88

DOCUMENTATION OF THE JANUARY, 1978 FLOODS IN PRETORIA AND IN THE CROCODILE RIVER CATCHMENT

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1. INTRODUCTION

The following report was requested by the Chief: Division of Hydrology in his note dated 1 February 1978.

During the early hours of 28 January 1978 Pretoria was hit by the heaviest storm in her recent history. Between midnight and 08h00 up to 280 mm rain fell. The Weather Bureau station in the Forum Building recorded 160 mm and this brought up the monthly total to nearly 500 mm, the second biggest in the century. The ensuing floods caused havoc in many areas, especially in Wonderboom, Pretoria North, Lynnwood, Silverton and Mamelodi. Eleven people died. Roads, bridges, the storm water drainage system, buildings, etc. were damaged, thousands of telephones and powerlines were cut. The material loss ran into hundreds of thousands of rands according to the most optimistic estimates.

In the Crocodile River catchment area floods were caused mainly by the same storm that covered the south eastern corner of the area. However, the contribution of fairly heavy rainfalls elsewhere and far above average wet antecedent conditions was significant as well. According to first estimates the damage to agriculture along the Crocodile, Pienaars and Apies Rivers amounted to several millions of rands.

The extraordinary event of 28 January 1978 in Pretoria will be remembered for a long time and will become a basis for comparison in the future both for laymen and professionals. The Division of Hydrology had a special interest in gathering relevant flood information. Firstly because of the absence of gauging stations in the urban area and also because of the relative scarcity of flood and storm-rainfall data in small catchments. Rainfall data at ± 40 stations and flood marks at 12 sites made it possible to draw an isohyetal map of the storm, calculate flood peaks, estimate the return periods of storm rainfall and flood peaks and obtain valuable information on runoff coefficients.

With regard to the Crocodile River catchment, where the floods varied from insignificant to very rare, the fairly abundant rainfall and flood information enabled the calculation of flood peaks, hydrographs, runoff volumes and percentages, and travel times of flood waves.

Both flood surveys proved to be an excellent opportunity for the comparison and testing of various methods of flood peak estimation.

The extensive field work, the collection of recorded rainfalls and hydrographs and the large volume of calculations were carried out by C.J. Botha, C.A. da Silva, C.J. de Jager, S. Mullineux and J. van der Westhuysen, members of the Flood Section.

Thanks are due to the Weather Bureau for providing the rainfall information, the Bridge Planning and Design Section of the Transvaal Province Roads Department for making available a great number of bridge plans and high water marks, and the City Engineer's Department of the Pretoria Municipality also for bridge plans. Without the co-operation of these departments the flood survey could not have been carried out with the necessary detail.

2. THE PRETORIA FLOOD

2.1 The storm

2.1.1 Meteorological cause (Fig. 1, Ref. (1))

As early as 19 January moist warm air was being fed from the north and east caused by a low in Botswana and a high above the south-eastern part of the country. The result was widespread showers in the affected areas. On the 26th a strong cold front moved in from the south-west. On the 27th the sharp trough of low pressure which had developed in the interior in a north-south direction became pinched between the high pressure systems over the eastern and western interior. The warm and cool air masses met on the night of the 27th in Transvaal causing exceptionally heavy rains in the Pretoria area.

2.1.2 Rainfall observations

The information included 29 official Weather Bureau stations and 11 private stations. Out of these three were autographic stations, the rest had daily rainfall totals. The graphs of the former showed that the storm lasted approximately from midnight till 07h00 on the 27th and there was no appreciable extra rainfall in the 24 hour period between 08h00 27th and 08h00 28th. Fig. 2 shows the accumulated storm rainfall at station No. 513/314A. Owing to above mentioned fortunate circumstance the daily figures were representative for the storm and could be used to construct the isohyetal maps shown in Fig. 3. It can be seen that the heaviest rainfall, say more than 200 mm, was restricted to a relatively small area of $\pm$ 30 km^2. The storm had an apparent ridge of maxima directed WNW - ESE. Noteworthy is the very rapid decrease of rainfall towards the north-east: only 11 km from the storm centre it was less than 10 mm.

2.1.3 Return period

(a) Point rainfalls

Within the area of the heaviest rainfall, station 513/404 (Bryntirion) had the longest record: 73 years. At the same time it was the Weather Bureau station with the highest rainfall on 28 January: 245 mm. Fig. 4(a) shows the observed and theoretical frequency distributions of annual maximum one-day rainfalls at above station. The observed data were ranked according to the Weibull formula as

$$T = \frac{N + 1}{m} \quad (\text{or } p\% = \frac{100}{T})$$

Where T = return period in years

p = probability of exceedance in %

N = length of record in years

m = rank in decending order

From the figure it is obvious that the recent storm was by far the biggest on record. The theoretical distributions fitted to the historical data were the Log Normal, Log Pearson III and General Extreme Value (GEV).

The estimation of a realistic return period for the event was made difficult by the clear upward swing of observed points in the low frequency range, from $T > 15$ ($p < 7\%$). In other words, the frequency distribution of the actual data is not homogeneous but composed of, at least, two distributions which presumably correspond to different storm-generating conditions. As in the range of $T > 20$ yr the GEV distribution seemed to provide the best fit the return period was estimated from that curve and was read off as T = 140 yr.

With the view of obtaining representative return periods for storm rainfall averaged over selected catchments the same analysis was performed for two more stations outside the storm centre. These were No. 513/255, 8 km WSW of the storm centre where the rainfall was 137 mm and No. 513/524, 7½ km E of the storm centre where 134 mm was measured, see Fig. 4(b), 4(c). The mean long-term MAP at the three selected rain gauges was 711 mm, practically the same as the average MAP calculated from more than 30 Pretoria stations. The character of the respective frequency distributions is similar to that at Bryntirion, but the absolute rainfalls are lower and are seemingly related to MAP. Again the GEV line gave the best fit. The respective return periods read off were 40 year and 30 year. From the series of annual maxima at some of other stations with more than 50 years of record it could be established that the 28 January rainfall was the biggest on record in the area enclosed approximately within the 175 mm isohyet.

(b) Mean rainfall

The return period of mean rainfall over the catchments selected for flood peak determination (see Fig. 3) was estimated as follows:

1. Mean rainfalls were computed from the isohyetal map.

2. Mean rainfalls were converted into point rainfalls by using Fig. C6 from Ref. (2).

3. Point rainfalls were projected on the GEV frequency distribution lines of the three selected rainfall stations and the corresponding return periods were read off. The representative return period was in each case taken as

$$T = \text{antilog} \left(\frac{\sum_{i=1}^{3} \log T_i}{3} \right)$$

The results were listed in col (9) to (12) of Table 1 and will be discussed in part 2.3.

It is probable that by calculating frequency distributions at a greater number of stations and introducing areal subdivisions the reliability of return period estimates could have been marginally improved. Unfortunately the extra work involved in such an exercise was prohibitive for the scope of this survey.

2.2 Flood peaks

2.2.1 Selection of sites

Flood peaks were calculated at 12 sites, see Fig. 3. Site 4 (Wonderboompoort) was the combination of a weir and a bridge, site 5 was the Bon Accord weir, the rest were bridges. In selecting sites the following aspects had to be considered:

(a) Inclusion of areas with the heaviest rainfall.

(b) Method of flood peak calculation. Preference was given to bridges because of

- lack of hydrographic stations

- conditions were generally not favourable for the application of slope-area methods in built-up areas. In addition the latter would have required more survey work.

(c) Available suitable bridges. Only those having construction plans were considered.

(d) Possibility for checks:

- up and downstream of confluence points: such were sites 1, 2, 3, 10, 11 and 12

- independent calculation: sites 4(a), 4(b).

Some of the relevant features of the catchments are listed in col. (5) to (8) of Table 1.

2.2.2 Field work

Field work was carried out within weeks after the flood so that flood marks could still be found with relative ease, although in cases some difficulty was experienced.

At bridges flood marks were sought and surveyed upstream and downstream of the contraction. A typical cross section of the water course in the vicinity of each structure was also surveyed. Photographs and sketches served for the estimation of roughness and for the visual reconstruction of the probable flow pattern during the peak. In this regard a realistic assumption for approach flow directions was in some instances of great importance.

At Wonderboompoort weir floodmarks were surveyed up and downstream of the crest and the main dimensions of the weir were measured.

At Bon Accord Dam the length of weir crest was measured and a sketch was drawn of the structure and the encroachment of hyacinths. The height of water above the crest was obtained from the secretary of the local Water Board.

2.2.3 Peak discharge

Bridges. The most reliable floodmarks were used to draw the HFL (high flood level). With the background of HFL and bridge section the likely flow types were established. These included one or, in case of doubt, more than one of the following:

(a) Free surface contracted flow not reaching bridge soffit or culvert box top (Fig. 5(a)).

(b) Orifice flow: entrance section of structure submerged (Fig. 5(a)).

(c) Pipe flow: entrance and outlet sections submerged (Fig. 5(b)).

(d) Combined pipe and weir flow: flow through bridge opening and over deck and approach embankments (Fig. 5(b)).

Symbols not shown on the drawings are the following:

Q	=	discharge
A_B	=	area of contracted wet section
C_a, C_b, C_f	=	discharge coefficients
h_f	=	friction loss between section (1) and (B)
α_1	=	velocity distribution coefficient
L	=	width of flow over road
g	=	gravity acceleration

The hydraulic background of the calculations is described in Ref. (3) and (4). In most cases the outcome of calculations was tested against that derived by the Chézy-Manning equation. The latter was applied in a typical section with deduced normal HFL. Normal slope was estimated from 1:50 000 maps. Manning's n values used at the particular sites are listed in Table 2. With a few exceptions the various methods agreed reasonably well.

Weirs. Both Wonderboompoort (site 4) and Bon Accord (site 5) were considered as submerged broad-crested weirs. Weir profiles as well as obstacles at the Bon Accord weir were taken into account.

The calculated peak discharges are listed in col (13) of Table 1. Discussion follows in part 2.3.

2.2.4 Return period

In absence of river gauging stations in Pretoria the return period of flood peaks had to be estimated by a rough approximative procedure. (Station A2M07 at Daspoort on the

Apies River was closed in 1951). For each catchment the 10 year and 100 year peak flows were calculated either by the Rational method only (in case of more than 50% urban area) or by the Rational and Synthetic Unitgraph methods. Standard sheets for the application of these methods are included in the Appendix. Results were plotted in Fig. 6 and connected with straight lines. The peak discharges obtained from the flood survey were then projected on the corresponding line and the return periods were read off, see col (15) of Table 1.

Admittedly this method could not make claim for accuracy and the return periods should therefore be viewed only as indicators of the expected order of years. This is especially true for the higher return periods. In spite of its inherent shortcomings the above method has helped to gain a realistic idea of the variation of return period along the drainage systems covered by the survey. Discussion follows in part 2.3.

2.3 Evaluation of results

2.3.1 Reliability of peak discharges (consult Table 1)

The peak discharges were calculated with care and sufficient attention was paid to factors by which the hydraulic condition at each site was determined or influenced. In such indirect flood peak calculations (i.e., when the discharge is not measured directly or not computed from velocity-area measurements) it is generally not possible to eliminate a number of inherent sources of errors. These included in the present case the following:

- inaccuracy of floodmarks, which depended largely on their quality

- lack of sufficient number of reliable floodmarks at places

- error in the estimation of roughness factor.

In the absence of any direct discharge measurement that could have served as a control it was not possible to estimate the error in the calculated peaks at individual sites with accuracy. Fortunately a few indirect controls seem to indicate that the error at most sites was moderate, most probably less than 10%. The indirect controls were the following:

(a) Noord and Suid Spruit confluence

$Q_{(3)} \cong Q_{(1)} + Q_{(2)}$ (i.e. from col (13) 252 $\cong$ 65 + 196)

(b) At Wonderboompoort sites 4(a) and 4(b) had practically the same peak discharge. The calculated peaks were 438 m^3/s and 462 m^3/s.

(c) Moreleta and Hartbees Spruit confluence

$$Q_{10} + Q_{11} = 396 + 222 = 618\ m^3/s$$
$$Q_{12} = 520\ m^3/s$$

Thus the sum of peaks above the confluence was about 20% more than downstream at site 12, in spite of 16 km^2 additional catchment area at the latter. Apart from possible errors in the above figures the reason for the discrepancy could well have been that

- peaks from the two tributaries most likely did not arrive at site 12 at the same time owing to the much shorter time of concentration for the Hartbees Spruit catchment, see col (8)

- there is large flood plain storage between the upstream and downstream sites.

(d) Moreleta Spruit. In Fig. 7 the peak discharge and mean storm rainfall have been plotted against respective catchment areas all along the river including site 12. Between sites 6 and 12 the mean rainfall showed only a slight increase (from 99 mm to 113 mm) while the peak discharge increased nearly linearly with the catchment area: $A_{(12)}/A_{(6)} = 3{,}19$ vs $Q_{(12)}/Q_{(6)} = 3{,}04$. As the slight increase in mean rainfall and urban area (mostly residential) was probably balanced by decreasing average surface slope, above "near-equality" is further proof that the peak estimations are reliable.

2.3.2 Reliability of return periods (consult Table 1)

The calculated return periods of the storm rainfall can be accepted as fairly reliable for they were obtained from statistical analysis of long records. On the other hand, the return periods of flood peaks were estimated by a rudimentary process. In comparing col (12) and (15) the disparity between corresponding return periods becomes striking. The next listed deliberations, however, may explain the matter, at least partially.

(a) Apies River catchments (sites 1 to 5). The return periods of rainfall, with the exception of Bon Accord Dam, were much higher than those of the flood peaks. This seems to be contrary to expectation because of the 186 mm antecedent rainfall recorded in Pretoria between 19 - 25 January that presumably saturated the catchments. One might argue nevertheless that in the relatively small urban catchments 1, 2 and 3, which have experienced the heaviest rainfall, the role of antecedent conditions was probably not decisive and, what is more important, the time of concentration of these catchments is much shorter (col (8)) than was the duration of storm i.e., the rainfall intensity was relatively low. To substantiate this point depth-duration-frequency data of autographic Weather Bureau station No. 513/405A, Ref (5) were contrasted with Fig. 2 and the following information was derived:

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Duration (hr)	Rainfall recorded (mm)	Return period (year)
1	max: 40	5
2	max: 76	100
24	164	500

It is seen that the return period of the rain falling during the critical duration which is per definition the time of concentration (approximately 2 hours for the catchments concerned) is much lower than that of the total rainfall. In the Apies River itself the relatively short return period of the flood peak was probably due to the lagged arrival of peaks from the tributaries. The reasonableness of the estimated T at Wonderboompoort was supported by records over the period 1905-1951 at Daspoort some distance upstream with a catchment area of 142 km^2. The LOG NORMAL frequency distribution computed for the annual maxima at this station (A2M07) indicated a peak discharge of 350 m^3/s for a 30 year return period. Applying an areal correction coefficient of $1.26\sqrt{(226/142)}$ the corresponding 1:30 year flood at Wonderboompoort would be 441 m^3/s which is very close to the January 1978 event. Because of urban development in the catchment since 1951 the true return period of the January peak was probably somewhat less than 30 years.

(b) Moreleta-Hartbees Spruit catchments (sites 6 to 12). Here the calculated T of flood peaks were consistently much higher than those of the rainfall. This was to be expected because of the saturated state of catchments at the time of the storm. These catchments are much less urbanized (with exception of No. 11) than those of the Apies River. Consequently the role of antecedent rainfall was presumably much more pronounced in producing high peaks, especially along the lower reaches that included large flat areas.

(c) General remarks

When assessing flood peak return periods it should be kept in mind that a relatively modest increase in discharge entails a much greater jump in return period. Figures below obtained from South African flood peak frequency analyses may illustrate the point:

Return period T (year)	Relative peak q in terms of qT=5	% Δ q in terms of previous q
5	1,00	
10	1,45	45
20	1,95	34
50	2,70	38
100	3,10	15
200	3,50	13
500	3,80	9

Thus a hundredfold increase in T corresponded to less than fourfold increase in q. What is more instructive, however, that while a 38% increase in discharge was needed to push up the return period from 20 years to 50 years, 9% will suffice to change it from 200 years to 500 years. By admitting an error of $\pm$ 10% in the calculated flood peaks it is clear that the return periods listed in col (15) of Table 1, especially the high ones, should be accepted with proper caution. For instance, the T = 330 years given for site 12 could have been anything, say, between 150 and 500 years.

It is important to emphasize that the return periods were estimated on basis of actual catchment conditions. Future urban development will inevitably lead to higher flood peaks which means that the return period of a given peak discharge will be lower.

2.3.3 Comparison of peak discharges calculated by three methods

In Table 3 flood peaks calculated from floodmarks and listed in col (15) of Table 1, herein called Q_o, were compared with those obtained by the Rational (Q_R) and Synthetic Unitgraph (Q_u) methods. These latter were applied for the actual storm by

- taking the storm duration as D = 6 hr

- considering the catchments, because of the already mentioned huge antecedent rainfall, as fully saturated.

