TAIZ MUNICIPAL DEVELOPMENT AND FLOOD PROTECTION PROJECT: Phase II

HYDROLOGICAL ANALYSIS

1. Preamble.

The hydrological analysis undertaken for this Phase II study of flood risk in Taiz differs from that undertaken for Phase I largely on the basis of the methodology used. The principles, however, remain the same – that is estimating the design flood discharges in the wadis on the basis of the storm rainfall on the catchments. The recurrence interval of the flood response is assumed to be the same as the depth of the storm rainfall that caused it.

During the ten or so years since the completion of the Phase I Study little additional data has become available beyond some further daily rainfall records – which are themselves incomplete. This study therefore represents a methodological refinement of the work undertaken in Phase I and its extension to provided flood estimates for the design of the Phase II works.

2. Hydrological and Storm Rainfall Data.

The hydrological and storm rainfall data made available to the current study that are additional to those employed during Phase I (1990) permitted only very modest additions to the hydrometeorological database. It is the refinement and updating of the methodology and not the impact of additional data that explains any significant differences between the estimates of flood discharges in the wadis.

As before no direct measurements of flood discharge in the channel systems have been undertaken, though it is acknowledged that given the velocity of flow this would be difficult. Local short duration storm rainfall data are as limited as before and the small amount of additional material made available was judged to be insufficient to justify any modification to the temporal pattern of storms assumed in Phase I.

Specifically, the following data, additional to those available during Phase I, are referenced or included :-

- Daily rainfall observations at two local sites (Ossefra and Shemasi) for the years 1995 to 1999, from which events exceeding 20mm were abstracted and added to the the Phase I sample for the updated statistical analysis. This gave a total of 20 station years of daily storm observations beyond the 20mm threshold (these data are given in Appendix 1). The post 1995 daily data at the Taiz- NWRA site were not used as they were far from complete.

- Short duration (5 to 10 minute) storm intensity observations for two events at the Qurf and Miqab autographic stations during 1998 and 1999 were examined. In this Report they are only referenced. Although the total storm depths were significant :-

  1) 47mm in 60 minutes on 20/2/98, and

  2) 33mm in 30 minutes on 24/8/99

  the sample is so small that no adjustment was made to the storm profile (temporal pattern) adopted for the Phase I study, which was based on a much wider evaluation of storm rainfall in the Southern Arabian Peninsula. (Nouh. 1987)

3. Storm Rainfall Studies.

3.1 Depth – Duration – Frequency.

The following strategy was adopted to estimate the storm depth – duration – frequency (DDF) relationship :-

- A statistical analysis was undertaken of the 20 station years of daily rainfall observations assembled for a number of sites in Taiz. A necessary working assumption was that the sample, though incomplete (in that it does not represent a continuous sequence of years but a subset of the years between 1962 and 1999) and drawn from observations from several sites, can be taken to be a random, homogenous and representative. Exploratory analyses of the data indicated no evidence to challenge this assumption.

- In order to maximise the value of these data, all daily storm events with rainfall in excess of 20 mm were identified for analysis, which gave 120 such days over the 20 years. A partial duration series model (independent events over a threshold) was used to estimate the quantiles of 1 day storm risk for selected recurrence intervals. Statistical details are given in Appendix 1 and the key result reported in Table A1 below. Given the combination of additional data and a more refined statistical model these estimates are significantly greater than those reported in Phase I.

TABLE A1: TAIZ: Estimated Probability Distribution of Annual Maximum Daily Rainfall. (mm)

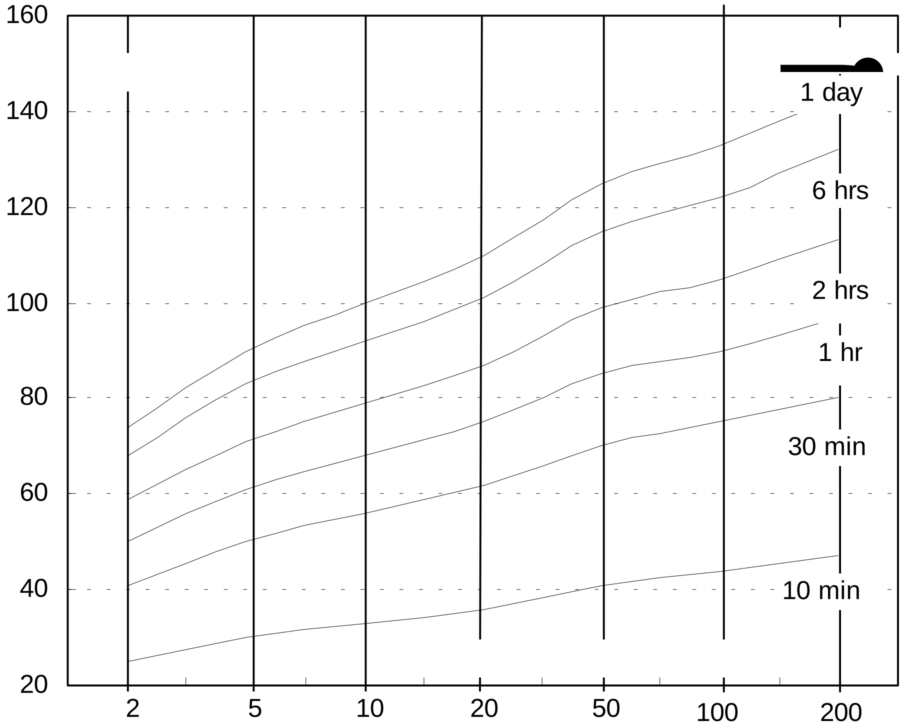
Recurrence Interval (years).						
2	5	10	20	50	100	200
74	90	100	110	125	133	143

- This Report departs significantly from the Phase I study in regard to the procedure employed to scale the 1 day storm rainfalls (for a given recurrence interval) down to shorter durations. In Phase I the ratios published by Nouh (1987) based on an extensive study of short duration storm rainfall data in SW Saudi Arabia were used. These same data were subsequently analysed by Wheater et al (1989) and it is these results that are adopted here. Although they are more conservative they were shown to be more consistent with relationships obtained for other arid regions, for example data analysed for the Gulf Sates. It should be noted that this type of ratio is generally taken to be a constant for all recurrence intervals

TABLE A2: SW SAUDI ARABIA. Ratio of the storm rainfall of duration ‘D_t’ to the 1 day depth. ‘$D_{1\ day}$’ (Source: Wheater et al.1989)

Duration (D_t)	10 min	30 min	1 hour	2 hours	6 hours	1 day
D_t / $D_{1\ day}$	0.33	0.56	0.68	0.79	0.92	1.00

- A simple scaling of the 1 day storm risk quantiles of Table A1 by the ratios of Table A2 leads to the complete DDF relationships estimated for Taiz that are given as part of Figure A1.



Recurrence Interval (years)	Duration					
	10 min	30 min	1 hour	2 hours	6 hours	1 day
2	25	41	50	59	68	74
5	30	50	61	71	83	90
10	33	56	68	79	92	100
20	36	62	75	87	100	110
50	41	70	85	99	115	125
100	44	75	90	105	122	133
200	47	80	97	113	132	143

FIGURE A1. TAIZ: Estimated Depth - Duration – Frequency (DDF) of Storm Rainfall (mm) for Selected Durations and Recurrence Intervals..

The combination of higher estimates of the 1 day storm risk and a significant modification of the scaling ratios results in storm DDF figures that are generally higher (for a given duration and recurrence interval) than those estimated during the course of Phase I. This difference is greatest for the shorter durations (< 1 hour) and for recurrence intervals below 1:20 years. This fact should be kept in mind when considering the flood estimates, given that the design standard is 1:20 years and the critical storm duration for all of the wadi systems is less than one hour.

3.2 Storm Profiles.

The temporal pattern of storm rainfall is a significant factor in determining the magnitude of the flood response and in particular the losses (to infiltration and so on) and the peak discharge. As part of the study of storms recorded by the comprehensive observation network in SW. Saudi Arabia, Nouh (1998) also considered their temporal structure. The analysis led to a classification according to the cumulative fraction of the total rainfall occurring as a function of the cumulative duration of the event.

The result is shown in Figure A2 which indicates that less than 10% of storms are characterised by a more or less constant rate of accumulated rainfall during the duration of the event (or equivalently by an approximately constant intensity). The typical pattern is one involving an initial maximum rainfall burst and a decreasing intensity over the rest of the event. For example, for 50% (or more) of storms it was observed that almost half of the total event rainfall occurred in the initial 20% of its overall duration.

The median (or 50%) rainfall pattern may be taken to be an indication of the 'characteristic' storm profile and is adopted for the study, as it was in Phase I. The two storms observed in Taiz during 1998 and 1999 indicate a much more constant intensity but the sample is far too small to challenge the conclusions drawn form a far wider regional study.

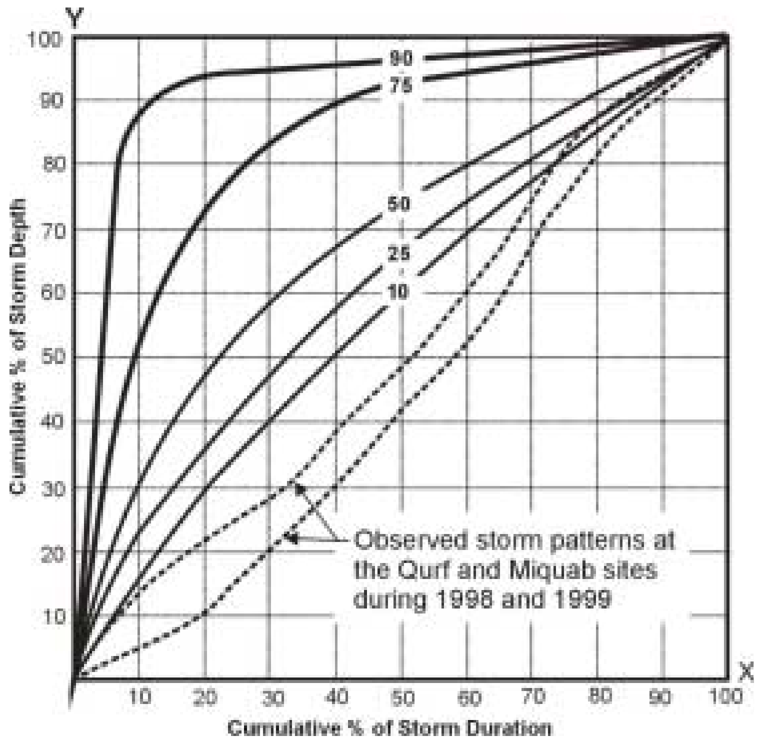


FIGURE A2. SW SAUDI ARABIA. Storm Profile Patterns (after Nouh. 1998).

3.3. Storm Rainfall. Areal Reduction Factors and Adjustments for Altitude.

Although the Phase I study considered and applied adjustments for area and altitude to derive the final estimates of storm rainfall, the basis was largely subjective. The wadi catchments are relatively small anyway and so areal adjustments of point rainfall would not be significant. Altitude is a factor but reliable quantitative data regarding its influence are not available.