(a) Rational method vs Q_o

In the Rational method the maximum values of the runoff-coefficient, C_{MAX}, were taken from the standard sheet, see Appendix. It is seen from col (10) that, excepting sites 4 and 5, the calculated Q_R agreed fairly well with Q_o, particularly in the more urbanized catchments No. 1, 2, 3 and 11. The probable reason for the great discrepancy in the two Apies River catchments is the already mentioned lagged arrival of peaks from the tributaries.

To complete the comparison runoff coefficients C_o corresponding to Q_o were listed in col (5). These figures were computed from the Rational formula as

$$C_o = 3.6 \frac{Q_o(m^3/s) \times 6\ (hr)}{h\ (mm) \times A\ (km^2)}$$

It is seen from col (5) that in the most urbanized catchments the "actual" values of the runoff coefficient were not far from unity.

The comparison thus leads to the conclusion that in comparatively small and mainly urbanized catchments the peak discharge could be estimated with reasonable accuracy from the data of a given storm if

(i) the catchment was previously saturated

(ii) C is taken unity

(iii) the storm duration was significantly longer than the time of concentration (in order to minimize possible lagged arrival of tributary peaks at the catchment outlet).

(b) Synthetic unigraph method vs Q_o

This method was applied only for those catchments where the urbanized area was less than 50%. The maximum values of the storm runoff factor were taken from Fig. G1 of Ref. (2). As for the Rational method the agreement was good, again with the exception of the two Apies River catchments, see col (11). Note that while the Rational method gave generally slightly lower values than the indirectly measured Q_o, the unitgraph method showed the opposite trend. The same conclusions are valid as for the Rational method with the storm runoff factor now equal to unity.

2.4 Resumé and recommendation

The large number of Weather Bureau stations and private rainfall gauges facilitated the drawing of a fairly accurate isohyetal map of the storm. The autographic stations were useful in determining the representative storm duration. It was fortunate that the storm rainfall was practically the same as the corresponding daily rainfall, because without this coincidence it would have been most difficult to draw a reliable isohyetal map. Statistical frequency analysis was carried out for the annual one-day rainfall maxima at three representative stations with more than 50 years of observation. This facilitated the determination of return periods for the storm over selected catchments.

Bridges proved to be most advantageous for the calculation of realistic flood peaks from floodmarks. Inaccuracy and uncertainty resulting mainly from poor quality floodmarks could be reduced by applying more than one approach in the hydraulic calculations. The relatively great number of bridges allowed the comparison of peaks and the detection of anomalies which consequently could be corrected.

The return periods had to be estimated in an indirect way by applying the Rational and Unitgraph methods. As compensation for the extra work valuable conclusions could be drawn regarding the use of runoff coefficients for these methods. Comparison with storm rainfall return periods proved to be a good indirect check and attested to the reliability of results. Furthermore it drew attention to the important role of antecedent catchment wetness in causing exceptional flood peaks when the storm rainfall itself has been less extreme such as happened in the Moreleta-Hartbees Spruit area.

The recent extraordinary floods in Pretoria have underlined the urgent need for flood flow measurement in urban areas. The most imperative reasons for gathering regular data in this aspect are:

- increased flood risk due to urban development
- increased flood damage risk and
- the already mentioned lack of flood information in small catchments.

Supported by the experience gained during the recent flood survey the following recommendations can be made:

(a) erect flow gauging stations in urban areas. These will provide checks for the less accurate indirect methods and can thus contribute to improve the latter. Permanent flow gauges are urgently needed also for the compilation of flow statistics without which the determination of return period will remain inaccurate or even unreliable;

(b) creation of autographic rain gauge network in urban areas where floods are caused most often by short duration intensive storms;

(c) selection of bridges, culverts and possible other structures that are suitable for flood peak determination. These will be necessary for a long time to come i.e., until a regular flow gauging network is operative. Floodmarks should be surveyed as soon as the flood has receded in order to minimize poor quality ones. The reconstruction of reliable HFL is a most important requisite for indirect flood peak determination.

3. THE CROCODILE RIVER CATCHMENT FLOODS

3.1 Rainfall

The whole of the Crocodile River catchment experienced a very wet January and the rainfall for the month amounted to more than twice the normal, Ref. (1). The biggest flood peaks were caused chiefly by the heavy storm over Pretoria and in the south-eastern corner of the catchment. However, copious rainfalls occurred at places both before and after the 28 January storm.

The review of daily rainfall data at more than 50 Weather Bureau stations revealed that the floods in the main collector could most conveniently be characterized by precipitation fallen in the period 24-30 January. The isohyets for that period are shown in Fig. 8. It is seen that apart from the main storm centre there were marked secondary centres north of the Hartbeespoort Dam and near the confluence of the Marico and Crocodile Rivers.

3.2 Floods

3.2.1 Observations

Observations were made at 22 sites, see Fig. 8 and col (1) to (5) of Table 4. The sites were concentrated along the Crocodile River itself and on the Apies and Pienaaars Rivers, and comprised 12 official river gauging stations, 5 dams, 2 slope-area reaches and 6 bridges. At sites 11, 17 and 19 the combined data of weirs and slope-area or bridge contraction methods were used to derive the flood hydrograph. Sites 11 and 16 were the same as sites 12 and 15 in the Pretoria flood survey.

At the river gauging stations the information was obtained from automatic water level recording charts. At dams from automatic and visual water level records, spillway capacity diagrams or gate operation schedules. At Bon Accord Dam only the maximum water depth over the weir crest was recorded. At the slope area stations the slope was deduced from floodmarks, four cross sections were surveyed and the roughness was estimated during site visit and from photographs. At bridges the high water level at the upstream side of the structure and the bridge plans were obtained from the Transvaal Roads Department. The roughness was estimated as in the slope-area reaches.

3.2.2 Flood peaks (col (6) to (8) of Table 4)

(a) Peak discharges were calculated as follows:

(i) at river gauging stations where the recorded level appeared reliable the peak discharge was derived from the discharge table (DT) either directly or by graphic extrapolation. In the latter case a possible error was brought into the estimate because due to lack of information, the extrapolation was of necessity arbitrary. Sites 1 to 6 and 15 belonged to this group;

(ii) at dams the inflow has been calculated from the recorded levels for sufficiently short time intervals as

$$I = \Delta S + O$$

where I = inflow

ΔS = change in dam storage

O = outflow

ΔS data was obtained from the dam capacity tables and O from the spillway capacity tables, but at Hartbeespoort Dam from the gate operation schedule.

At that dam there was no autographic recorder and only hourly observations were made.

At Roodeplaat Dam the peak inflow calculated in this manner led to the extravagantly high value of 2 670 m^3/s (compared with the maximum outflow of only 1 165 m^3/s.) The most probable explanation is that the water level in the dam was far from horizontal and in the relatively narrow and long dam it had a steep slope during the sudden rising stage of the flood wave and a mild one at the time of the maximum level at the dam spillway. With other words, storage component ΔS was in reality much smaller than the one computed from horizontal levels. The sum of peak discharges of the three main tributaries of the dam i.e., Hartbees Spruit (site 12 in Table 1 = site 11 in Table 4), Pienaars River (site 13 in Table 4) and Edenvale Spruit (site 12 in Table 4) was only 1 900 m^3/s (these tributaries comprise 94% of the catchment). Because of lagged arrival of the three flood waves into the dam, the resulting peak ought to have been less than 1 900 m^3/s. The application of the Rational formula for the actual storm rainfall of 88 mm falling during 6 hours indicated a possible maximum runoff coefficient of $C_{MAX} = 0,62$ (taken from the Division's standard calculation sheet). This was then reduced to $C = 0,54$ by using corresponding data at site 12, see col (5) and (7) in Table 3. The outcome was a peak inflow of 1 510 m^3/s;

(iii) slope-area method: it was used at sites 9 and 17. Detailed description of the method can be found in Ref. (6);

(iv) at bridges: at sites 11, 13 and 19 to 22 only one HFL was known. Consequently great care was taken to use in each case several methods of calculation, Ref. (2), (3), (6). The influence of river channel erosion and debris caught at the structure have been taken into account by correcting the throughflow area.

(b) Return periods were estimated only at those sites where the peak was presumably a rare event or where the work was facilitated by already available information. The estimated return periods appear in col (8). In col (15) the method used is indicated. The statistical analysis mentioned refers to annual maxima of recorded peaks and it has been carried out by the Division for other purposes. The role of "non statistical" methods in deducing return periods was the same as explained earlier in part 2.2.4.

3.2.3 Flood hydrographs

These were computed at 15 sites from water level records and were needed for the calculation of flood volume, runoff percentages and the times of peak and flood wave gravity centre (TGC).

Figs. 9(a)-(f) show flood hydrographs at two river gauging stations and four dams. At the latter both inflow and outflow hydrographs are shown except at Roodeplaat Dam where, as mentioned earlier, the inflow hydrograph could not be computed with sufficient accuracy. Flood volumes that in all likelihood resulted from rains between 24-30 January and from the main storm on 28 January were indicated on the hydrograph. The separation of flood flow and base flow in a flood hydrograph remains a controversial topic. There are several more or less arbitrary methods described in the technical literature. In this study a simple and straightforward technique was used which, nevertheless, could be seen as a satisfactory solution for the problem. 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. The base flow that became included in the above chosen period was automatically cancelled by the cutting off of the descending limb at the above indicated time.

At dams the flood volume is indicated on the outflow hydrograph.

At those river gauging stations where the hydrograph was obtained from extrapolated discharge tables a possible error was introduced in the calculated flood volumes.

Calculated flood volumes were listed in col (9) and (10) of Table 4.

Rainfall volumes in col (11) and (12) were obtained from the isohyetal map by planimetering and were used to calculate runoff percentages defined as:

$$\frac{\text{flood volume}}{\text{rainfall volume}} \times 100 \text{ for the 24-30 January rainfall.}$$

The runoff percentage was used as an indirect check of flood volume.

In calculating flood wave travel times the TGC (time of wave gravity centre) was preferred to the commonly used "time of peak". This latter is representative only in those cases when the flood wave has only one peak and it travels downstream without significant change in shape. In the Crocodile River catchment such conditions did not exist.

Col. (4) and (5) of Table 5 contain the time of peak and TGC. In col (6) the propagation of flood wave gravity centres is characterized by the respective distances, travel times and velocities between two sites.

3.3 Evaluation of results (Fig. 8, Tables 4 and 5)

In evaluating flood peaks, flood volumes, runoff percentagas and flood wave propagation one has been interested firstly in the reliability of the estimations and secondly in their practical value for future problems.

3.3.1 Reliability

(a) Flood peaks and volumes

The only direct controls were the inflow and outflow measured at dams.

Crocodile River upstream of Hartbeespoort Dam (site 7)

The sum of peak discharges at sites 2, 3 and 4 was much bigger than at site 5: 861 m^3/s vs 385 m^3/s. This can be attributed to the difference in the respective times of peak, col (4) in Table 5. On the other hand, the sum of flood volumes of the three tributaries was the same as the flood volume at site 5, which is evidence that the recorded peaks were reliable.

The sum of peak discharges at sites 5 and 6 was much less than at site 7 (Hartbeespoort Dam): 690 m^3/s vs 995 m^3/s. The most likely reason is that although sites 5 and 6 comprised more than 90% of the catchment of site 7 the remaining part, which included the dam itself and the catchments of Moganwe Spruit and Swart Spruit, experienced the heaviest rainfall, see Fig. 8 (area between Pretoria and Hartbeespoort Dam). The comparison of flood volumes and runoff percentages led to the same conclusion, see col (9), (10) and (13) in Table 4.

Crocodile River downstream of Hartbeespoort Dam

The peak of 758 m^3/s at site 8 (26 km downstream of site 7) was 67 m^3/s less than the maximum outflow from site 7. This could not be attributed to reduction by channel storage, because there was reasonably heavy rain in the area. As the flood volume was also less at site 8 the conclusion is that the peak determination was inaccurate, see note in col (15) in Table 4. The true peak discharge could have been 850 to 900 m^3/s. At site 9 the 1 180 m^3/s obtained by the slope-area method seems very realistic. At site 19 the peak of 1 520 m^3/s was obtained at a nearby bridge and should be a good estimate, because the peaks at sites 9 and 10 should have coincided while the peak outflow from site 18 arrived much later. The comparison of flood volumes and runoff percentages at the four sites also points to a realistic peak estimate.

As sites 20, 21 and 22 only peak discharges could be calculated. The rainfall pattern downstream of site 19 suggests, however, that the figures listed in col (7) of Table 4 are realistic. One must keep in mind that in the lower reaches of the Crocodile River the flood plain storage plays an increasing role in flattening flood peaks that arrive from upstream and to counter such effect the local rainfall has to be substantial. According to available information the present peaks in the lower Crocodile River were very similar to those that occurred in March 1976.

Pienaars River catchment

Two of the furthest upstream sites, namely No. 11 and 16, were already treated with the Pretoria flood and the respective peaks are fairly accurate.

At site 13 the peak of 1 340 m^3/s would seem to be far too high, but was nevertheless supported by HFL marks observed at the bridge of the National Road to Witbank some distance upstream. This latter has been designed for a 50 year flood peak of 765 m^3/s. On 28 January the HFL was nearly 3 m higher than the design flood level and the water flowed over the bridge-deck and adjoining road stretches along a total width of 580 m. A rough calculation indicated a minimum peak flow of 1 200 m^3/s.

As mentioned in part 3.2.2(a) the peak inflow of 1 510 m^3/s at site 14 (Roodeplaat Dam) should be a reasonable value. It is furthermore supported by comparison with the peak outflow of 1 165 m^3 at the same place and the order of peak absorption at three other dams.

At site 15, 32 km downstream of site 14, the peak of 1 105 m^3/s appears at first glance quite realistic. However, the comparison of respective flood volumes (col (9) and (10) in Table 4) shows more than 50% increase at site 15 in spite of insignificant rainfall over the intermediate catchment. The anomaly is also manifested in the jump of runoff percentage from 45 to 53 notwithstanding lower mean rainfall over catchment 15. Evidently, therefore, the DT of station A2M06, which has one of the longest uninterrupted flood peak records in the country, ought to be questioned.

The actual peak may have been about 700 m^3/s. This figure is supported by the fact that the bridge on the old Pretoria-Warmbaths road (situated some 30 km downstream) had been designed for 765 m^3/s but was not flooded on this occasion. (Information from Transvaal Roads Department).

The peak of 678 m^3/s at site 17 on the Apies River corroborates well with that of site 16 and this speaks for the fair reliability of the slope-area method when carried out properly.

Noteworthy is the drastic reduction of peak between sites 14 and 17 on the one hand and site 18 (Klipvoor Dam). There were obvious reasons for this: very short duration peaks in the upstream catchments, large very flat intermediate area where the rainfall was only moderate and possibly the lagged arrival of peaks into the dam. As noted earlier the peak outflow from site 18 arrived in the Crocodile River after the main peak had already passed.

(b) Return period of flood peaks

The most reliable return period estimates were obtained from statistical analysis of annual peak flows at sites 5, 6 and 15. For the rest T was estimated indirectly from the Rational, Unitgraph and Roberts methods as explained in part 2.2.4. Estimates taken from Ref. (7) were used at some of the dams in order to save time. There are two indications that the return periods derived indirectly are nevertheless realistic:

(i) the comparison of T for sites 5 and 6 (statistical method) and site 7 (indirect) is very satisfactory, see col (8) in Table 4;

(ii) the comparison for two dams of Ref. (7) data with those calculated by the Flood Section showed the following:

Site	Peak m^3/s	T (yr) obtained by	
		HRU (= Unitgraph)	Flood section (3 methods)
14	IN: 1 510	435[$]	500
	OUT: 1 165	130[$]	170
16	OUT: 535	50[$]	45

[$]Not included in Table 4

The agreement between the two calculations is good and this points to reliable estimates.

From col (8) in Table 4 the general picture is clear: the flood peaks were exceptionally rare in the upper Pienaars River catchment, rare along the Apies River and the Pienaars River between Roodeplaat Dam and Klipvoor Dam, moderate in the upper Crocodile River and quite common elsewhere, say $T \lesssim 5$ year.

(c) Runoff percentage

Figures in col (13) of Table 4 should be considered as approximate because of

- inaccuracy in flood volume at some of the river gauging stations caused by faulty discharge tables,

- possible inaccuracy in flood volume due to the definition of the same for the purposes of this study (see part 3.2.3),

- possible inaccuracy in rainfall volumes owing to the chosen uniform period of 24-30 January.

In spite of above negative factors the calculated runoff percentages were valuable, not only in helping to detect erroneous flood volumes (see part 3.3.1) but also for providing direct information which is unfortunately so scarce. It is particularly interesting to acknowledge the marked reducing influence of dolomitic areas on runoff, see figures for sites 2, 4 and 5.

At present the Synthetic Unitgraph method developed for South African conditions by the HRU is one of the most frequently used in the country for determination of design flood hydrographs. It is therefore instructive to compare storm runoff figures of the investigated flood with those obtained from Fig. G2 of Ref. (2). (Note that catchments with appreciable dolomitic areas were not included).

Site No.	River	Storm rain= fall (mm)	Veld type zone (from Ref. (2))	Storm runoff percentage	
				from flood survey	from Ref. (2)
3	Jukskei	103	4	31	38
6	Magalies	87	0,75x4+0,25x8	15	29
10	Elands	59	8	15	16
11	Hartbeesspruit	145	4	47	47
14	Pienaars	110	0,6x4+0,4x8	45	34
17	Apies	122	0,5x4+0,5x8	37	34
18	Pienaars	74	0,15x4+0,85x8	25	17
19	Crocodile	78	0,15x4+0,85x8	22	15

The comparison of the two methods reveals a very good agreement at some sites, but at other sites there are large differences. Without entering into speculation about the causes, the discrepancies are an indication that Fig. G2 of Ref. (2) often can not give the right answer because it is oversimplified. There are surely other factors than storm rainfall, veld type zone and catchment size that exert significant influence on the storm runoff percentage. Undoubtedly the most important among them is the antecedent catchment wetness which could be characterized either by antecedent rain or flow rate of the beginning of a storm.

(d) Flood wave propagation

The time of peak and TGC as shown in col (4) and (5) of Table 5 can be taken as correct as they are not affected by inaccuracies in flood peak and volume estimates.

On the other hand, flood wave travel times and velocities in col (6) of the same table should be interpreted with some care. The reason is that a water particle forming part of the flood peak at an upstream station will not necessarily do so somewhere downstream due to the transformation of the flood wave shape during its travel. This was even more so in the present case where the floods were generated in several parts of the catchment and arrived lagged at downstream confluences. A quick glance at Table 5 will suffice to prove this point: see the negative travel times between sites 4 and 7, 5 and 7, and sites 18 and 19. In spite of the complex nature of the phenomenon it is possible to state that:

- in the Crocodile River the velocity of flood wave propagation was 4 to 5 km/h upstream of site 7 (Hartbeespoort Dam), about 4 km/h between sites 7 and 19 and 2 km/h in the lower reach,

- in the Pienaars River the velocity between site 14 (Roodeplaat Dam) and site 15 was 4 km/h and between sites 15 and 18 (Klipvoordam) was only slightly more than 2 km/h.

It should be emphasized that above values were obtained from this particular flood and the velocity could be different during other floods, depending on the position, intensity and duration of the flood generating rainfall and on the antecedent flow in the channel system.

3.3.2 Practical value of the flood survey

Results of the flood survey could help in the solution of future flood analysis and operation problems in the catchment. Some of the benefits achieved are:

(a) The large amount of data obtained on flood peaks (levels, discharges and return periods), flood volumes, runoff percentages and flood wave propagation consitutes in itself a profitable reference.

(b) The indirect flood peak measurements performed at or near flow gauging stations have furnished a valuable calibrated point for the respective stage-discharge curves in the high flow range. It should now be possible to extend the discharge tables at three gauging stations with a lot more confidence. The stations in question are:

A2M25
A2M26
A2M28
A2M37

(Consult col (6) and (7) in Table 4)

(c) The comparison of flood volumes of neighbouring stations facilitated a fair estimation of flood peaks at stations A2M06 and A2M48. It proved at the same time that the DT of the former was erroneous in the experienced flood range by more than 50%.

(d) A reasonable idea has been obtained regarding flood wave propagation. Data compiled in Table 5 showed clearly that the velocity of flood wave decreased by at least 50% between the upper catchments, say those of Hartbeespoort Dam and Roodeplaat Dam and the lower Crocodile. It can be expected that for floods of approximately the same size and originated mainly in the upper catchments the average velocity of travel between above dams and the Limpopo River would be about 3 km/h.

3.4 Recommendations

(1) The slope-area and bridge contraction methods should be used whenever possible to estimate flood peaks. The accuracy and above all the reliability of these methods is much better than those of arbitrarily extrapolated discharge tables. By using the methods at several points along the same river the ensemble of results could form a effective check and amomalies of individual figures could then be corrected. In other words, the maximum error which could occur at a given site under unfavourable conditions (from experience $\pm$ 30% for the slope-area method and somewhat more for the bridge contraction method) could be greatly reduced, say to less than 20%. The only precondition for the use of these methods is that they should be applied at suitable places as soon as the flood has receded and carried out with sufficient care. Guidelines for the correct application of the slope-area method have been set out in Ref. (6). A similar guide referring to the bridge contraction method will be issued by the Division of Hydrology in the near future.

(2) Preparation of pilot plans for important catchments. These should contain properly selected sites for the application of indirect methods. At each site the necessary cross-sections, normal slope, structural dimensions, roughness etc. should be compiled. All that would then be needed in case of a flood is to survey floodmarks. Flood peaks could then be calculated with minimal effort. At present such a pilot plan is being compiled by the Flood Section for the Crocodile River catchment.

(3) At present the great majority of river gauging stations are not yet calibrated above the top level of the weir. In flood terms it means that the reliable weir measuring capacity is not higher than the mean annual flood peak (equivalent to a return period of $\pm$ 2½ years) and very often is even less. This is a very serious shortcoming and the consequences are far reaching. In South Africa, namely, due to the rather extreme hydrological regime of most rivers, the flood volume can constitute a substantial part of the annual runoff which, in turn, is a basic data in water resource projects. Errors in the former will thus inevitably affect the annual runoff. It should therefore be a most urgent task to calibrate the river gauging stations in flood flow range by all possible means. It is here where the indirect methods are of great practical value.

(4) Comprehensive flood surveys should be carried out in future for each important flood. The diverse information (rainfall, flood peaks, hydrographs etc.) documented together will permit to extract more reliable and meaningful conclusions than derived by considering only isolated flood peaks.

Though it did not fall within the scope of this survey, the estimation of flood damage should form part of future surveys. After all it is flood damage information that could justify the implementation of proper and economically viable flood defence measures. This task unfortunately can not be tackled by the Division of Hydrology but would require a co-ordinated action by all the interested parties.

LIST OF REFERENCES

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3. World Meteorological Organization: Measurement of peak discharge by indirect methods, Technical Note No. 90, Geneva, 1968.

4. U.S. Bureau of Public Roads: Hydraulic of bridge waterways, Hydraulic design series No. 1, Washington, D.C., 1970.

5. Department of Water Affairs, Division of Hydrology: Extreme values and return periods for rainfall in South Africa, Technical Note No. 78, 1977.

6. Department of Water Affairs, Division of Hydrology: An investigation into the methods of peak flood discharge estimation from slope-area data, Technical Note No. 76, 1976.

7. Hydrological Research Unit, University of the Witwatersrand: Amendments to Design flood manual HRU 4/69, Report No. 1/71.

APPENDIX

Calculation sheets for the Rational and Unitgraph methods

TABLE 1

RAINFALL AND FLOOD-PEAKS IN 12 CATCHMENTS ON 28 JAN. '78 IN PRETORIA

SITE No	RIVER	PLACE	FLOOD MARKS AT:	CATCHMENT				STORM RAINFALL				FLOOD PEAK			
				AREA		AVERAGE SURFACE SLOPE	TIME OF CONCENT-RATION	MEAN	AREAL FACTOR	POINT	RETURN PERIOD	DISCHARGE		RETURN PERIOD	
				A [km^2]	URBAN %	%	t_c [hr]	$\bar{h}$ [mm]	a	$h=a\bar{h}$ [mm]	T [yr]	Q [m^3/s]	Q/km^2	T [yr]	METHOD
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)
1	NOORD SPRUIT	VOORTREKKER ROAD	MUNICIPAL BRIDGE	7,9	80	6	1,6	169	1,00	169	65	65	8,23	30	RATIONAL
2	SUID SPRUIT	LOUIS TRICHARDT RD	MUNICIPAL BRIDGE	19,6	90	3	1,7	226	1,02	231	295	196	10,00	85	RATIONAL
3	NOORD + SUID SPRUIT	5th AVENUE WONDERBOOM S	MUNICIPAL BRIDGE	29,2	85	5 1/2	2,1	206	1,03	212	205	252	8,63	145	RATIONAL
4a	APIES	WONDERBOOM-POORT	WEIR	226	45	7 1/2	4,4	151	1,14	172	70	438	1,94	30	RATIONAL UNITGRAPH
4b	APIES	WONDERBOOM-POORT	T.P. BRIDGE No 1223	227	45	7 1/2	4,5	151	1,14	172	70	462	2,04	35	RATIONAL UNITGRAPH
5	APIES	BON ACCORD DAM	WEIR	315	33	3	5,6	141	1,14	160	50	535	1,70	45	RATIONAL UNITGRAPH
6	MORELETA SPRUIT	MILITARY ROAD	TP BRIDGE No 751	48,9	5	11	2,2	99	1,02	101	12	171	3,50	25	RATIONAL UNITGRAPH
7	MORELETA SPRUIT	LYNNWOOD ROAD	TP BRIDGE No 2022	68,7	15	10 1/2	2,3	101	1,02	103	12	258	3,76	35	RATIONAL UNITGRAPH
8	MORELETA SPRUIT	WATERMEYER ROAD	MUNICIPAL BRIDGE	81,3	20	10 1/2	3,0	110	1,04	115	15	311	3,83	70	RATIONAL UNITGRAPH
9	MORELETA SPRUIT	PRETORIA RD SILVERTON	TP BRIDGE No 1342	83,1	20	10 1/2	3,2	111	1,04	116	16	361	4,34	110	RATIONAL UNITGRAPH
10	MORELETA SPRUIT	WONDERBOOM CULLINAN RD	TP BRIDGE No 3205	107	25	5 1/2	4,8	107	1,05	113	15	396	3,70	200	RATIONAL UNITGRAPH
11	HARTBEES SPRUIT	WONDERBOOM CULLINAN RD	TP BRIDGE No 3204	32,9	55	6	2,9	167	1,03	172	70	222	6,75	255	2/3 RATIONAL + 1/3 UNITGRAPH
12	HARTBEES SPRUIT	KAMEELDRIF FARM	TP BRIDGE No 2315	156	30	6	5,6	113	1,06	120	18	520	3,33	330	RATIONAL UNITGRAPH

TABLE 2

SITE No	BRIDGE No.	MANNING'S n FACTORS (LOOKING DOWNSTREAM) LEFT BANK	MAIN CHANNEL	RIGHT BANK
MANNING'S n VALUES USED IN THE CALCULATION OF FLOOD PEAKS FOR THE STORM OF 28 JANUARY IN PRETORIA				
1	BRIDGE IN VOORTREKKER ROAD	0,050	0,045	0,050
2	BRIDGE IN LOUIS TRICHARDT STR.	0,050	0,045	0,050
3	BRIDGE IN FIFTH AVENUE	0,050	0,045	0,050
4	BRIDGE No. 1223	0,065	0,040	0,065
6	BRIDGE No. 751	0,120	0,120	0,120
7	BRIDGE No. 2022	0,050	0,075	0,060
8	BRIDGE IN WATERMEYER STR.	0,050	0,045	0,100
9	BRIDGE No. 1342	0,055	0,040	0,055
10	BRIDGE No. 3205	0,090	0,060	0,090
11	BRIDGE No. 3204	0,060 AND 0,050	0,050	0,070
12	BRIDGE No. 2315	0,080	0,060	0,080

TABLE 3

COMPARISON OF FLOOD PEAKS CALCULATED BY THREE METHODS FOR THE FLOOD OF 28 JANUARY 1978 (PRETORIA)

SITE No	RIVER	CATCMENT AREA	FROM FLOOD MARKS		FROM RATIONAL* METHOD		FROM UNITGRAPH* METH			
			PEAK DISCARGE	RUNOFF COEFFICIENT	PEAK DISCARGE	RUNOFF COEFFICIENT	PEAK DISCARGE	STORM RUNOFF FACTOR		
		A [km²]	Qo [m³/s]	Co	Q_R [m³/s]	$C_{MAX.}$	Qu [m³/s]		$\frac{Q_R}{Q_o}$	$\frac{Q_u}{Q_o}$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	NOORD SPRUIT	7,9	65	1,05	57	0,92	—	—	0,88	—
2	SUID SPRUIT	19,6	196	0,95	197	0,96	—	—	1,01	—
3	NOORD+SUID SPRUIT	29,2	252	0,90	262	0,94	—	—	1,04	—
4	APIES	227	450	0,28	1086	0,79	905	0,95	2,41	2,01
5	APIES	315	535	0,26	1287	0,69	1419	0,93	2,41	2,65
6	MORELETA SPRUIT	48,9	171	0,76	148	0,66	212	1,00	0,87	1,24
7	MORELETA SPRUIT	68,7	258	0,80	222	0,69	300	1,00	0,86	1,16
8	MORELETA SPRUIT	81,3	311	0,75	294	0,71	366	1,00	0,95	1,18
9	MORELETA SPRUIT	83,1	361	0,84	300	0,70	373	1,00	0,83	1,03
10	MORELETA SPRUIT	107	396	0,75	376	0,71	400	1,00	0,95	1,01
11	HARTBEES SPRUIT	32,9	222	0,87	211	0,83	—	—	0,95	—
12	HARTBEES SPRUIT	156	520	0,64	595	0,73	549	0,98	1,14	1,06

NOTE: * CALCULATED WITH D = 6 hr. STORM DURATION

TABLE 4

RESUMÉ OF FLOOD SURVEY IN THE CROCODILE RIVER CATCHMENT FOR THE FLOODS OF JANUARY 1978

SITE No (1)	RIVER (2)	HYDROGRAPHIC STATION No OR BRIDGE (3)	GEOGRAPHIC POSITION (4) LAT.	LONG.	CATCHMENT AREA A [km²] (5)	MAXIMUM GAUGE HEIGHT [m] (6)	FLOOD PEAK DISCHARGE Q [m³/s] (7)	RETURN PERIOD T [yr] (8)	FLOOD VOLUME ∀ FROM RAIN FALLEN [10⁶ m³] ON 28 JAN (9)	BETWEEN 24-30 JAN (10)	RAINFALL OVER CATCHMENT BETWEEN 24-30 JAN [mm] (11)	[10⁶ m³] (12)	RUNOFF % (10)/(12) × 100 (13)	DISCHARGE TABLE LIMIT [m³/s] (14)	REMARKS (15)
1	CROCODILE	A2M50	25° 59½	27° 50½	148^{D}	3,00	142	—	5,1	5,7	110	16,1	35	62	D: 40% DOLOMITIC
2	CROCODILE	A2M45	25° 53½	27° 54¾	653^{D}	2,46	267	—	12,2	13,6	116	75,7	18	396	D: 50% DOLOMITIC
3	JUKSKEI	A2M44	25° 53¾	27° 56	798	2,95	304	—	21,2	25,4	103	83,2	31	550	
4	HENNOPS	A2M14	25° 47¾	27° 59½	1007^{D}	$5{,}00^{*}$	172	—	12,0	18,0	114	115	16	17	*ESTIMATE. D: 60% DOLOMITIC
5	CROCODILE	A2M12	25° 48½	27° 54½	2551^{D}	3,84	385	16^{*}	45,2	57,5	115	293	20	108	*STATISTICAL ANALYSIS. D: 40% DOLOMITIC
6	MAGALIES	A2M13	25° 45½	27° 45¾	1171	3,72	305	3^{*}	10,2	14,8	87	102	15	499	*STATISTICAL ANALYSIS
7	CROCODILE	HARTBEESPOORT DAM A2R01	25° 43½	27° 51	4112	20,40	IN. 995 OUT. 825	13^{*}	72,7	103	109	448	23	2322	*INTERPOLATION IN TABLE 2 OF REPORT HRU 1/71
8	CROCODILE	A2M48	25° 34	27° 45¼	4691	4,00	758^{*}	—	66,4	83,9	105	493	17	58	*FROM FLOOD WAVE TRAVEL TIME AND CROSS SECTION AT WEIR. SEE TEXT.
9	CROCODILE	A2M19	25° 23¼	27° 34½	6131	—	$1180^{SL:}$	—		—	—	—	—	207	SL: SLOPE AREA MEASUREMENT
10	ELANDS	VAALKOPDAM A2R14	25° 18½	27° 28½	6110	12,33	IN. 501 OUT. 377	4^{*}	37,2	53,0	59	360	15	5226	*INTERPOLATION IN TABLE 2 OF REPORT HRU 1/71
11	HARTBEESSPRUIT	A2M28	25° 39	28° 19¼	161	$6{,}24^{*}$	520^{**}	330^{**}	10,4	11,0	145	23,3	47	36	*ESTIMATE **SEE TABLE 1
12	EDENVALESPRUIT	A2M29	25° 39	28° 23½	129	1,30	46	—	—	—	—	—	—	20	
13	PIENAARS	TP BRIDGE 1787 ON ROAD P2-5*	25° 40¾	28° 21½	357	—	1340	$>500^{**}$	—	—	—	—	—		*4km UPSTREAM OF STATION A2M27 **RATIONAL, UNITGRAPH AND ROBERTS METHODS
14	PIENAARS	ROODEPLAATDAM A2R09	25° 37¼	28° 22¼	684	30,68	IN. 1510^{*} OUT. 1165	500^{**} 170^{**}	28,7	33,6	110	75,2	45	1020	*ROUGH ESTIMATE **RATIONAL, UNITGRAPH AND ROBERTS METHODS
15	PIENAARS	A2M06	25° 23	28° 19	1028	4,98	1105^{*}	60^{**}	46,7	51,1	94	96,6	53	535	**STATISTICAL ANALYSIS. *SEE TEXT.
16	APIES	BON ACCORD DAM A2R02	25° 37¼	28° 11¼	315	$3{,}00^{*}$	535^{**}	45^{**}	—	—	—	—	—	—	*ESTIMATE **SEE TABLE 1
17	APIES	A2M26	25° 24½	28° 16¾	676	$6{,}50^{*}$	$678^{SL:}$	60^{**}	25,2	30,4	122	82,5	37	192	*ESTIMATE. SL: SLOPE AREA MEASUREMENT. **RATIONAL, UNITGRAPH AND ROBERTS METHODS
18	PIENAARS	KLIPVOORDAM A2R12	25° 08	27° 48½	6138	17,48	IN. 497 OUT. 445	4^{*}	78	116	74	454	25	3464	*INTERPOLATION IN TABLE 2 OF REPORT HRU 1/71
19	CROCODILE	A2M25	24° 56	27° 33	21349	9,28	1520^{B}	—	304	369	78	1665	22	277	B: CALCULATED AT TP BRIDGE 346 6,4 km DOWNSTREAM
20	CROCODILE	TP BRIDGE 2111 ON ROAD P16-2	24° 40	27° 27½	23719	$10{,}00^{*}$	1515	—	—	—	—	—	—	—	*ESTIMATE AT A2M37 5km DOWNSTREAM
21	CROCODILE	TP BRIDGE 1192 ON ROAD 115	24° 24½	27° 07	28284.	—	1300	—	—	—	—	—	—	—	
22	CROCODILE	TP BRIDGE 1329 ON ROAD 1173	24° 13	26° 54	29071	—	1400	—	—	—	—	—	—	—	

TABLE 5

PROPAGATION OF FLOOD WAVE GRAVITY CENTRES IN THE CROCODILE RIVER CATCHMENT. JANUARY 1978.