It is recommended therefore that these effects be considered as part of the sensitivity analysis that should be undertaken at the design stage. The final design flood discharges are estimates to which should be attached levels of confidence. For example, the hydraulic design implications of errors of +/- 20% in the 1:20 year flows should be evaluated as a matter of course.

4. Hydrological Analysis:

4.1 Principles and Method.

The Phase I Study employed a modified Rational Type of methodology (the so-called 'B. D. Richards' model) to estimate peak discharge from catchment storm rainfall. Here a unit-hydrograph procedure is favoured for the following reasons :-

- Rational methods do not output the full hydrograph which gives key insights into the dynamics of the flood response and enable a more sound evaluation of the validity of assumptions and results. Rational methods are also relatively prescriptive and do not enable evaluations of how the flood hydrograph is modified between channel reaches. In the present study this latter issue is important.

- Unit hydrograph methods make it is far easier to undertake structured sensitivity analyses of the modelled flood response as a function of assumptions and inputs. There is a far more systematic linkage between the amount and pattern of the storm rainfall, the catchment conditions and the resulting flood runoff elements.

A key addition to this study which acknowledges hydrological / hydraulic fact is the recognition that the duration of the critical design storm is equal to the time of concentration of the catchment. The B. D. Richards model does account for this but not on the basis of generally recognised relationships between the key variables.

The time of concentration (T_c) estimator used here is :-

$$T_c = 4.096\,(L_c \cdot 10^3)^{0.23} \cdot S_{\%}^{-0.25} \cdot \zeta_{\%}^{-0.18} \cdot \Phi^{1.57}$$

where L_c is the length of the main channel in km.

$S_{\%}$ is the percent mean slope of the main channel.

$\zeta_{\%}$ is the percentage of the catchment area estimated to be impervious, and

Φ is a watershed conveyance factor ranging between 0.6 and 1.3 and is a measure of the hydraulic efficiency of the drainage network. A value of 1.3 would represent a low efficiency, highly vegetated network.

Once computed T_c is then set equal to the design storm duration for which the 1:20 year rainfall depth is interpolated from Figure A1.

Φ is fixed at 1.0 to strike a balance between terracing on the upper catchment slopes of the larger wadis, which would reduce the hydraulic efficiency of the runoff response, and the steep slopes, which would enhance it.

Attaching a value to $\zeta_{\%}$ is more problematical since there are conflicting influences on the amount of runoff generated by a given storm input over the catchments. The extremely steep slopes, particularly in the upstream parts of wadis Seena and Al Kamet (that is upstream of hydrological nodes 0 and 6 – Figure A4) and the considerable expanses of bare rock would tend to reduce infiltration (and therefore storm rainfall losses) to a minimum. On the other hand the terracing on the upper catchment slopes would have the impact of increasing storm losses.

Given the balance of these influences and the large scale urbanisation of the lower catchments it was assumed that a value of $\zeta_{\%}$ of 75 to 85% (depending on conditions upstream of a node) was realistic.

The key influence of the estimate of T_c on the overall analysis is that it defines the duration of the critical storm rainfall and therefore its depth and intensity. As reported below the sensitivity of the design flood estimates to the computed value of T_c was considered in detail.

The design storm depth for the duration T_c interpolated from Figure A1 is then distributed in time according to the (dimensionless) 50% storm profile of Figure A2. For the areas of the the Taiz catchments a 5 minute unit hydrograph was considered to be appropriate which leads to the disaggragation of the storm rainfall itself into 5 minute incremental depths.

This storm 'hyetograph' (or distribution in time of the rainfall) is then convoluted with the computed unit graph defined in terms of the flow response to 10 mm of rainfall occuring in 5 minutes over the catchment. The shape of the unit hydrograph used is given in Figure A3 below.

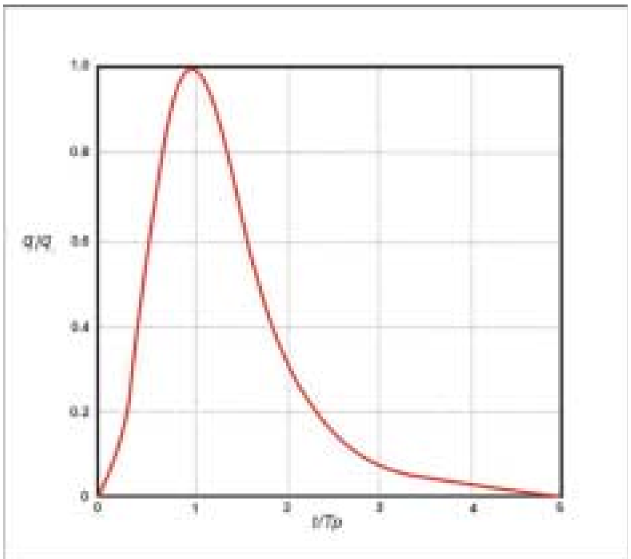


FIGURE A3: Dimensionless Unit Hydrograph indicating discharge ' q_t ' at time ' t ' as a ratio of peak discharge ' q_{max}' (y axis) and as a function of event time ' t ' as a ratio of hydrograph time to peak ' T_p ' (x axis).

This defines the standard US Soil Conservation Service (SCS) unit hydrograph in dimensionless terms, for which the definitive characteristics are :-

- The time base is approximately 5 times the time to peak.
- 38% of the discharge volume occurs prior to the peak flow.

As reported in Section 6 (Sensitivity Analyses) the shape of the unit hydrograph proved not to be a key determinant of the design flow estimates.

The full SCS procedure was concluded to be appropriate for the estimation of the flood hydrographs for Taiz wadis since conceptually it is straightforward and has minimal data requirements. Its strength lies with the fact that the relationship between storm rainfall, catchment losses and final flood runoff is defined by a single choice of ' Curve Number ' .This represents a graphical solution to the equations relating runoff response to catchment conditions and the initial and continuing processes of storm losses. Although empirical the procedure has been developed and refined using studies and data across the complete range of catchment conditions and it does have a sound physical basis.

The 'Curve' numbers (CN) range from 0 to 100 with 100 defining the condition of 100% storm runoff or zero losses. In the present application numbers between 80 and 90 were generally applied, though it is important to underscore at the outset that a CN = 85 does not imply 15% storm losses. The relationship between CN and the losses is non-linear and a function of the temporal distribution of the rainfall input and the rate of initial and ongoing catchment storage.

An example of the output from the SCS software system used to model the flood flows in the wadis is given in Appendix A2.

5. Results and Comparison with the Study of 1990.

Table A3 gives a summary of the key inputs and estimates peak discharges and flood runoff volumes for the 1:20 design flood events for each of the hydrological node points shown on Figure A4. The design storm rainfall input is the 1:20 year depth for the duration equal to T_c. The flow estimates are mapped on Figure A5.

In considering the results, specifically the flood peak discharges, the following points should be kept in mind :-

- The peak flows are large when considered in terms of specific or unit area discharge. The estimates all exceed a peak rate considerably in excess of 10 cumecs / square kilometre. One of the principal factors determining this is the design storm rainfall input which, for the short durations considered (in terms of T_c.), represents extreme intensities of over 150mm / hour. However, at a probability of occurrence of 1:20 years this is not considered to be unrealistic.

- The sub-catchment estimates of T_c. itself indicate a very rapid accumulation of flood runoff of +/- 20 minutes for the larger sub-basins. Given the extreme slopes, shallow soils, limited vegetation cover and the general imperviousness of the land surface combined with the intensity of the design storm conditions then a fast runoff response would be expected. It is accepted that the catchment response times might be slower under less extreme storm conditions.

- Counterbalancing these factors that maximise the peak runoff rate is the effect of the adopted storm profile. Under design conditions it has been assumed that almost half of the total event rainfall occurs in the initial 20% of total storm duration. It is during these first stages that losses (to infiltration) are at a maximum so overall the percentage runoff is somewhat reduced since most rainfall is assumed to occur when abstractions will be highest.

- This effect combined with the setting of the CN in the range 85 – 90 results in an effective rainfall (that is the fraction that directly generates the flood response) of 50 to 60%. For the topographic conditions that prevail in the upper reaches of the larger wadis and the fact that their middle and lower catchments are almost completely urbanised, these figures may appear to be too low. However, strictly speaking the SCS procedure, like most single event flood models, only considers the 'fast response' to storm rainfall and most of the so called losses only appear later as part of the longer recession once the core hydrograph has occured. This core is the focus of design interest and only has a total time base that is approximately 5 times the time to peak., which for the wadi systems is very rapid. In this respect the figures are perceived to be in

TABLE A3 : TAIZ. Estimated 1:20 year design flood discharges at system nodes, given catchment and critical storm data as shown.

Node	Catchment Data				Design StormRainfall mm	Flood Runoff	
	A km^2	L km	S %	T_c min		Q_{max} cumecs	Vol $m^3.10^3$
0	5.4	4.8	20	20	50	94	146
1	0.7	1.6	48	12	39	14	13
2	0.4	0.8	17	14	41	7	7
3	0.6	0.9	57	10	36	10	7
4A	0.25	0.9	60	10	36	4	3
4B	0.16	0.6	54	10	36	2.7	2
5	0.37	1.2	46	12	39	7.2	6
6	7.0	4.7	23	20	50	124	190
7	8.1	6.0	20	22	53	152	240
8	3.6	2.0	44	13	40	70	67
10A	0.50	1.2	50	11	38	9.2	9
10B	0.40	1.4	45	13	40	7.8	7
10	1.0	1.5	45	13	40	19	18
11A	1.1	1.8	35	15	43	20	23
11B	0.8	1.4	50	12	39	14	14
11	1.9	1.8	35	15	43	33	40
13	3.9	2.5	40	15	43	72	82
14	7.2	7.5	18	22	53	133	213
A12	1.1	1	10	16	44	19	23
A13	20.5	7.8	45	20	50	270	390
A8	0.29	1.5		6	28	5	
A9	0.1	0.1	20	8	32	2	1
A10	0.1	0.1	20	8	32	2	1
A7	0.35	1.75		7.3	31	6.4	
A6	0.36	1.94		8	33	6.9	
A4	0.42	1.0		4.2	22	5.8	
A5	1.66	2.7		15	38	24.1	
A3	1.78	3.2		19.7	41	25	
A2	21.5	8.8	20	23	55	315	500
A1	23	8.8	20	23	55	325	530
A14	2.41	3.7		20	51	43.6	

accordance with physical reality in that they represent the fraction of storm rainfall that contributes to the 'fast flood response'.

- The contribution of the middle and lower urban reaches to the major peak flows entering the system, specifically at nodes 0, 1, 2, …..,5 and 6, is not clear. At present it is understood that the flood flow to the wadis through the town is mostly on the road surface and so paths that the water takes is dictated by the street layout. It may therefore reach the wadis after the principal peak from upstream has passed and so contribute little to it. On the other hand part of the planned drainage infrastructure is to control the urban runoff fraction and divert it to a proper system of drains, which would reduce its arrival time at the wadi channel. Another issue to be considered in terms of its effect upon the design discharge and how it accumulates downstream is whether the design storm is centred over the city or over the upper catchment systems. It is assumed here that rainfall is equally distributed over both and that the point storm intensity is the same for all sub-catchments.