SITE No (1)	RIVER (2)	HYDRO STATION No A2 --- (3)	DAY AND HOUR OF FLOOD WAVE: PEAK (4)	DAY AND HOUR OF FLOOD WAVE: GRAVITY CENTRE (5)	DISTANCE ΔL [km], TRAVEL TIME OF FLOOD WAVE GRAVITY CENTRE ΔT [hr], VELOCITY OF FLOOD WAVE GRAVITY CENTRE V [km/hr] BETWEEN STATIONS (6)																
1	CROCODILE	M 50	28th 00h00	28th 03h15																	
2	CROCODILE	M 45	28th 04h30	28th 09h30	ΔL ΔT V	18 6¼ 2,9															
3	JUKSKEI	M 44	28th 03h00 to* 17h00	28th 13h00																	
4	HENNOPS	M 14	28th 13h00	28th 16h30																	
5	CROCODILE	M 12	28th 04h00 to** 19h00	28th 16h30	ΔL ΔT V	34 13¼ 2,6	16 7 2,3	15 3½ 4,3	19 0 N												
6	MAGALIES	M 13	28th 13h30	28th 13h30																	
7	CROCODILE	R 01	28th I.) 04h30 to*** O.) 18h00	28th I.) 13h30 O.) 16h00	ΔL ΔT V	48 10¼ 4,7	30 4 7,5	29 ½ N	33 -3 N	14 -3 N	12 0 N										
8	CROCODILE	M 48	28th 11h00	28th 17h00	ΔL ΔT V	74 13¾ 5,4	56 7½ 7,5	55 4 N	59 ½ N	40 ½ N	38 3½ N	26 1 N									
10	ELANDS	R 14	28th I.) 10h30 O.) 15h00	28th I.) 13h30 O.) 20h00																	
11	HARTBEESSPRUIT	M 28	28th 06h30	28th 06h30																	
14	PIENAARS	R 09	28th I.) 07h15 O.) 07h30	28th I.) 07h15 O.) 10h15	ΔL ΔT V										8 ¾ 10,7						
15	PIENAARS	M 06	28th 11h00	28th 16h45	ΔL ΔT V										40 10¼ 3,9	32 6½ 4,9					
17	APIES	M 26	28th 11h00	28th 15h30																	
18	PIENAARS	R 12	30th I.) 01h00 O.) 06h00	30th I.) 07h00 O.) 11h00	ΔL ΔT V										126 48 2,6	118 45 2,6	86 38 2,3	80 39 2,1			
19	CROCODILE	M 25	29th 20h00	29th 21h30	ΔL ΔT V	167 42 4,0	149 36 4,1	148 33 4,5	152 29 5,2	133 29 4,6	131 32 4,1	119 30 4,0	93 28 3,3	54 25 2,2	181 39 4,6	173 35 4,9	141 29 4,9	135 30 4,5	55 -13½ N		
20	CROCODILE	M 37	30th 16h00	30th 17h30 ****	ΔL ΔT V	216 62 3,5	198 56 3,5	197 53 3,7	201 49 4,1	182 49 3,7	180 52 3,5	168 50 3,4	142 48 3,0	103 45 2,3	230 59 3,9	222 55 4,0	190 49 3,9	184 50 3,4	104 6½ N	49 20 2,4	
21	CROCODILE	TP BRIDGE No 1192 (MAKOPPO)	1 FEBR 02h00	1 FEBR 03h30 ****	ΔL ΔT V	268 95 2,8	250 89 2,8	249 85 2,9	253 81 3,1	234 81 2,9	232 84 2,8	220 82 2,7	194 81 2,4	155 77 2,0	282 91 3,1	274 88 3,1	242 81 3,0	236 82 2,9	156 39 4,0	101 52 1,9	52 32 1,6

SCHEDULE

FROM M14

TO M12

TO R01

TO M48

NOTES

I) INFLOW HYDROGRAPH

O.) OUTFLOW HYDROGRAPH

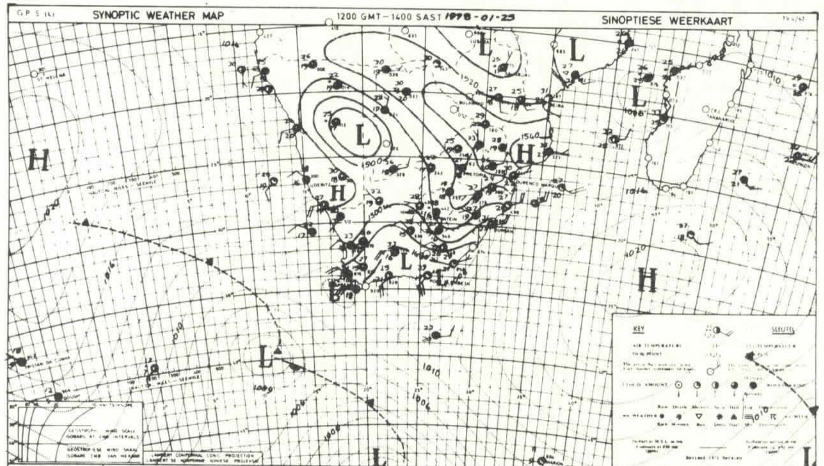
N : NOT REALISTIC

* THREE PEAKS

** FOUR PEAKS

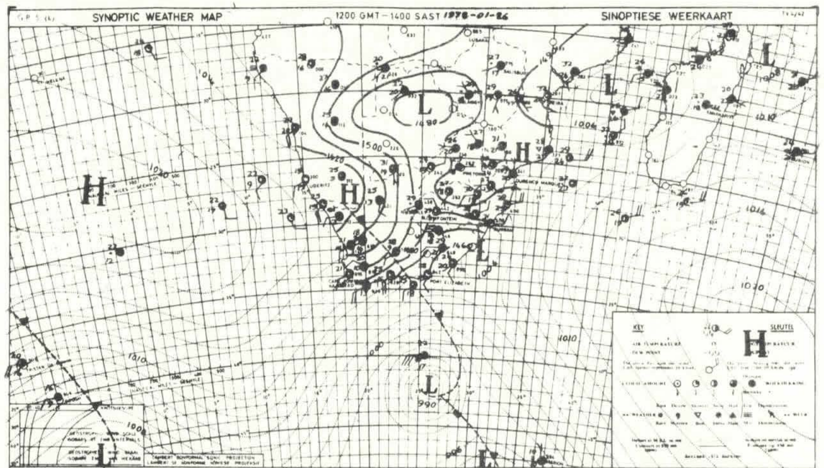
*** THREE PEAKS BETWEEN 04h30 - 17h30

**** ESTIMATE



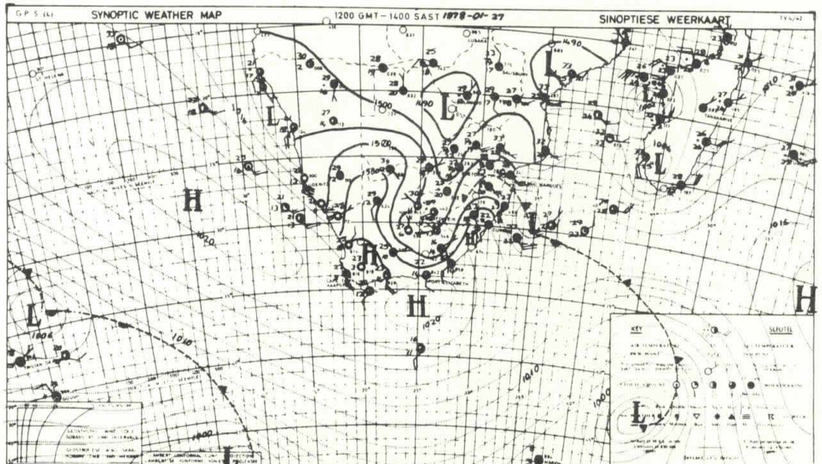
INFERENCE The frontal system west of the continent is causing rainy weather over Western and South-western Cape. With the eastward movement of this system drier air will push in over the Cape Province and also over the southern parts of the Free State tomorrow.

AFLEIDING Die frontale stelsel wes van die land veroorsaak reënweer oor Wes- en Suidwes-Kaap en soos die sisteem ooswaarts beweeg sal droër lug instoot oor die Kaapprovinsie en môre ook die suidelike dele van die Vrystaat.



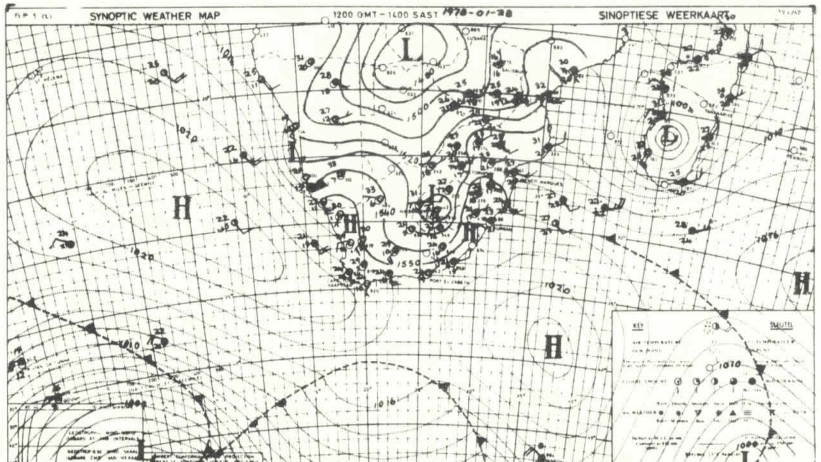
INFERENCE Scattered thundershowers will occur today east of the trough over the interior but the dry air is moving rapidly eastwards. Cooler weather with showers will spread eastwards along the coast behind the cold front

AFLEIDING Verspreide donderbuie sal vandag nog oos van die trog oor die binneland voorkom maar die droë lug beweeg vinnig ooswaarts. Koeler weer met buie sal oor die kusstreke ooswaarts uitbrei agter die koufront.



INFERENCE The sharp trough of low-pressure over the interior is being pinched between the high-pressure systems over the eastern and western interior and will cause scattered thundershowers with heavy falls in the vicinity of the trough.

AFLEIDING Met die skerp trog van lae druk oor die binneland wat vasgeknyp word tussen die hoëdrukstelsels oor die oostelike en westelike binneland sal verspreide donderbuie voorkom met swaar neerslae in die omgewing van die trog.



INFERENCE The trough over the interior has weakened considerably but further showers will develop in the moist air over the eastern half of the country.

AFLEIDING Die trog oor die binneland het heelwat verswak maar verdere buie sal nog in die vogtige lug oor die oostelike helfte van die land voorkom.

FIG.1 SYNOPTIC WEATHER CHARTS ON 25, 26, 27 AND 28 JAN 1978

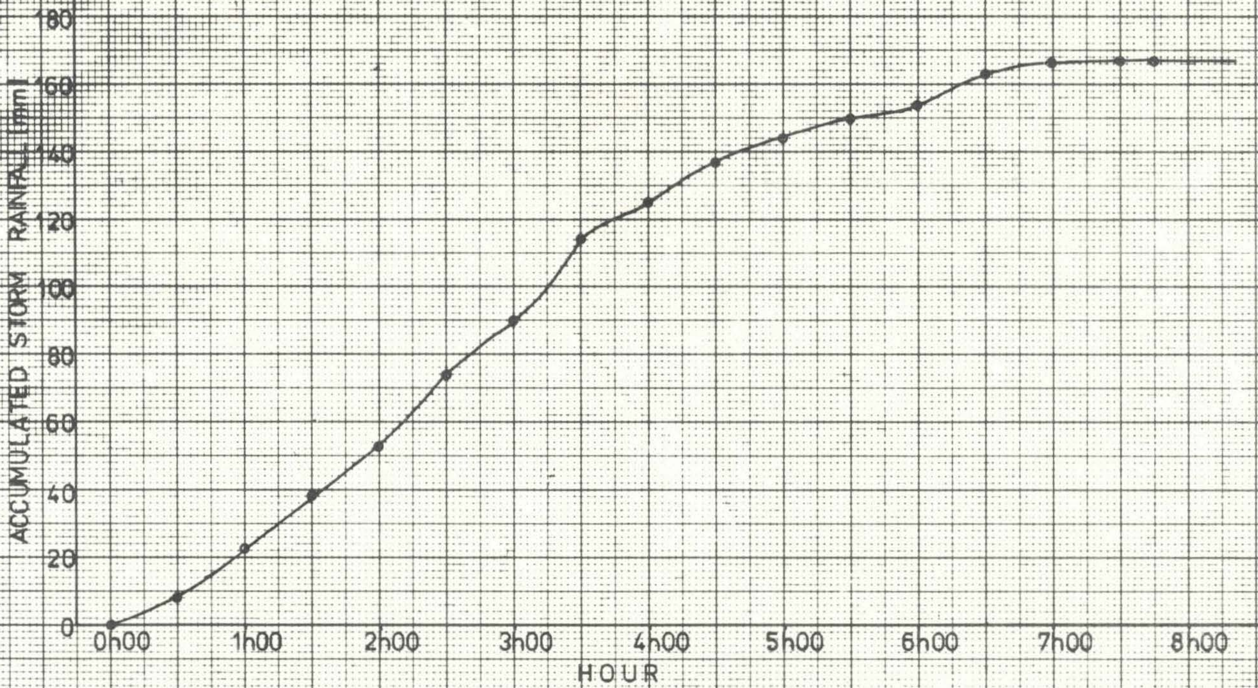


FIG. 2 ACCUMULATED STORM RAINFALL ON 28 JAN 1978
AT WEATHER BUREAU STATION No 513/314A (FORUM)

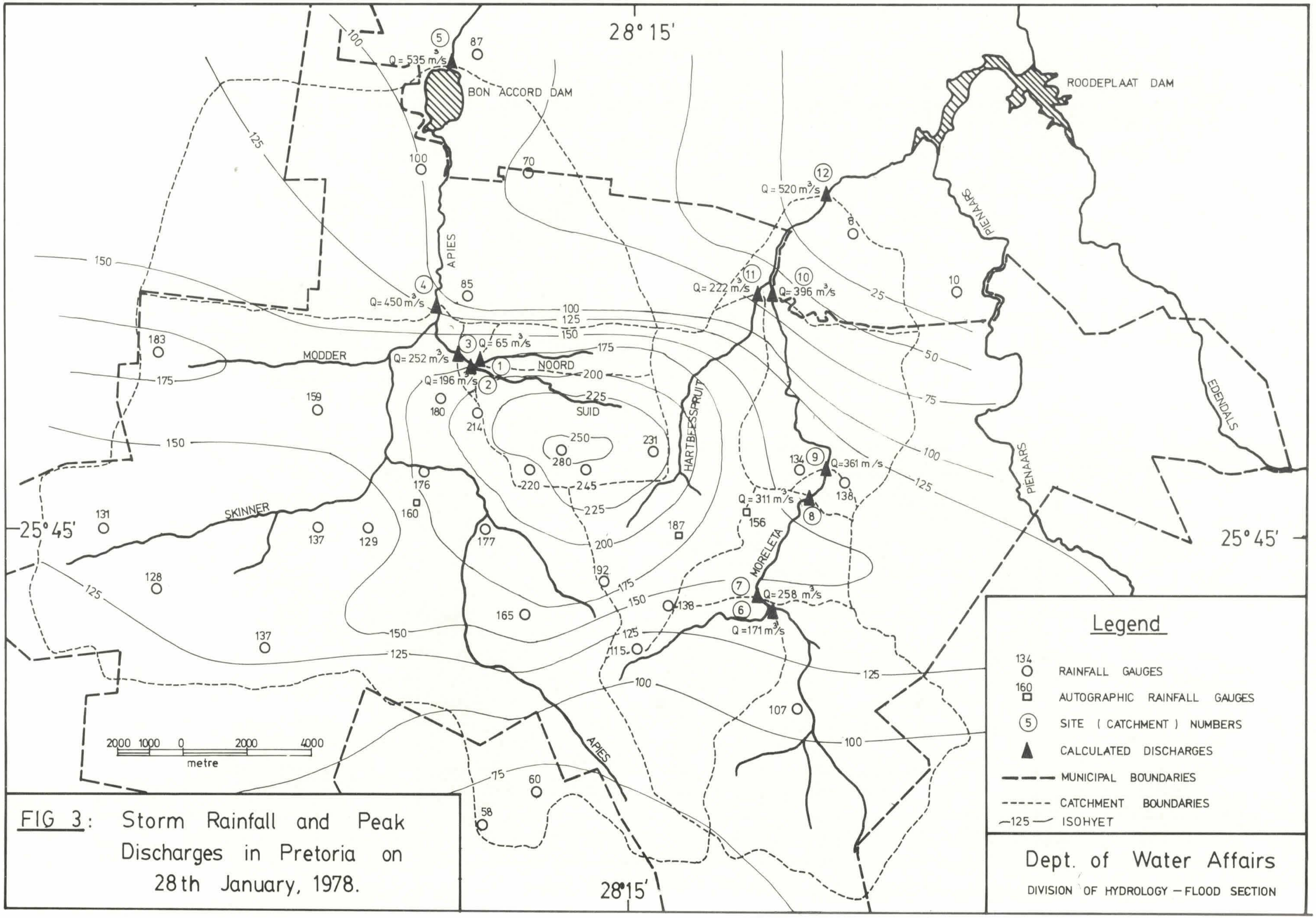


FIG 3: Storm Rainfall and Peak Discharges in Pretoria on 28th January, 1978.

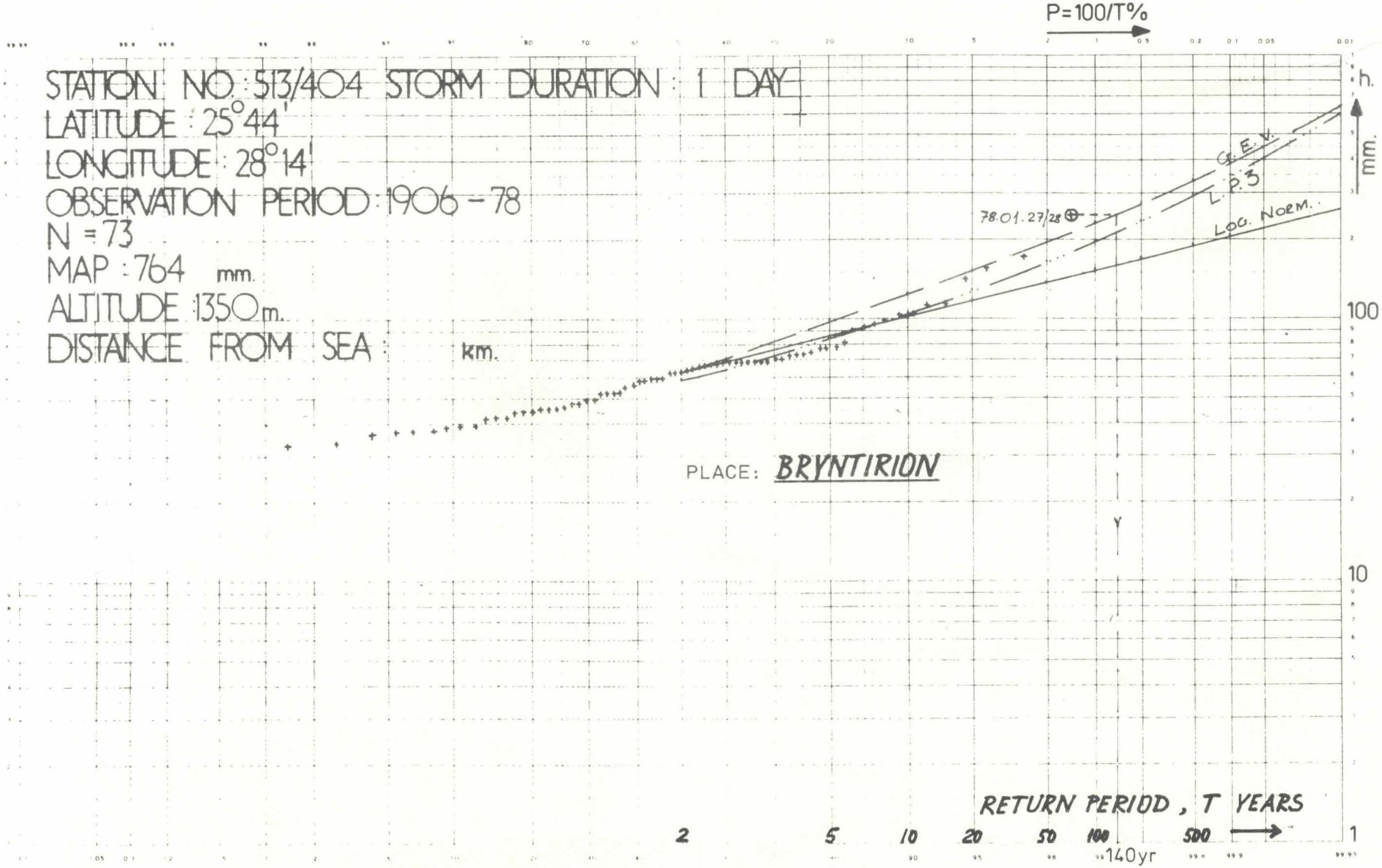

P=100/T%
STATION NO. 513/404 STORM DURATION : 1 DAY
LATITUDE : 25°44'
LONGITUDE : 28°14'
OBSERVATION PERIOD : 1906 - 78
N = 73
MAP : 764 mm.
ALTITUDE : 1350 m.
DISTANCE FROM SEA : km.
G.E.V.
L.P.3
LOG. NORM.
78.01.27/28
PLACE: BRYNTIRION
h.
mm.
100
10
1
RETURN PERIOD , T YEARS
2
5
10
20
50
100
500
140yr

FIG. 4a

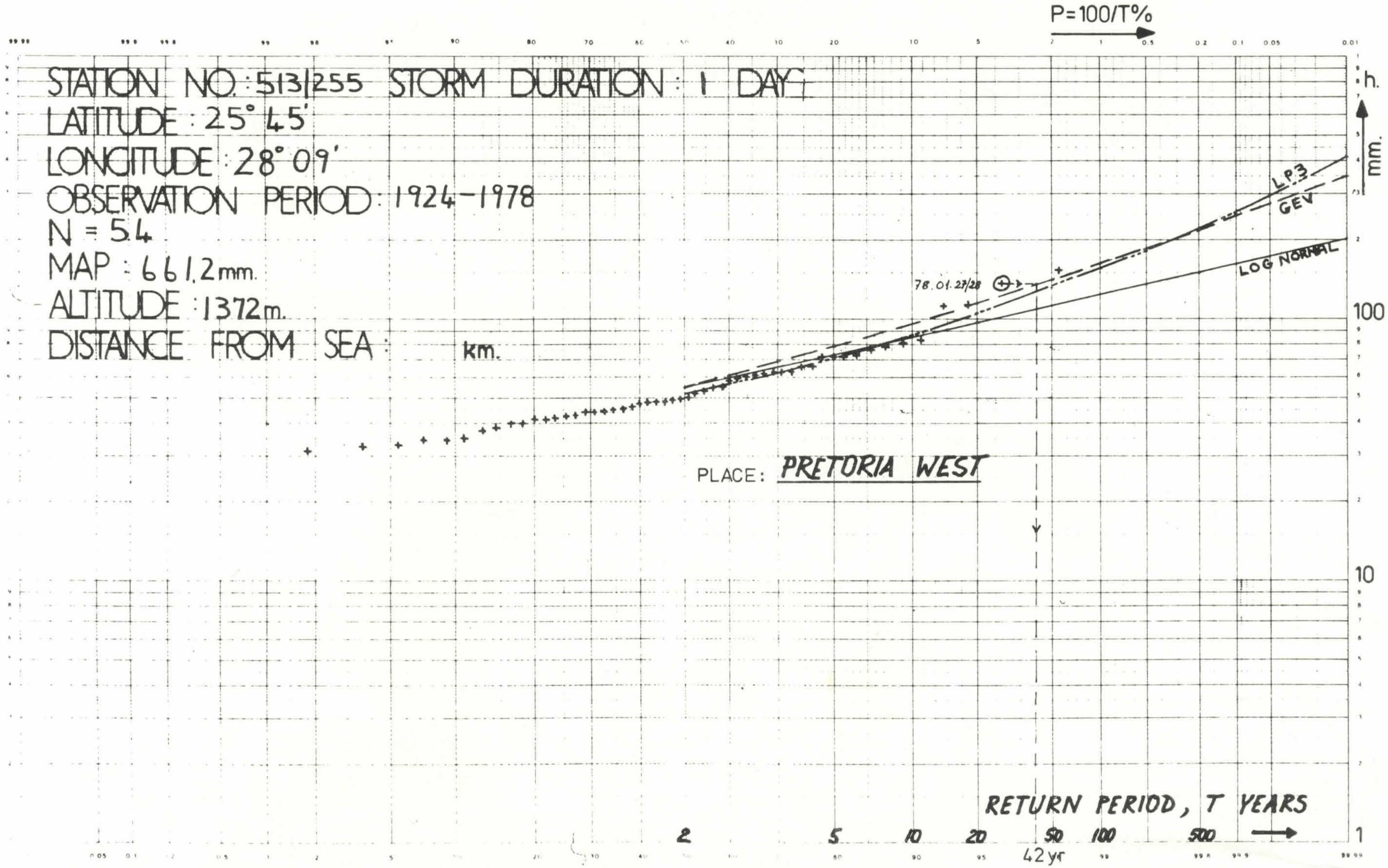
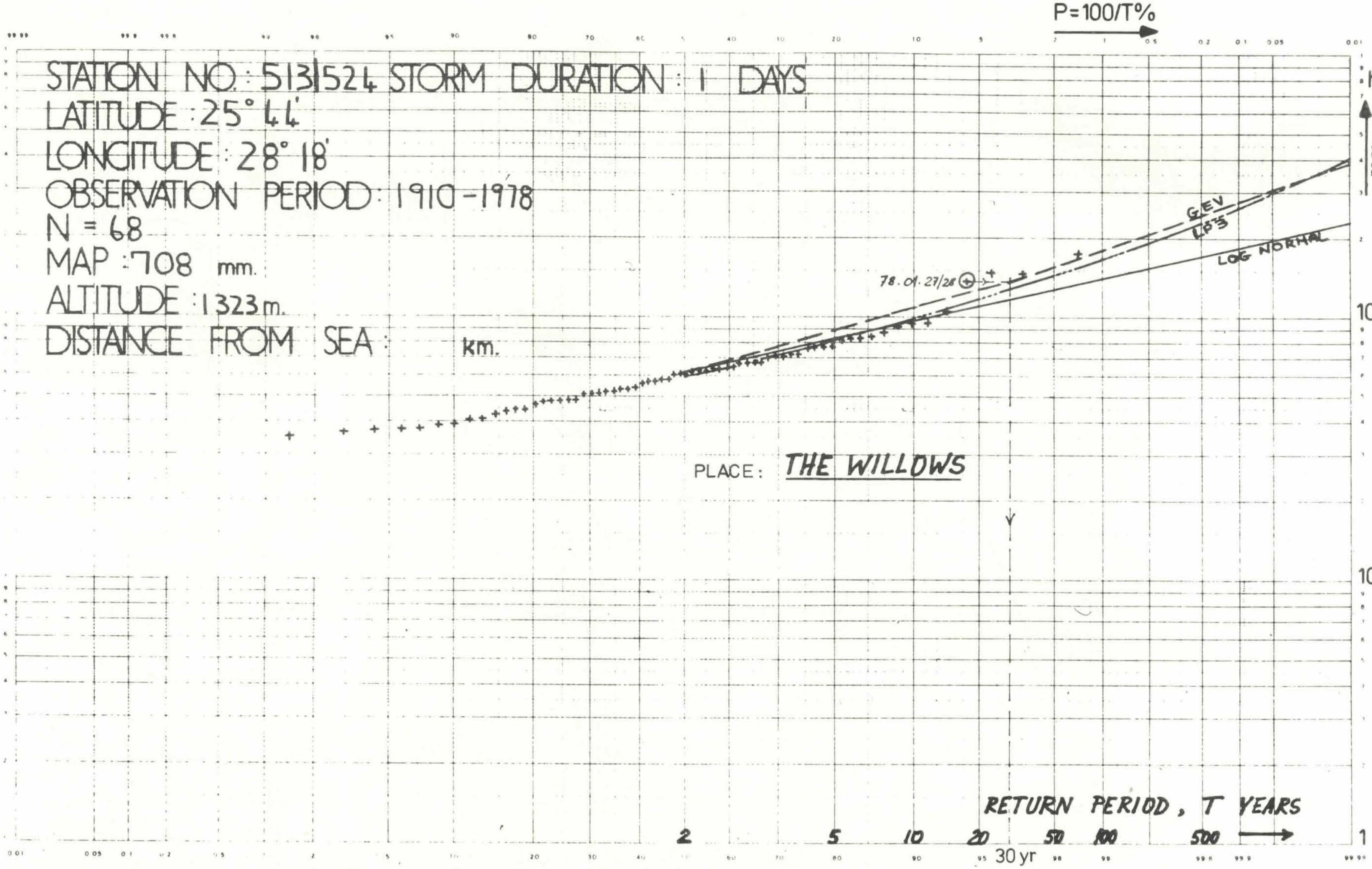

P=100/T%
STATION NO: 513/255 STORM DURATION: 1 DAY
LATITUDE: 25° 45'
LONGITUDE: 28° 09'
OBSERVATION PERIOD: 1924-1978
N = 54
MAP: 661.2mm.
ALTITUDE: 1372m.
DISTANCE FROM SEA: km.
h.
mm.
L.P.3
GEV
LOG NORMAL
78.01.27/28
100
10
1
PLACE: PRETORIA WEST
RETURN PERIOD, T YEARS
2
5
10
20
50
100
500
42yr

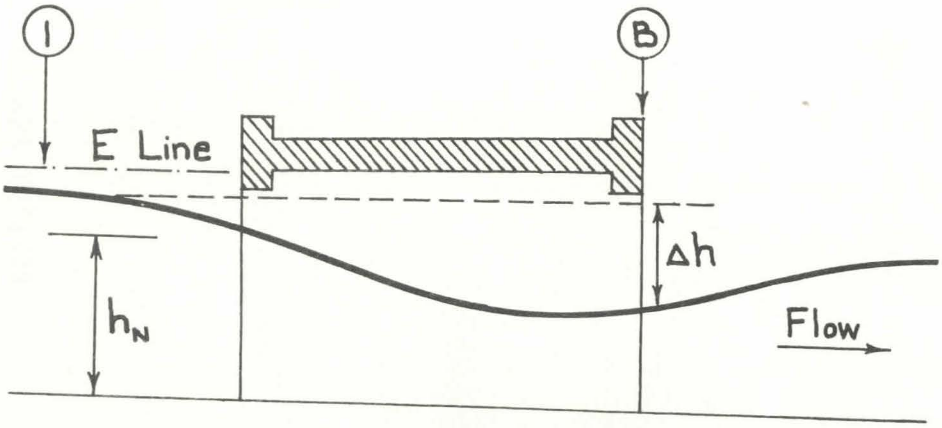
FIG. 4b



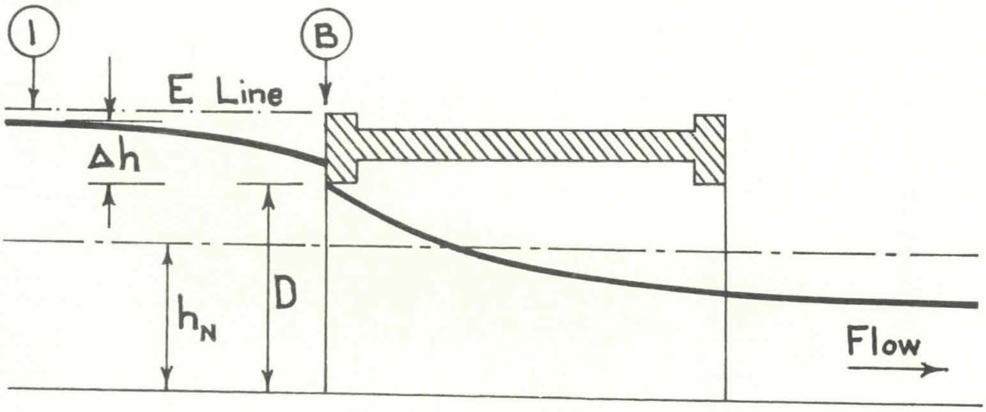
P=100/T%
STATION NO: 513|524 STORM DURATION: 1 DAYS
LATITUDE: 25° 44'
LONGITUDE: 28° 18'
OBSERVATION PERIOD: 1910-1978
N = 68
MAP: 708 mm.
ALTITUDE: 1323 m.
DISTANCE FROM SEA: km.
h.
mm.
100
10
1
GEV
LP3
LOG NORMAL
78.01.27/28
PLACE: THE WILLOWS
RETURN PERIOD, T YEARS
2
5
10
20
50
100
500
30 yr