- Given these factors and uncertainties it was concluded that it would not be sound to assume the same runoff conditions in the middle and lower reaches that were accepted for upstream. That is it was assumed that the contribution to peak discharge would reduce in the downstream direction and not linearly reflect the systematic increase in catchment area. This was achieved in terms of the modelling strategy by reducing the CN value to 80.

- Another key issue controlling the potential increase in the peak discharge as the flood flows proceed downstream is the number of junctions in the system and how the flows are assumed to converge under design conditions. The spatial distribution and timing of the design storm rainfall is a key hydrological factor in addition to the physical and hydraulic conditions in the channels. It was concluded that it would be unrealistic simply to sum the sub-system peaks at each junction. The tactic adopted was to 'lump' the subsystems at each junction into a single catchment and then make a separate estimate of the flood hydrograph, adjusting the input variables (specifically the CN, T_c and design storm rainfall) to account for the different conditions within this larger 'lumped' catchment. This procedure usually produced flows less than those that would be obtained by summing the sub-system design flows.

Table A4 (below) presents a comparison of the design flood discharge estimates with those given in the 1990 Study at the key upstream nodes in addition to some brief comments on the difference.

TABLE A4: TAIZ. Comparison of 1:20 year design flood estimates at key hydrological node points reported in the Phase I Study (1990) and those reported in Table A3 (above).

<table>
<tr><th rowspan="2">Node</th><th colspan="2">Design Flood Estimate</th><th rowspan="2">Comments</th></tr>
<tr><th>1990 Study</th><th>This Study</th></tr>
<tr><td>0</td><td>78</td><td>94</td><td>The higher estimate in this study is the result of a combination of a shorter estimate of T_c.and a higher design storm rainfall from the updated DDF analysis.</td></tr>
<tr><td>1
2
3
4A
4B
5</td><td>11
18
19
8
6
10</td><td>7
14
10
4
3
7</td><td>The sub-basins at each of these nodes are >>> 1 km^2 in area for which the Modified Rational Procedure used in the 1990 Study could be inappropriate. In particular the DDF relationships for storm rainfall are approximated using a simple numerical function which would not be very accurate for durations >> 10 minutes. Additionally, losses in these smaller sub-basins are set to be higher in this study.</td></tr>
<tr><td>6</td><td>98</td><td>124</td><td>As for Node 0.</td></tr>
</table>

6. Sensitivity Analyses.

The deficiencies in the data available to the hydrological component of the study mean that the design flood estimates have to be based on assumptions that are considered reasonable and realistic. For example, the amount of surface runoff generated by a given storm input (defined by the selection of the CN) and the computation of T_c. (which defines the critical storm duration and therefore the storm depth) are both based on judgements of the hydrological response of the catchments. Such assumptions cannot be verified.

On the other hand, the DDF relationships for short duration storm rainfall that are reported in Figure A1 can be confirmed in the context of wider regional results, for example those that apply in SW. Saudi Arabia.

The impact of potential error in the inputs to the SCS flood estimation procedure may be summarised as follows :-

- For an arbitary 1 km^2 sub-catchment in the Taiz system with T_c = 15 minutes and a design (1:20 year) storm rainfall of 43 mm, increasing the CN from 80 to 90 doubles the peak discharge from 9 to 18 cumecs, underscoring the sensitivity of the whole procedure to the selection of the CN value. Correspondingly the ‘ fast ‘ flood runoff increases from 25 to 50% of the storm rainfall input. Generally CN values have been set within the range of 85 to 90 in the study.

- The sensitivity of the estimates of peak flow to other input changes is modest in comparison. Changing the storm profile from one where most of the rainfall occurs in the initial stages of the storm to one for which the rainfall is evenly distributed in time has only a minor impact on the computed maximum discharge. Adjustments to the dimensionless form of the unit hydrograph also have a minor influence compared to the sensitivity of the result to the selection of the CN value.

- The estimate of the depth and duration (which is equal to the value of T_c) of the storm rainfall for each sub-catchment is clearly a key influence on the result. For a given CN increasing rainfall will lead to a corresponding increase in the maximum flow, bearing in mind that the total amount of losses will remain the same. Roughly speaking a 10% increase in rainfall input (for a given duration) will generate an increase of between 13 and 15% in peak discharge for CN values in the range 80 to 90..

Given this sensitivity of the design discharge estimates to key inputs, which have in the main to be assumed on the basis of the hydrological conditions that apply in the Taiz wadi system, it is important to attach some level of confidence to the results presented here. It is proposed that a margin of +/- 20% would be appropriate as a basis for evaluating the implications of hydrological error on the hydraulic modelling studies and in turn on the structural design and capacity of the drainage network.

7. Conclusions.

Hydrological conditions in the Taiz wadi system are extreme. Intense storm rainfall intensity, precipitous slopes, the virtual lack of natural catchment storage that would attenuate the flood response and the rapid expansion of urbanisation (and therefore of impervious surfaces) combine to ensure a flood regime that is characteristically violent and torrential. Estimating a peak discharge with a given frequency of occurrence in such a situation would be difficult even with a reasonable database and a modelling methodology developed and proven for such conditions. The latter does not exist in any practical sense, so classical procedures (such as that applied here) have to be adopted and the required inputs based on sound assumptions. The only strictly data based aspect of this study is the estimation of the depth – duration and frequency of storm rainfall.

The hydrological response variables that are proposed are not overtly conservative in that there was an intention to define an upper limit to the design discharges. The estimates are considered to be realistic given the conditions. However, they have a potential error attached to them and given all of the 'unknowns' it has been recommended that the confidence interval be set at a relatively wide +/- 20% and sensitivity analyses be undertaken accordingly.

These sensitivity analyses should consider the impact of the possible error margin on the hydraulic and structural design criteria and the cost implications. Feedback is an important part of this process. For example, are the hydraulic conditions in the wadi channels feasible in terms of depth and velocity given the design discharge? If the required velocities are unrealistically high then this may be taken to be evidence of a possible overestimation of the design hydrology.

An important issue that should be considered is the contribution of the urban fraction of the catchment system to the design discharges. This may be defined as the portion downstream of hydrological nodes 0 to 6 as shown on Figure A4. It could be argued that the probability of the simultaneous arrival of the maximum runoff from the upstream and urban components in these middle and downstream reaches is so small that it can be discounted. It would therefore only be necessary to rout the flows from Nodes 0 to 6 down the system. In effect this strategy would indicate a lower bound to flood discharge in the lower channel network. The urban contribution would in all probability be only a relatively minor fraction of the maximum discharge arriving from the mountainous upstream reaches. In this study contributing sub-system maxima have not simply been superimposed at channel junctions but the whole upstream system modelled as a single 'lumped' catchment, with inputs adjusted accordingly. Even this strategy indicates a considerable increase in the peak discharge as the potential contributing area increases down stream.

This may not be the case to the extent that the results reported here might indicate. As proposed an alternative scenario is one where the urban fraction arrives after the upstream peak has passed and that only a relatively minor hydrological adjustment needs to be made as the flood wave passes through the city in the major channel system. The concluding point is therefore that the flood peak discharges estimated and presented here are in no way definitive and should be perceived as a sound basis for sensitivity analyses and modification on the basis of ' on – site 'hydraulic assessments.

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WHEATER. H., LARENTIS. P and G. S. HAMILTON. (1989). "Design Rainfall Characteristics for SW. Saudi Arabia.". Proc Inst Civ Eng. (London) Series 2. Vol 87. December.

DAILY RAINFALL OBSERVATIONS EXCEEDING 20mm FOR A NUMBER OF SITES IN TAIZ AND FOR AN INCOMPLETE SAMPLE OF YEARS BETWEEN 1962 AND 1999.

Table A1

Year	Sample Size	Xi > 20mm												
1962	5	60	54	38	38	24								
1963	12	64	43	35	33	32	29	26	25	25	24	21	21	
1974	11	60	57	42	30	29	25	24	23	21	21	21		
1976	9	37	35	35	32	30	29	27	24	23				
1977	10	63	50	42	40	39	34	31	25	24	21			
1978	13	72	38	35	35	34	28	28	26	24	23	23	21	21
1979	6	70	38	32	32	31	22							
1980	12	56	50	49	48	35	34	31	30	23	22	21	21	
1981	9	38	35	33	32	30	28	28	24	23				
1982	12	46	37	35	32	31	30	28	24	22	22	22	21	
1984	5	60	59	32	25	21								
1985	4	60	43	27	22									
1986	6	62	40	32	28	22	21							
1987	7	58	52	50	40	30	23	22						
1988	3	25	25	25										
1995	8	79	43	38	34	27	24	23	22					
1996	7	88	35	34	29	29	27	25						
1997	7	56	42	27	25	21	21	21						
1998	6	75	72	35	25	22	22							
1999	8	60	50	30	28	22	21	21	21					

Annual Maximum

STATISTICAL MODELLING OF DAILY RAINFALL DATA.

The distribution of the annual maximum daily rainfall for selected recurrence intervals given in Table A1 of the main text was estimated using the Generalised Pareto Distribution (GPD) which is an approximating model for the probability distribution of excesses over a given threshold. In inverse form the GPD is defined by :-

$$X(F) = \alpha \{ 1 - (1 - F)^{k} \} / k \qquad k \neq 0$$

and

$$X(F) = -\alpha \ln.(1 - F) \qquad k = 0$$

where "α" is a scale parameter and "k" a shape parameter which can be estimated using Probability Weighted Moments. F is the exceedance probability.

The relationship between the distribution of excesses over the threshold "t" and the distribution of the annual maxima is :-

$$z(F) = t + x_t [1 + \lambda(t)\text{-}1. \ln F] \qquad e^{-\lambda(t)} < F < 1$$

where z(.) are the quantiles of the distribution of the annual maxima, x_t (.) are the quantiles of the distribution of the escesses of the threshold "t" and $\lambda(t)$ is the mean of a Poisson model describing the distribution of the number of annual excesses of "t".

Setting "t" = 20mm and using the data in Table A1 above the following results are obtained :-

TAIZ INPUT

20 NUMBER OF YEARS OF DAILY DATA.
20. THRESHOLD RAINFALL. (mm).
160 NUMBER OF INDEPENDENT STORMS OVER THRESHOLD. (EXCESSES).

RANK ORDERED DATA.

21	21	21	21	21	21	21	21	21	21
21	21	21	21	21	21	21	21	21	22
22	22	22	22	22	22	22	22	22	22
22	23	23	23	23	23	23	23	23	24
24	24	24	24	24	24	24	24	25	25
25	25	25	25	25	25	25	25	25	26
26	27	27	27	27	27	28	28	28	28
28	28	28	29	29	29	29	29	30	30
30	30	30	30	30	31	31	31	31	32
32	32	32	32	32	32	32	33	33	34
34	34	34	34	35	35	35	35	35	35
35	35	35	35	37	37	38	38	38	38
38	38	39	40	40	40	42	42	42	43
43	43	46	48	49	50	50	50	50	52
54	56	56	57	58	60	60	60	60	60
60	62	63	64	70	72	72	75	79	88

F(%)	95	90	80	70	60	50	40	30	20	10	5	2	1	0 .5
x(F)	21	21	23	25	27	30	33	37	43	52	62	75	84	94
z(F)	54	57	62	67	70	74	78	84	90	101	111	124	133	143

GPD PARAMETERS. α = 14.00 k= .001
POISSON PARAMETER: $\lambda(t)$ = 8

SAMPLE OUTPUT FROM SCS FLOOD RUNOFF MODEL.