FIG. 4c

FLOW TYPES AT BRIDGES (i)



TYPE a FLOW : $Q = C_a A_B \sqrt{2g\left(\Delta h - h_f + \frac{\alpha_1 v_1^2}{2g}\right)}$



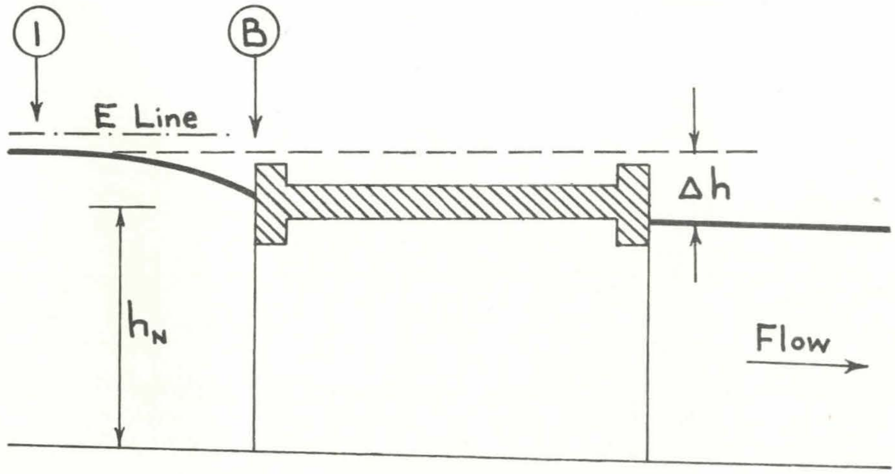
TYPE b FLOW : Orifice Flow

$$Q = C_b A_B \sqrt{2g\left(\Delta h + D/2 + \frac{\alpha_1 v_1^2}{2g}\right)}$$

NOTE: SYMBOLS IN TEXT (PART 2·2·3)

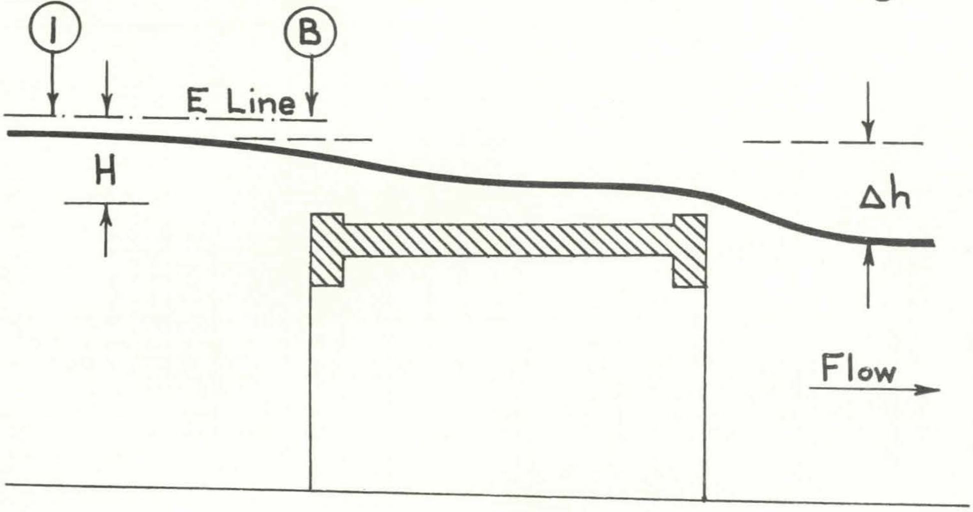
FIG. 5a

FLOW TYPES AT BRIDGES (ii)



TYPE c FLOW : Pipe Flow

$$Q = 0{,}8\, A_B \sqrt{2g\left(\Delta h + \frac{\alpha_1 v_1^2}{2g}\right)}$$



TYPE d FLOW : Over Road + Under Bridge

Over Road : $Q = C_f L H^{3/2}$

Under Bridge : $Q = 0{,}8\, A_B \sqrt{2g\left(\Delta h + \frac{\alpha_1 v_1^2}{2g}\right)}$

NOTE: SYMBOLS IN TEXT (PART 2·2·3)

FIG. 5 b

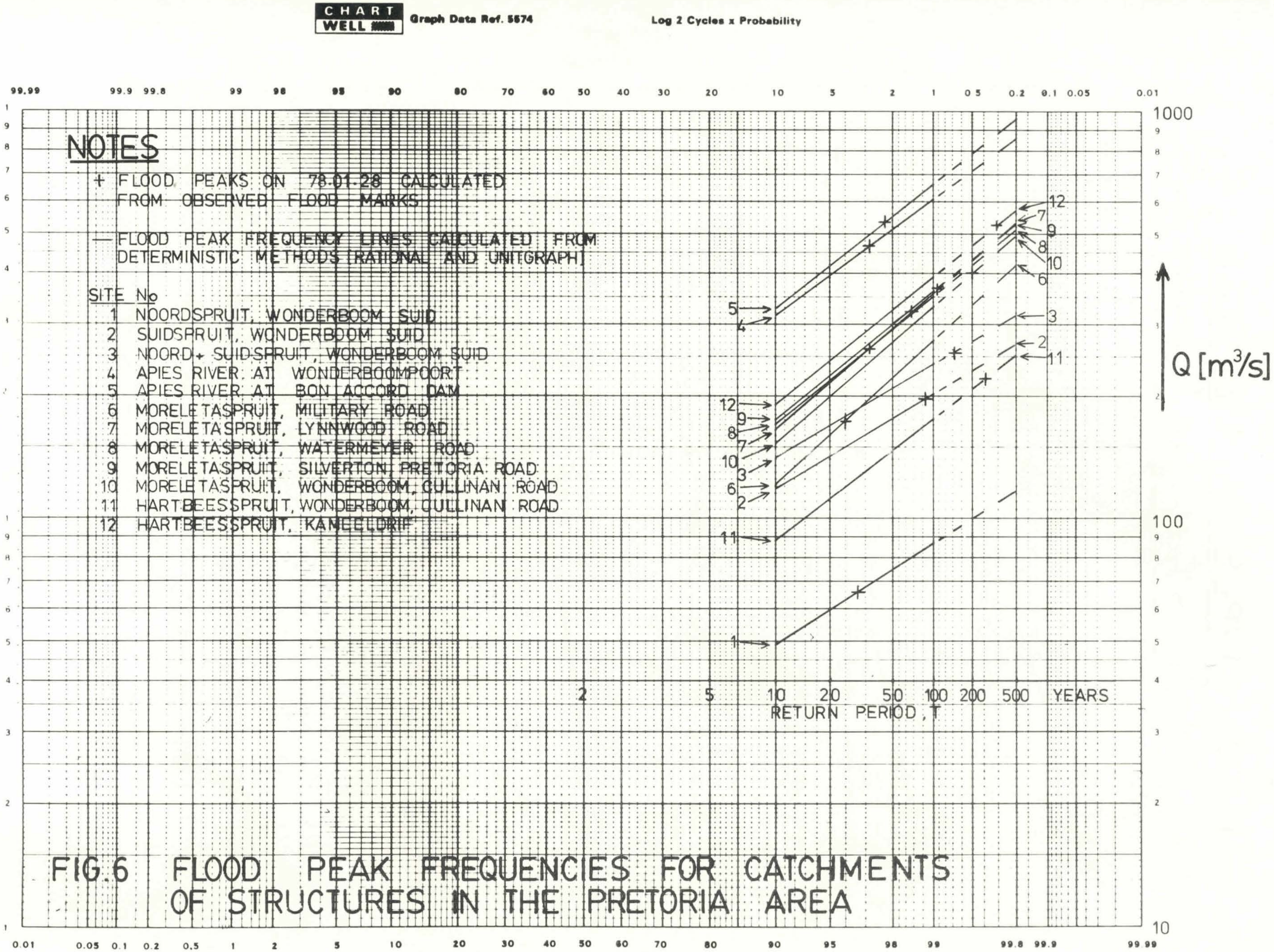


FIG.6 FLOOD PEAK FREQUENCIES FOR CATCHMENTS OF STRUCTURES IN THE PRETORIA AREA

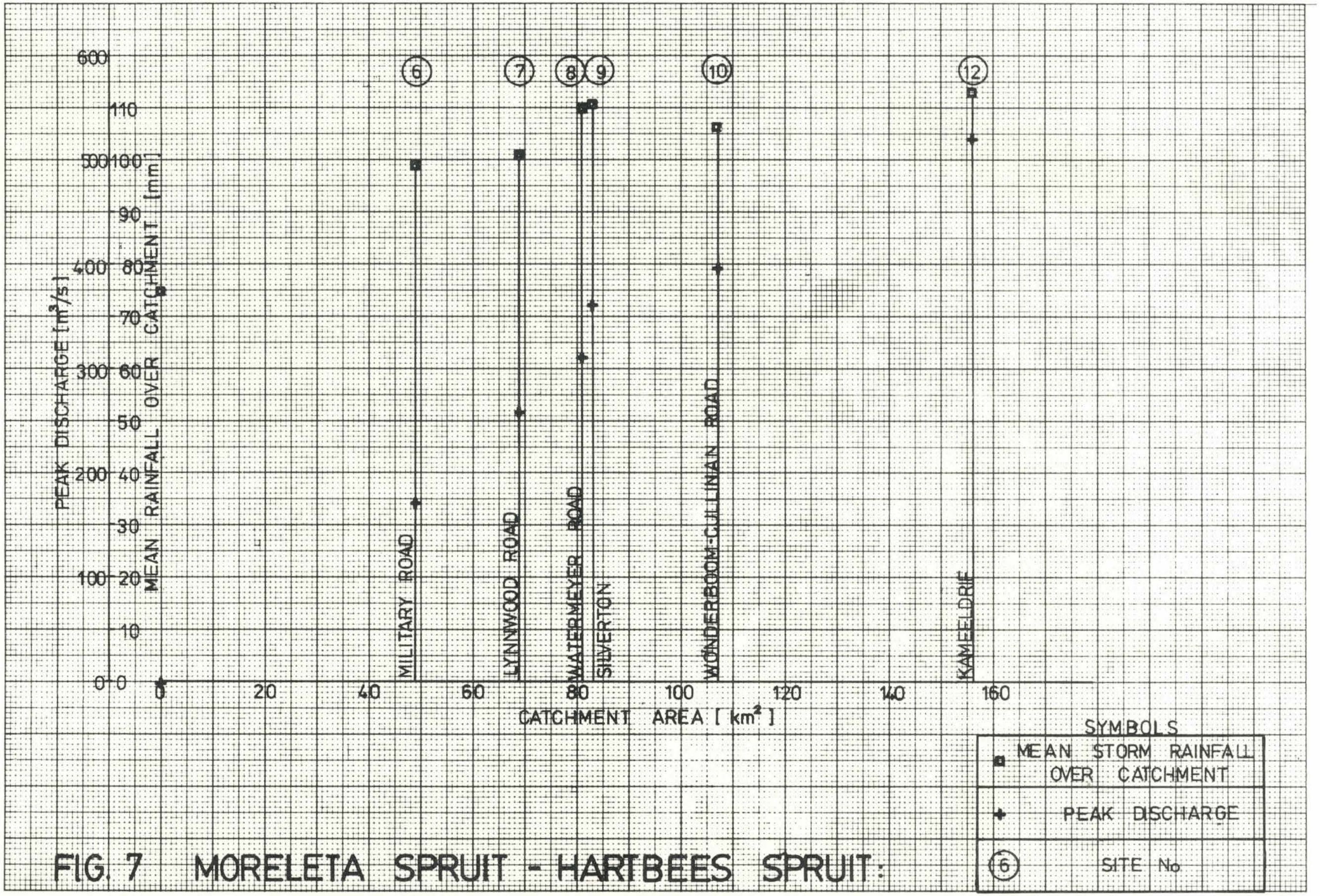


FIG. 7 MORELETA SPRUIT - HARTBEES SPRUIT:

PEAK DISCARGE AND MEAN STORM RAINFALL ON 78.01.28 VS CATCHMENT AREA

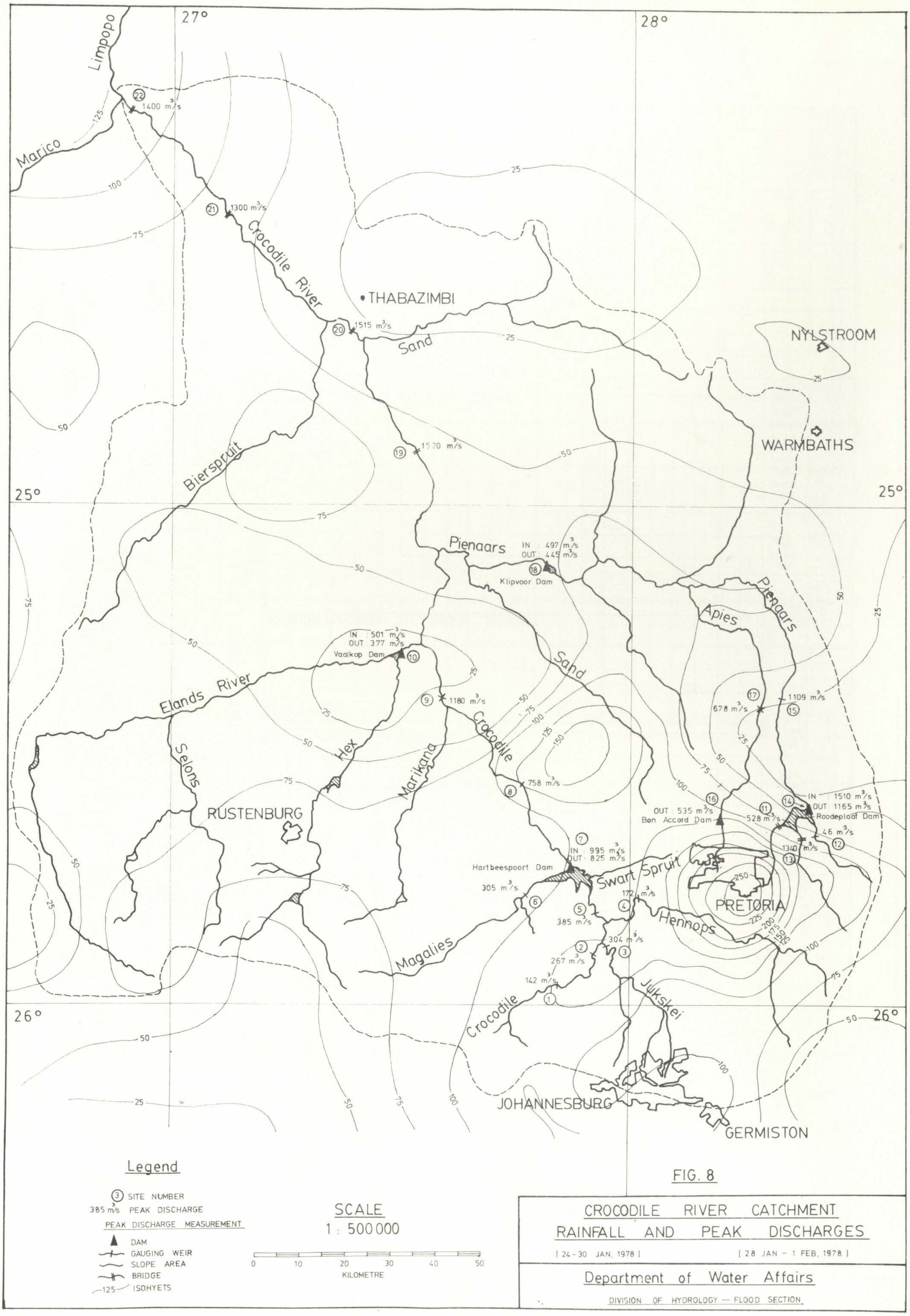


FIG. 8

CROCODILE RIVER CATCHMENT
RAINFALL AND PEAK DISCHARGES
(24-30 JAN, 1978) (28 JAN - 1 FEB, 1978)

Department of Water Affairs
DIVISION OF HYDROLOGY — FLOOD SECTION

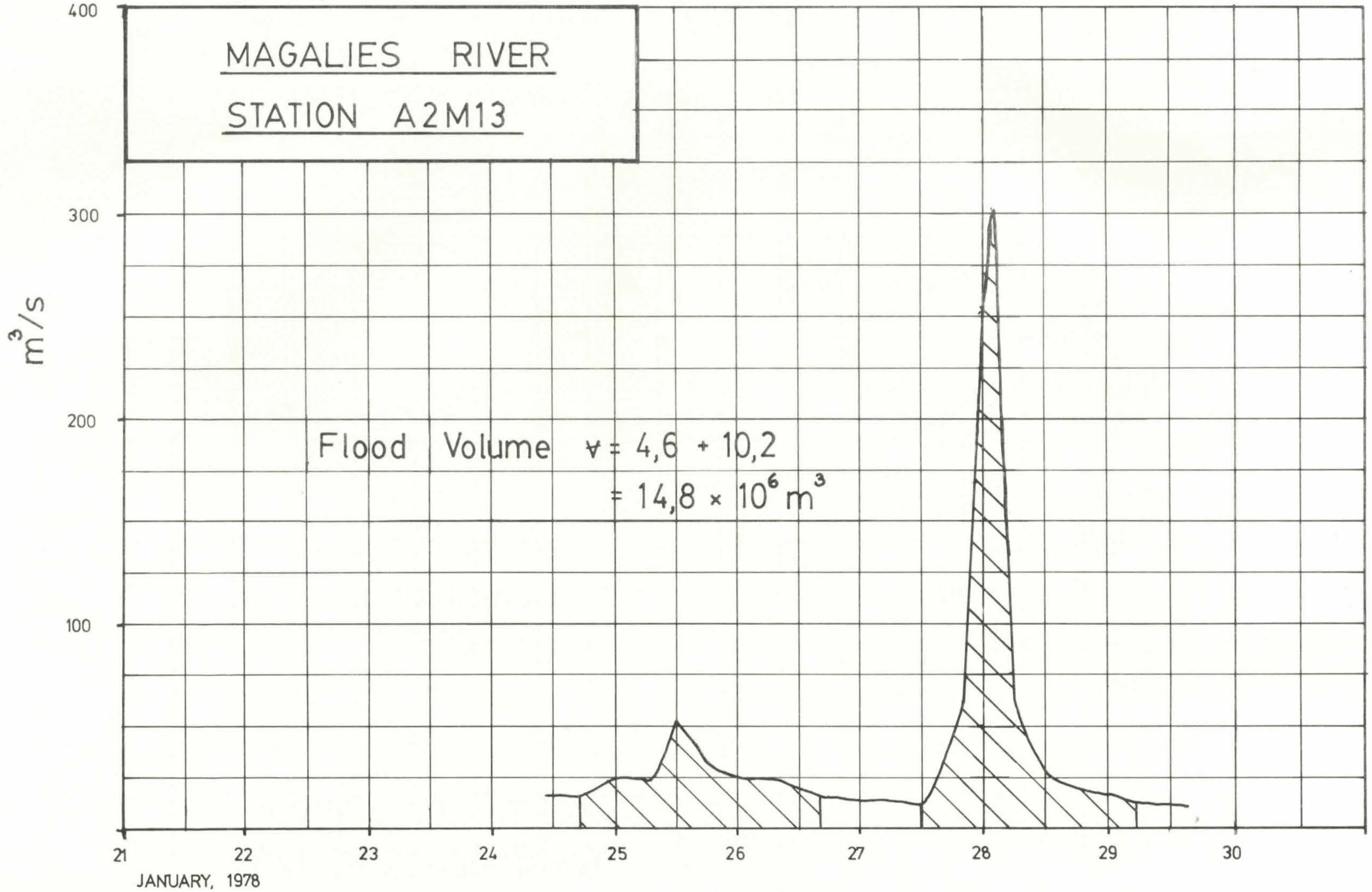
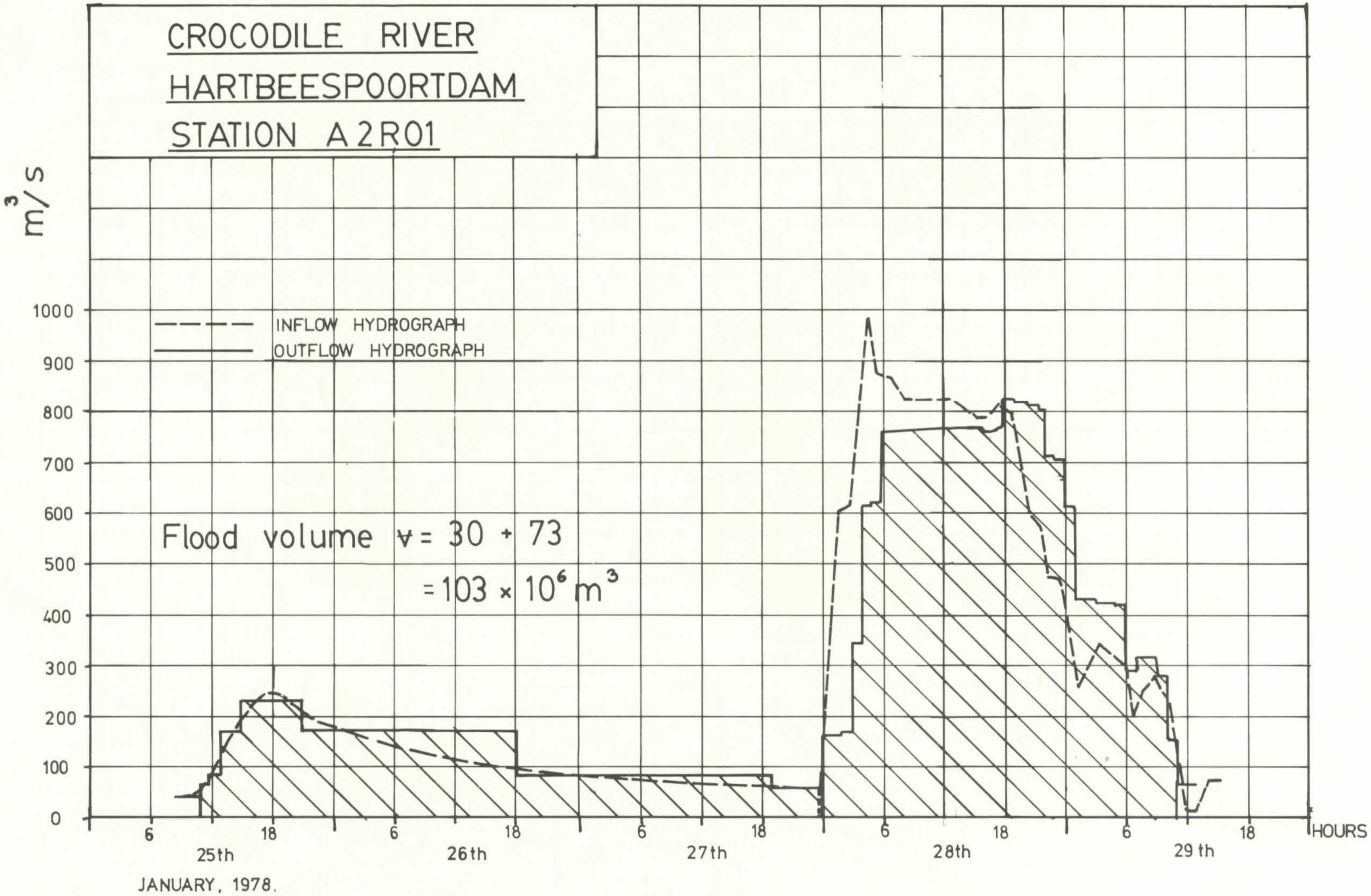

MAGALIES RIVER
STATION A2M13
400
300
200
100
m^3/s
Flood Volume $\forall$ = 4,6 + 10,2
= $14,8 \times 10^6\ m^3$
21
22
23
24
25
26
27
28
29
30
JANUARY, 1978

FIG. 9a



CROCODILE RIVER
HARTBEESPOORTDAM
STATION A2R01
m³/s
INFLOW HYDROGRAPH
OUTFLOW HYDROGRAPH
Flood volume ∀ = 30 + 73
= 103 × 10⁶ m³
1000
900
800
700
600
500
400
300
200
100
0
6
18
6
18
6
18
6
18
6
18
HOURS
25th
26th
27th
28th
29 th
JANUARY, 1978.

FIG. 9b

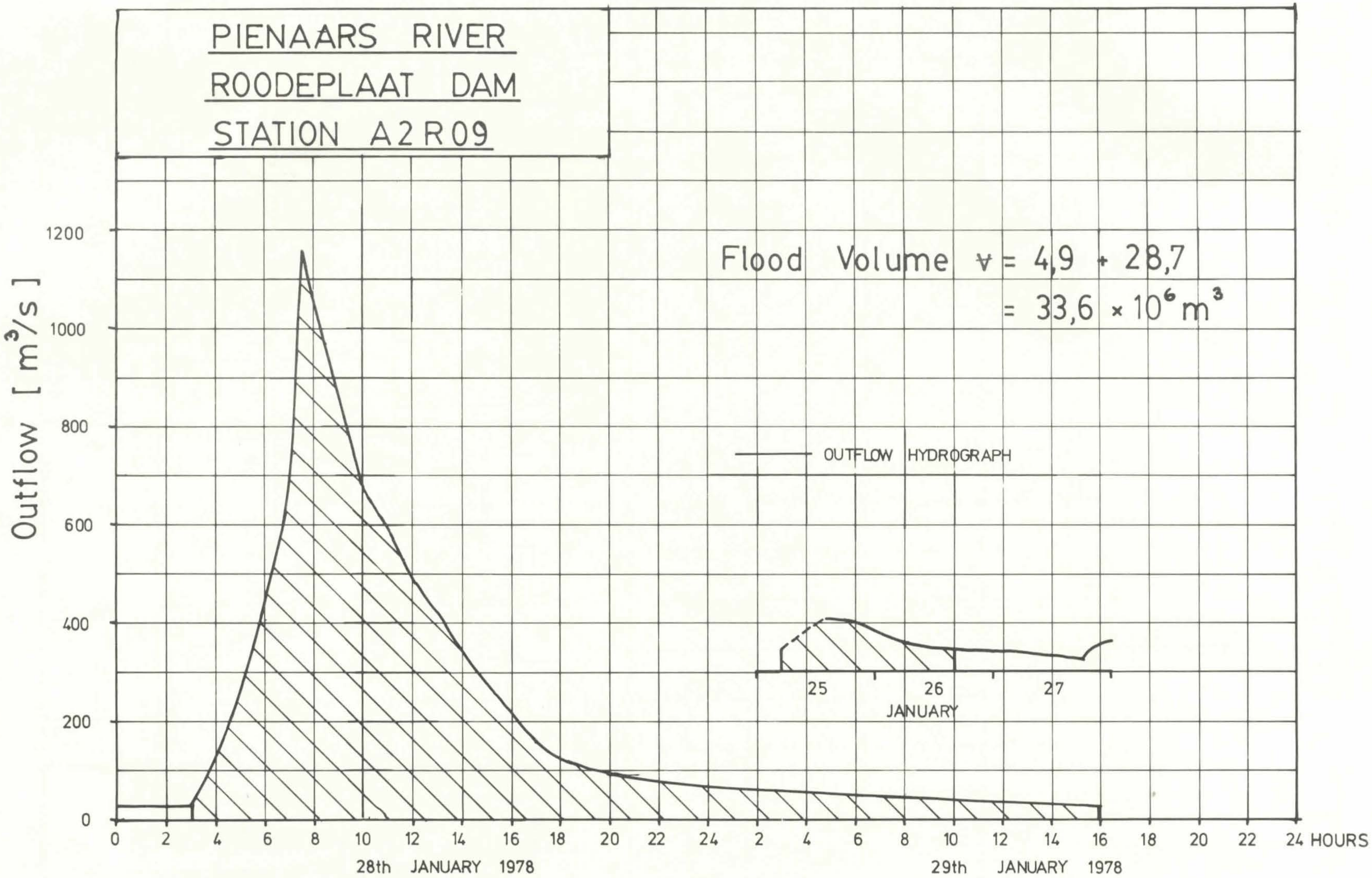

PIENAARS RIVER
ROODEPLAAT DAM
STATION A2R09
Flood Volume ∀ = 4,9 + 28,7
= 33,6 × 10⁶ m³
OUTFLOW HYDROGRAPH
25
26
27
JANUARY
Outflow [m³/s]
0
200
400
600
800
1000
1200
0 2 4 6 8 10 12 14 16 18 20 22 24 2 4 6 8 10 12 14 16 18 20 22 24 HOURS
28th JANUARY 1978
29th JANUARY 1978

FIG. 9c

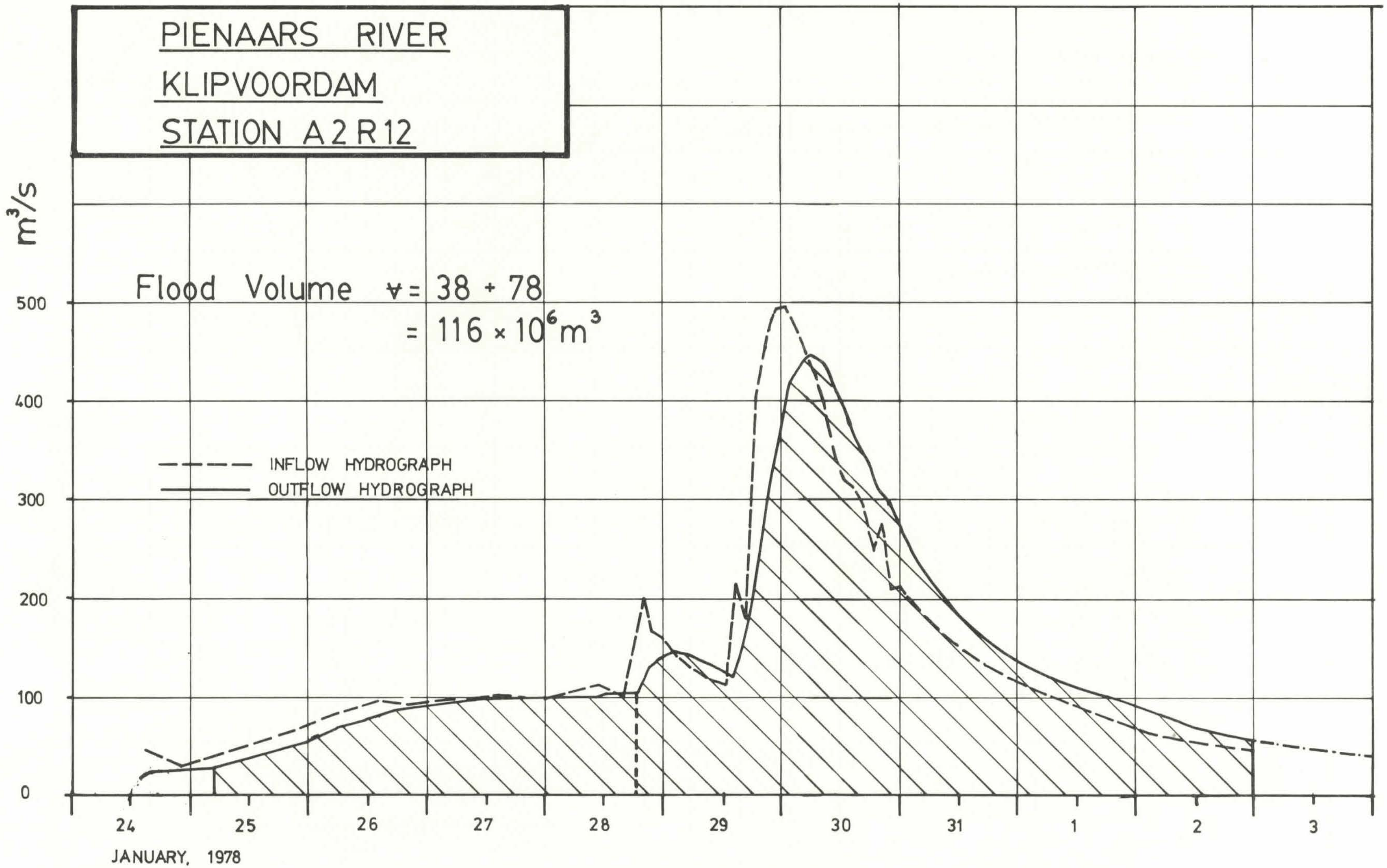

PIENAARS RIVER
KLIPVOORDAM
STATION A2R12
m³/s
Flood Volume ∀ = 38 + 78
= 116 × 10⁶ m³
INFLOW HYDROGRAPH
OUTFLOW HYDROGRAPH
500
400
300
200
100
0
24
25
26
27
28
29
30
31
1
2
3
JANUARY, 1978