Taiz : Node A12.

BASIC INPUT VARIABLES.

AREA sq km	MAIN CHANNEL LENGTH km	SLOPE %	RAINFALL DEPTH (mm)	RAINFALL DURATION (minutes)
1	1	10.	10	5

% WATERSHED IMPERVIOUS	WATERSHED CONVEYANCE FACTOR
85	1.00

FLOW RESPONSE TIMES, DEPTH, VOLUME & PEAKFLOW.

TIME OF CONCN(min)	LAGTIME (min)	TIME TO PEAK (min)	RUNOFF (mm)	RUNOFF (tcm)	PEAKFLOW (cumecs)
16	9	12	10	11	11.3

THE 5 MINUTE UNIT HYDROGRAPH WITH TIME ORDINATE AT 5 MINUTE INTERVALS.

ORDINATE No	TIME (min)	Q (cumecs)	VOLUME (tcm)
1	0	.00	.00
2	5	3.62	.55
3	10	10.78	2.73
4	15	10.25	5.92
5	20	5.74	8.35
6	25	2.88	9.66
7	30	1.47	10.32
8	35	.75	10.66
9	40	.39	10.83
10	45	.20	10.92
11	50	.10	10.97
12	55	.05	10.99
13	60	.01	11.00
14	65	.00	11.00

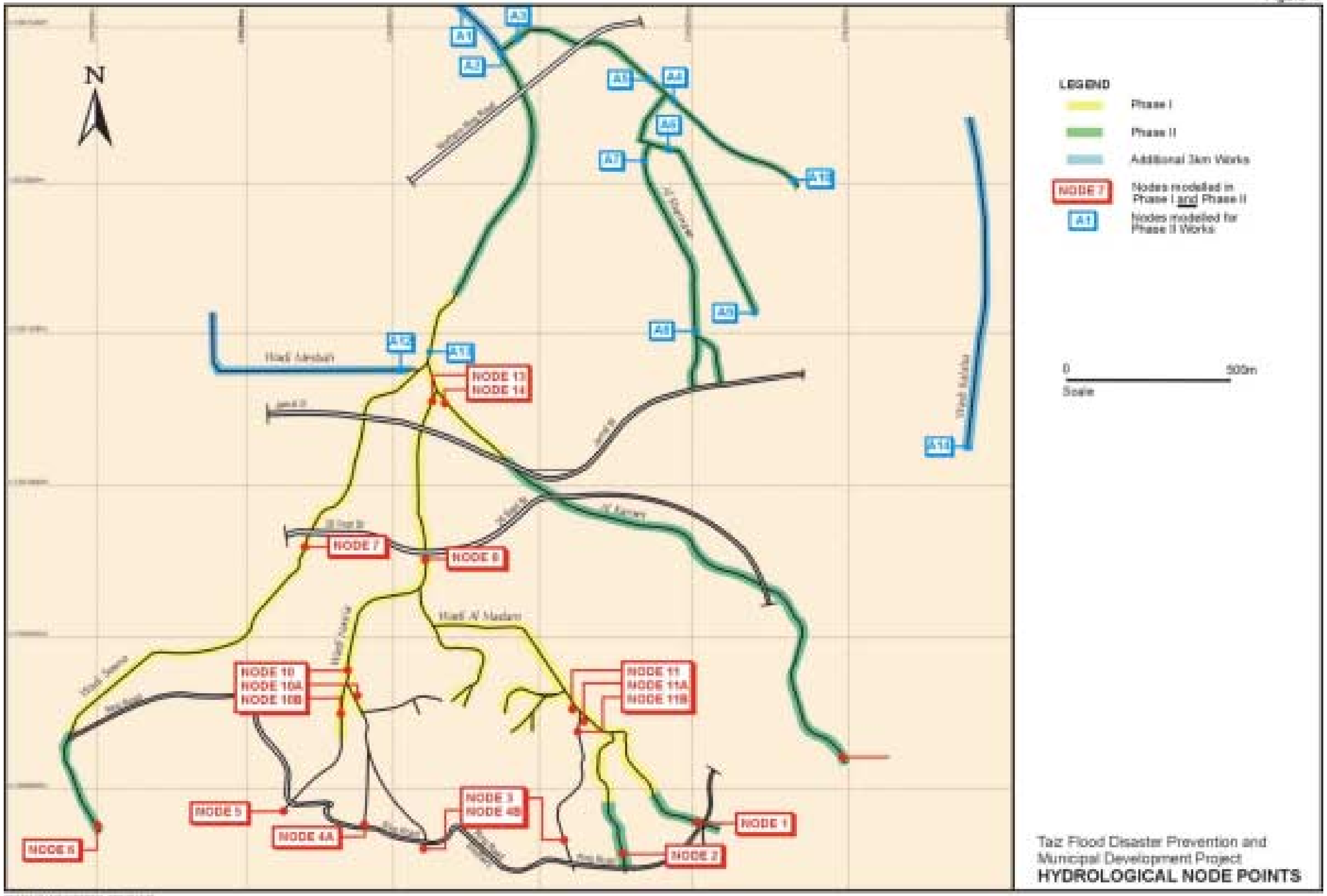
COMPUTATION OF EFFECTIVE STORM RAINFALL PROFILE
GIVEN :- i) SCS Curve No = 90
ii) NOUH 50%.

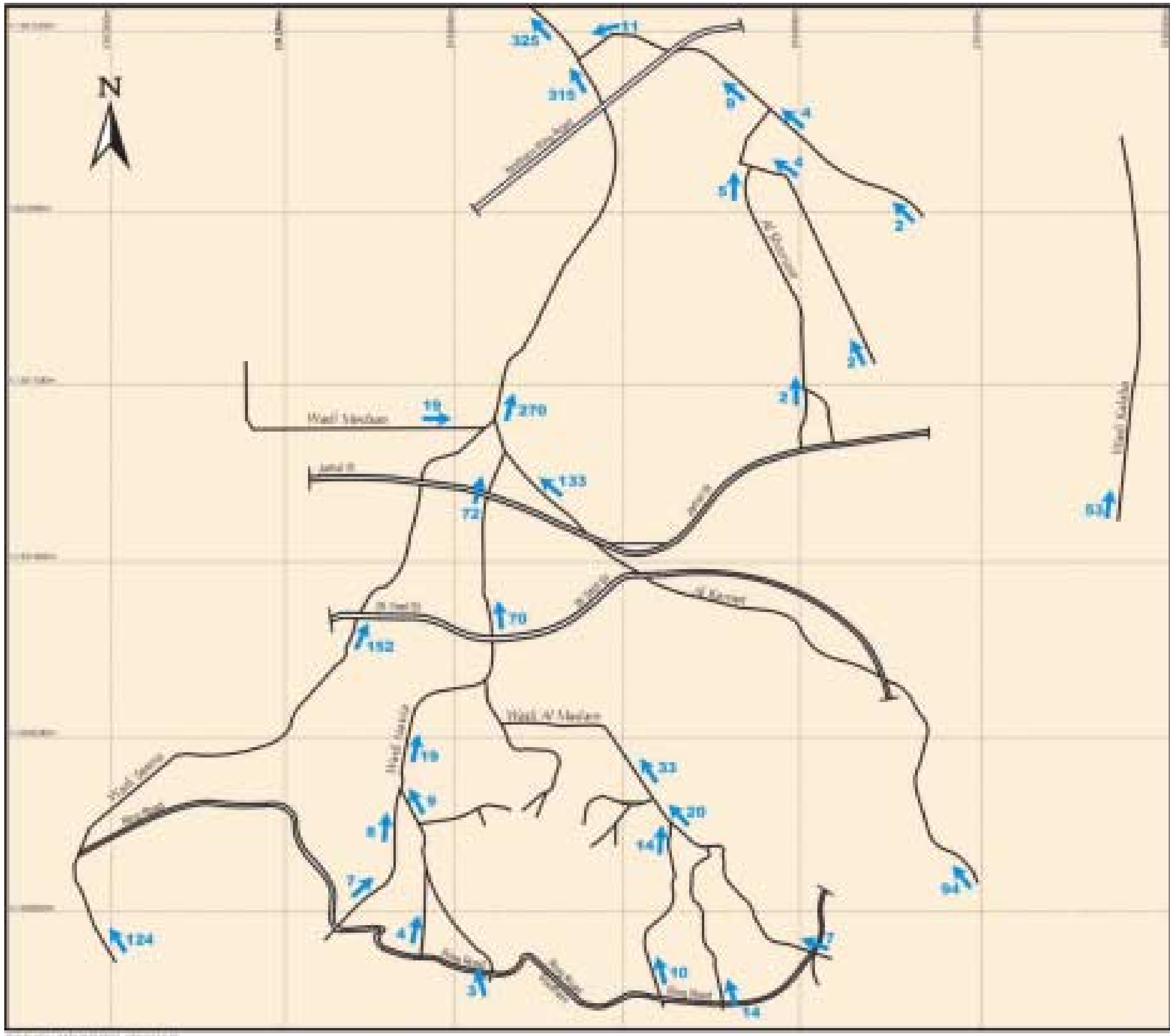
TIME minutes	RAINFALL (mm)	LOSSES (mm)	EFFECTIVE RAINFALL (mm)
5	25	17	8
10	11	3	8
15	8	2	6
TOTALS	44	22	22

CATCHMENT FLOOD RESPONSE HYDROGRAPH.

Time minutes	Cumulative Effective Rainfall (mm)	UH	Response Discharge (cumecs)	Cumulative Volume of Runoff (tcm)	Cumulative Depth of Runoff (mm)
0	0	.0	.0	0	0
5	7	3.6	.0	0	0
10	15	10.8	2.9	0	0
15	22	10.3	11.4	2	2
20		5.7	18.9	7	6
25		2.9	19.4	12	12
30		1.5	13.3	17	16
35		.8	7.1	20	19
40		.4	3.6	22	20
45		.2	1.8	23	21
50		.1	.9	23	21
55		.1	.5	23	22
60		.0	.2	23	22
65			.1	23	21
70			.0	24	21
75			.0	24	21
80			.0	24	21
85			.0	24	21

Figure 4


N
LEGEND
Phase I
Phase II
Additional 3km Works
NODE 7
Nodes modelled in Phase I and Phase II
A1
Nodes modelled for Phase II Works
0
500m
Scale
NODE 13
NODE 14
NODE 7
NODE 8
Wadi Al Madam
NODE 10
NODE 10A
NODE 10B
NODE 11
NODE 11A
NODE 11B
NODE 5
NODE 3
NODE 4B
NODE 4A
NODE 1
NODE 2
NODE 6
Taiz Flood Disaster Prevention and
Municipal Development Project
HYDROLOGICAL NODE POINTS



LEGEND

53 1:20 year design flood discharge

Taiz Flood Disaster Prevention and Municipal Development Project

1:20 YEAR DESIGN FLOOD DISCHARGE (cumecs)