FIG. 9d

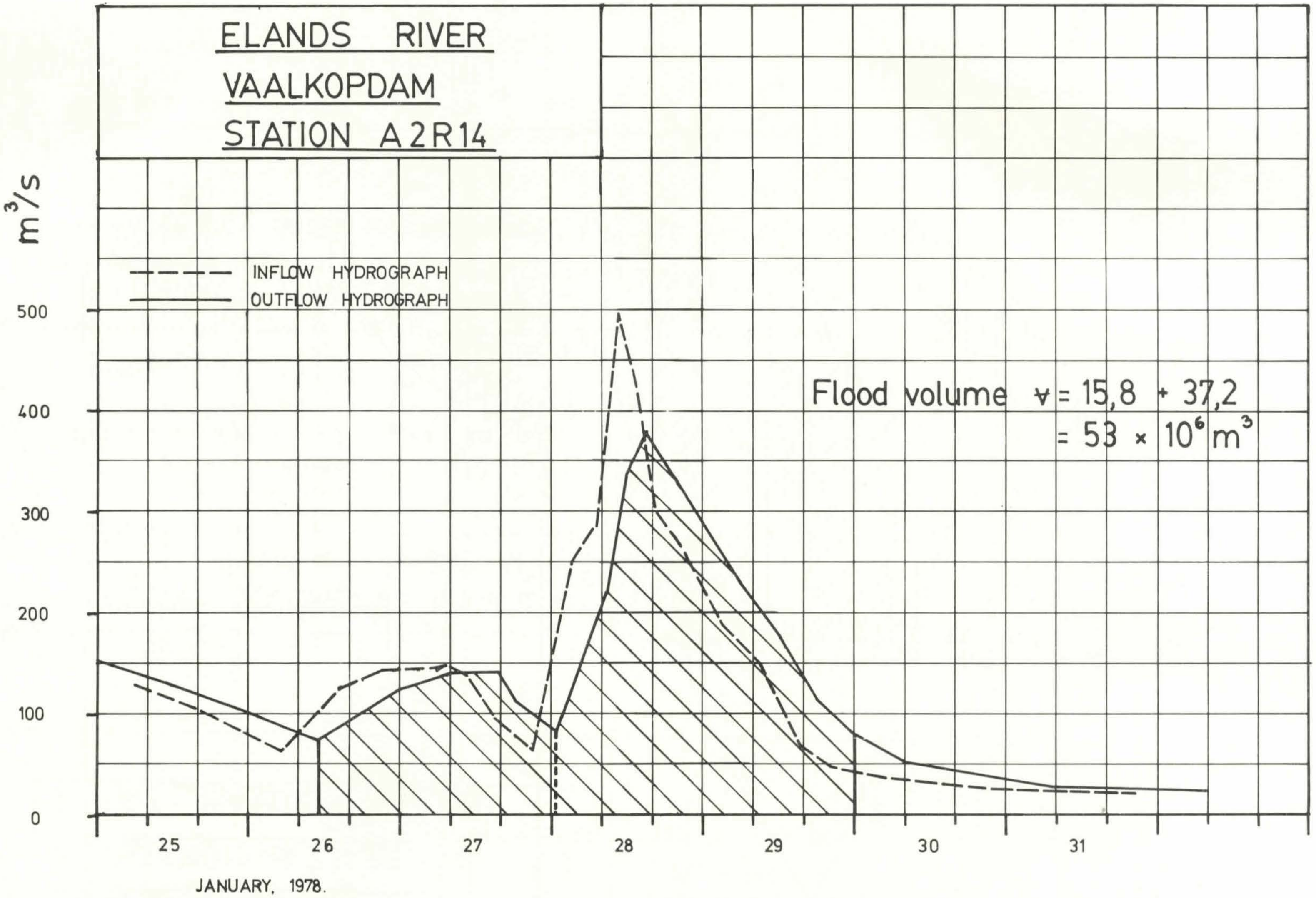

ELANDS RIVER
VAALKOPDAM
STATION A2R14
m³/s
INFLOW HYDROGRAPH
OUTFLOW HYDROGRAPH
Flood volume ∀ = 15,8 + 37,2
= 53 × 10⁶ m³
500
400
300
200
100
0
25
26
27
28
29
30
31
JANUARY, 1978.

FIG. 9e

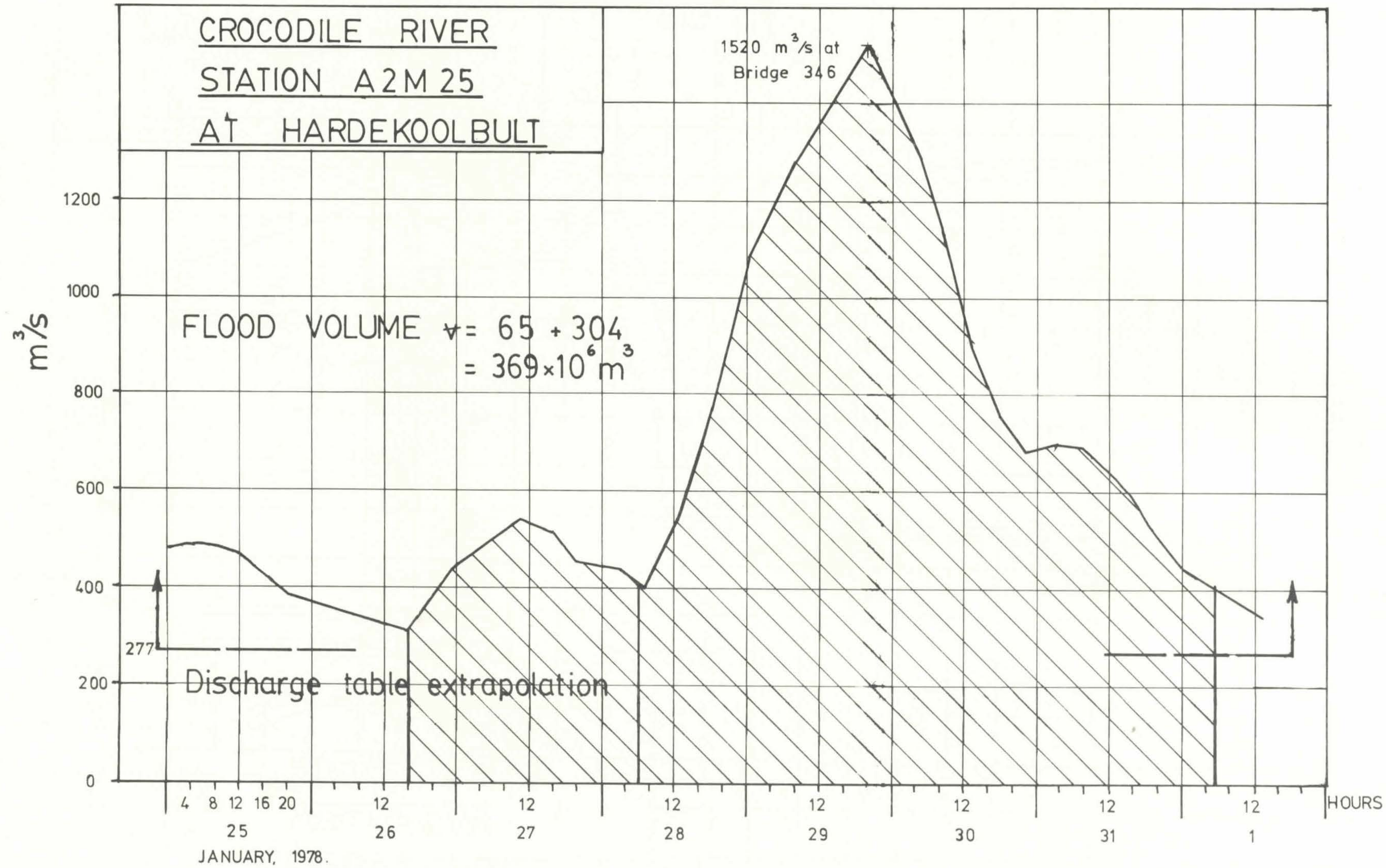

CROCODILE RIVER
STATION A2M 25
AT HARDEKOOLBULT
1520 m³/s at
Bridge 346
FLOOD VOLUME ∀ = 65 + 304
= 369×10⁶ m³
Discharge table extrapolation
m³/s
1200
1000
800
600
400
277
200
0
4 8 12 16 20
12 12 12 12 12 12 12
HOURS
25 26 27 28 29 30 31 1
JANUARY, 1978.

FIG. 9f

RATIONAL METHOD

DEPT. OF WATER AFFAIRS (HYDROLOGY) NOV. 1977

IDETIFICATION OF SITE

WATER COURSE : ....

FARM : ....

LATITUDE : ....

LONGITUDE : ....

PHYSICAL CHARACTERISTIC OF CATCHMENT

CATCHMENT AREA, A = ... km^2

AREAL WEIGHTING FACTORS $\alpha+\beta+\gamma=1$		
RURAL	URBAN	LAKES
α =	β =	γ =

PHYSICAL FEATURES IN % OF AREA

RURAL						URBAN	
SURFACE SLOPE	%	PERMEABILITY OF SOIL	%	VEGETATION	%	OCCUPATION	%
< 1%		VERY PERMEABLE		DENSE BUSH, FOREST		LAWNS, PARKS	
1 to 3		PERMEABLE		CULTIVATED LAND, THIN BUSH		RESIDENTIAL	
3 to 10		SEMI - PERMEABLE		GRASS LAND		INDUSTRIAL	
10 to 30		IMPERMEABLE		BARE SURFACE		DOWNTOWN	
> 30						STREETS	
TOTAL	100	TOTAL	100	TOTAL	100	TOTAL	100
AVERAGE SLOPE :		DOLOMITIC		DENSE WOOD (man made)			

LONGEST WATERCOURSE, L = ... km AVERAGE SLOPE ALONG L, S = ... m/m

RAINFALL INTENSITY

TIME OF CONCENTRATION

$$t_c = \left(\frac{0.87L^2}{1000S}\right)^{0.385} = \text{..........} \text{ hrs}$$

NOTE : FOR FLAT CATCHMENTS WITH $A < 5\ km^2$ WHERE OVERLAND FLOW IS DOMINANT, USE :

$$t_c = 0.604\left(\frac{rL}{S^{0.5}}\right)^{0.467} = \text{..........} \text{ hrs}$$

VALUES OF r	
BARE SOIL	0·1
POOR GRASS	0·3
AVERAGE GRASS, CULTIV.	0·4
DENSE GRASS	0·8

MEAN ANNUAL RAINFALL, MAP = ... mm

RAINFALL REGION : WINTER ☐, YEAR ROUND ☐, SUMMER ☐ (from Fig C1)

EXTREME POINT RAINFALL REGION NUMBER : .... (from Fig. C3)

RETURN PERIOD	T(Years)		10		100		MAX
POINT RAINFALL, h(mm)							
POINT INTENSITY, $i = h/t_c$ (mm/hr)							
AREAL REDUCTION, a							
INTENSITY AVERAGED OVER AREA, I(mm/hr)							

FIGURES USED : OTHERS :

C_2		C_4	
C_6		C_7	

D ....

RUNOFF COEFFICIENT

	T(Years)		10		100		MAX
RURAL C_1							
URBAN C_2							
LAKES $C_3 = 1$							
COMBINED $C = \alpha C_1 + \beta C_2 + \gamma$							

NOTES

PEAK DISCHARGE

	T(Years)		10		100		MAX
$Q = 0.278\ C\ I\ A$ m^3/s							
CORRECTED PEAK Q^* m^3/s							

NOTES

RECOMMENDED VALUES OF RUNOFF COEFFICIENT C

RURAL C_1

COMPONENT		CATEGORY	MAP(mm) <600	600 - 900	>900
C_s	SURFACE SLOPE IN %	< 3	0·01	0·03	0·05
		3 to 10	0·06	0·08	0·11
		10 to 30	0·12	0·16	0·20
		>30	0·22	0·26	0·30
C_p	PERMEABILITY OF SOIL	VERY PERMEABLE	0·03	0·04	0·05
		PERMEABLE	0·06	0·08	0·10
		SEMI - PERMEABLE	0·12	0·16	0·20
		IMPERMEABLE	0·21	0·26	0·30
C_v	VEGETATION	DENSE BUSH, FOREST	0·03	0·04	0·05
		CULTIVATED LAND, THIN BUSH	0·07	0·11	0·15
		GRASS LAND	0·17	0·21	0·25
		BARE SURFACE	0·26	0·28	0·30

NOTES: (1) Influence of return period T

T (Years)	C_1
≤ 20	$0.67(C_s+C_p+C_v)$
50	$0.83(C_s+C_p+C_v)$
100	$C_s+C_p+C_v$
MAX	$C_s+C_{pmax}+C_{vmax}$

(2) Dense wood (man made) applies only if more than 25% of A is covered by it.

S.face slope	T(Years)	C_1
<3%	10	0·10
	100	0·15
>10%	10	0·13
	100	0·20

for T=MAX refer to NOTE (1)

URBAN C_2

OCCUPATION	RUNOFF COEFFICIENT
LAWNS	
sandy, flat < 2%	0·05 - 0·10
sandy, steep > 7%	0·15 - 0·20
heavy soil, flat < 2%	0·13 - 0·17
heavy soil, steep > 7%	0·25 - 0·35
RESIDENTIAL	
single family area	0·30 - 0·50
apartment dwelling	0·50 - 0·70
INDUSTRIAL	
light areas	0·50 - 0·80
heavy areas	0·60 - 0·90
BUSINESS	
downtown	0·70 - 0·95
neibourhood	0·50 - 0·70
STREETS	0·70 - 0·95

NOTES

(1) if limited T has no influence on C_2

(2) for T = MAX use $C_2 = 1$

GENERAL NOTE : FIGURE NUMBERS REFER TO REPORT HRU 1/72

CALCULATED BY :	DATE :

UNITGRAPH METHOD	DEPARTMENT OF WATER AFFAIRS DIVISION OF HYDROLOGY JULY 1976

IDENTIFICATION OF SITE	WATERCOURSE : ______ FARM : ______	LATITUDE : ______ LONGITUDE : ______

BASIC DATA

CATCHMENT AREA, A = ______ km^2 LONGEST WATERCOURSE, L = ______ km L_c = ______ km

AVERAGE SLOPE OF LONGEST WATERCOURSE, S = ______ km/km

MEAN ANNUAL RAINFALL, MAP = ______ mm

RAINFALL REGION : winter ☐ year round ☐ summer ☐ (from Fig. C1)

EXTREME POINT-RAINFALL REGION NUMBER : ______ (from Fig. C3)

VELD TYPE ZONE NUMBER : ______ (from Fig. F1)

CATCHMENT INDEX, $I_c = \frac{L\,L_c}{\sqrt{S}}$ = ______ LAG COEFFICIENT, C_t = ______ (from Table F2)

BASIN LAG, $T_L = C_t\, I_c^{0.36}$ = ______ hrs COEFFICIENT K_u = ______ (from Table F4)

1 hr UNITGRAPH PEAK, $Q_p = K_u \frac{A}{T_L}$ = ______ m^3/sec

EFFECTIVE STORM RAINFALL

RETURN PERIOD (years)	T =				T =				T =			
DURATION OF STORM (D hours)												
POINT RAINFALL, h mm (from Fig. C2 or C4)												
POINT INTENSITY (mm/hr)												
AREAL REDUCTION, a (from Fig. C6 or C7)												
RAINFALL AVERAGED OVER AREA (ah or from Figs. D2-28)												
STORM RUNOFF FACTOR (from Fig. G1 or G2)												
EFFECTIVE RAINFALL h_e												

UNITGRAPH SYNTHESIS

Time		D = 1 hr	UH	D = hrs			UH	D = hrs			UH	D = hrs			UH
t hrs	$\frac{t}{T_L}$	Q/Q_p	S curve	lagged S_D	$\Delta S = S - S_D$	$\frac{\Delta S}{D}$	$Q_p \cdot \frac{\Delta S}{D}$	lagged S_D	$\Delta S = S - S_D$	$\frac{\Delta S}{D}$	$Q_p \cdot \frac{\Delta S}{D}$	lagged S_D	$\Delta S = S - S_D$	$\frac{\Delta S}{D}$	$Q_p \cdot \frac{\Delta S}{D}$

RETURN PERIOD, years	T =				T =				T =			
DURATION OF STORM, hrs												
UH PEAK												
PEAK DISCHARGE Q_T = UH PEAK x h_e												

NOTE : figure and table numbers refer to report HRU 1/72	CALCULATED BY:	DATE: