KENNEDY GROVE HOUSING SCHEME HYDROLOGICAL ASSESSMENT

Prepared by: Water Resources Authority
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EXECUTIVE SUMMARY

The Kennedy Grove Housing Scheme a Joint Venture Project between the then Ministry of Environment and Housing and KID Development Company Limited is located in the Palmers Cross area, Clarendon. This scheme developed in the 1990's with approximately 200 houses has experienced flooding on several occasions. Sections of the scheme are located within a natural depression, which receives surface runoff from surrounding areas. The capacity of the sinkhole is exceeded during heavy rainfall causing water levels to rise and threatening housing units.

The Water Resources Authority has undertaken a hydrological and hydraulic assessment for this housing scheme based on a request by the Ministry of Water and Housing. Four intervention measures were identified and this assessment seeks to evaluate the impacts of these intervention measures namely

1) leaving the depression undisturbed (do nothing intervention)
2) using three soak-aways to provide additional drainage
3) using two soak-aways and an improved sinkhole and
4) cutting a channel to drain floodwaters out of the scheme.

The intervention measures were investigated with a view to determining maximum water levels and duration of flood water levels being above the critical topographic elevation.

It is clear that the 'no intervention' scenario causes the highest water levels and it takes nearly three weeks for water levels to recede below the critical level.

The interventions 2 and 3 which looked at a) three soak-aways and b) improved sinkhole and two soak-aways revealed that although water levels still approach the maximum water levels as experienced during the rainfall event associated with Hurricane Wilma the time it takes for the pond water level to recede is reduced by 15 days.

The intervention 4 involving the cutting of a proposed channel to discharge floodwaters out of the scheme towards the Rio Minho was modeled. The results showed that water levels could be maintained at 98 m amsl, representing the base of the house at the lowest elevation. The design flow of the channel for maintaining this water level is 3.4 m^3/s simulated from the 100-year rainfall. The results of the backwater assessment show no significant impacts on the Rio Minho and its tributary even under the Probable Maximum Precipitation (PMP) scenario.

Interventions 1 –3 producing roughly the same results with respect to water levels would necessitate the relocation of at least 78 houses for rainfall events with a return period greater than 50 years. The sewage infrastructure will have to be redesigned, as the sewage lift pump needs to be relocated to a higher elevation. The main entrance to the Kennedy Grove scheme becomes inundated at water levels greater than 101.76 m amsl. An alternative access road needs to be established either to the west or the east of the main entrance.

Intervention 4 indicates that drainage of the floodwaters via the proposed channel is the most effective option for reducing the flood levels in the scheme. This channel should be designed to discharge peak flow of 3.4 m^3/s corresponding to the 100-year rainfall. It has to be considered that this will necessitate extensive cutting through rock material over a length of approximately 1.5 km.

A decision for the most suitable intervention has to be based on a socio-economic assessment.

SUMMARY

Four intervention scenarios were investigated with a view to determining maximum water levels and duration of flood water levels being above the critical topographic elevation. It is clear that the ‘no intervention’ scenario causes the highest water levels and it takes nearly three weeks for water levels to recede below the critical level. The intervention 2 and 3 which looked at a) three soakaways and b) improved sinkhole and two soakaways revealed that although water levels still approach the maximum water levels as experienced during the rainfall event associated with Hurricane Wilma the time it takes for the pond water level to recede is reduced by 15 days. The intervention involving the cutting of a proposed channel to discharge floodwaters out of the scheme towards the Rio Minho was modeled. The results showed that water levels could be maintained at 98 m amsl, representing the base of the house at the lowest elevation. The design flow of the channel for maintaining this water level is 3.4 m^3/s simulated from the 100 year rainfall. The results of the backwater assessment show no significant impacts on the Rio Minho and its tributary even under the Probable Maximum Precipitation (PMP) scenario. Interventions 1 –3 producing roughly the same results with respect to water levels would necessitate the relocation of at least 78 houses for rainfall events with a return period greater than 50 years. The sewage infrastructure will have to be redesigned as the sewage lift pump needs to be relocated to a higher elevation. The main entrance to the Kennedy Grove scheme becomes inundated at water levels greater than 101.76 m amsl. An alternative access road needs to be established either to the west or the east of the main entrance.

Intervention 4 indicates that drainage of the floodwaters via the proposed channel is the most effective option for reducing the flood levels in the scheme. This channel should be designed to discharge peak flow of 3.4 m^3/s corresponding to the 100-year rainfall. It has to be considered that this will necessitate extensive cutting through rock material over a length of approximately 1.5 km.

BACKGROUND

The Kennedy Grove Housing Scheme is a Joint Venture Housing Project between the then Ministry of Environment and Housing and KID Development Company Limited.
It is located in the Palmers Cross area in Clarendon approximately 1.8 km north of the main road leading to May Pen. This scheme has been developed in the mid 90s and approximately 200 houses have been constructed. A natural depression is located at the southern section of the scheme. This depression serves as a receptor for surface runoff and a sinkhole at the bottom of this depression allows for the percolation of surface runoff into the underlying limestone aquifer. During heavy rainfall events the capacity of the sinkhole is exceeded and the water levels in the pond rise and threaten housing units constructed within the flood levels of the pond. The Kennedy Grove main entrance serves as an outlet where the storage capacity of the depression is exceeded.
Since its completion, the development was impacted by flooding in May 2002, September 2004 and July and October 2005 posing a threat to the life and property of the residents.

In the case of the May 2002 floods, the records indicate that four (4) houses were partially submerged and floodwater rose on roadways to a level where access to four (4) additional houses was impeded. During flood rains of October 14 -17, 2005, approximately thirty-six (36) houses were affected. Four (4) of these were almost completely submerged, sixteen (16) houses were flooded up to a depth of 450mm (1ft-6 inches) above floor level and another sixteen (16) houses were restricted to access because of the flooded roadways. Compounding the flooding problem was a health hazard posed by the contamination of the floodwaters with raw sewage from the inundated sewage sump located in the depression.

Based on field reconnaissance carried out by the WRA in November of 2005, a number of flood mitigation options were proposed in a note to Cabinet. The recommendations were that a technical feasibility assessment of each option be undertaken so as to inform the decision for the best option or combination of options.

Mitigation Options

The mitigation options are:

1. Leave the depression area undisturbed, allowing natural drainage to take place.

2. Increase drainage to groundwater using soakaway(s) dug to the west of the pond.

3. Clean and rehabilitate sinkhole

4. Cut channel from the pond to convey water to the Chateau Gully and further to the Rio Minho river

Objective of Study

The objectives of the study are:

i. definition of the drainage area contributing to flows into the depression located within the Kennedy Grove Housing scheme
ii. to determine the floodwater elevations of the depressions corresponding to the 10-, 25-, 50-, and 100-year rainfall
iii. determination of the volume of water in storage for the areas of inundation determined under ii.
iv. Determination of the peak inflow rate into the depression
v. Determination of the outflow rate (percolation through the sinkhole) and the time to dewater the depression
vi. Determine the backwater effect of the Rio Minho at high flow on the gully transporting the outflow from the proposed drain from Kennedy Grove

APPROACH

Interventions 1 –3 were assessed by hydrologic analysis and intervention 4 by both hydrologic and hydraulic analysis. Under intervention 1 (*do nothing option*) the water levels for the various return periods were determined and the impacts assessed. Under interventions 2 – 4 the new flood levels were determined and changes in impacts assessed.

The hydrologic analysis was carried out using the HEC-HMS model developed by the US Army Corps of Engineers (USACE). Flows from the sub-basins draining to the depression were simulated and the depression was modeled using the storage-area-elevation technique.

The calibration and verification was done using rainfall associated with Dennis, Emily and Wilma. The calibrated model was used to determine flood levels in the depression and flows into the proposed channel to the Chateau-Rio Minho gully course.

The possible negative impact of the diversion was assessed using the HEC-RAS hydraulic model developed by the USACE to determine the water surface elevations corresponding to the designated flows.

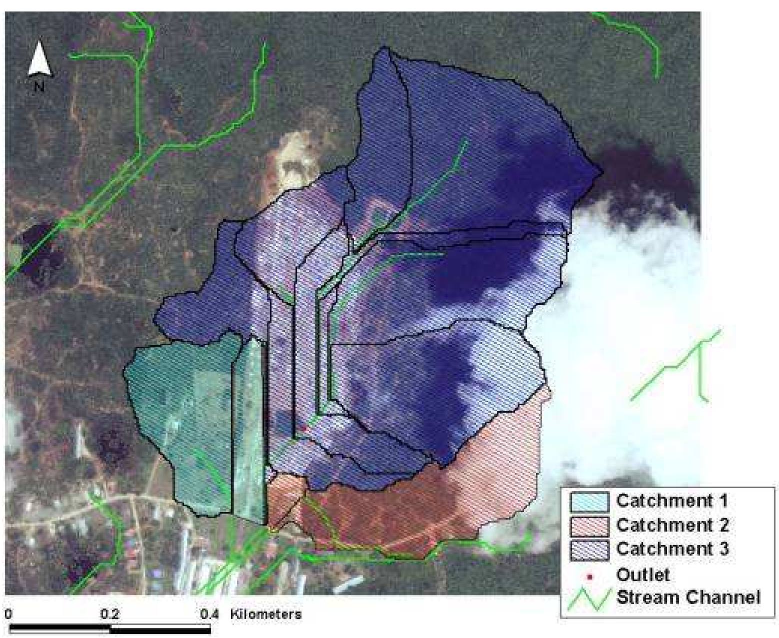
METHODOLOGY

Hydrologic Analysis

Drainage Area Determination

The drainage network was delineated from the contours associated with the Digital Surface Model (DSM) derived from the IKONOS images. The GeoHEC-HMS extension in ArcViewGis allows for the design of the river network and by choosing strategic basin outlets the software delineates the sub-basins contributing to flows at these particular outlets. The first outlet was placed on the main road approximately 150 m west of the Kennedy Grove main entrance. The contour information indicates that surface runoff from this catchment (# 1) flows along the main road and enters the scheme through the entrance. This sub-basin has a size of approximately 79,100 m^2 . The second outlet was placed south of the pond. This allows for the delineation of a catchment (# 3), which represents areas to the north of the scheme, the scheme itself and areas to the east. This sub-basin has the largest size with 360,797 m^2 . The third outlet was again placed on the main road as the contour information indicates that flows from the southeastern areas (catchment # 2) are channeled along the main road and enter the scheme through the entrance. The size of this sub-basin is 77,280 m^2. Surface runoff from sub-basins/catchments 1 and 2 are normally conveyed through a small drain leading from the main road to the south into neighbouring depressions. The access point to this drain is however blocked forcing water from catchment # 1 and 2 into the scheme.

Figure 1: Sub Catchments Kennedy Grove, Clarendon



Determination of Elevation vs Area-Storage Relationship of the Depression

The pond storage area was determined using ArcView GIS and the spot heights provided by the consulting engineer Mr. Poorman. Outflow from the scheme occurs at an elevation of 101.76 m amsl as determined by field surveys (Figure 1). The elevation- storage-area is as follows.

Table 1: Elevation-Storage-Area of Pond in Kennedy Grove Scheme

Elevation	Area	Storage	Elevation	Area	Storage	Elevation	Area	Storage
m amsl	m^2	m^3	m amsl	m^2	m^3	m amsl	m^2	m^3
96	0	0	98	3711	2907	100	12767	18793
96.25	330	44	98.25	5207	4196	100.25	17337	22706
96.5	458	141	98.5	5809	5566	100.5	18817	27216
96.75	653	284	98.75	6467	7092	100.75	20974	32240
97	1000	469	99	7270	8718	101	22631	37626
97.25	1764	859	99.25	9023	10911	101.25	28516	44346
97.5	2082	1332	99.5	9953	13271	101.5	32974	51949
97.75	3238	2053	99.75	10981	15866	101.75	40914	61103

For flood elevations greater than 101.76 m amsl, water flows from the depression and across the road through two distinct outlets to areas outside the scheme. One is a series of small openings in a wall located on the opposite side of the scheme leading to a drain and the second outlet is through a gate belonging to a private property opposite of the Kennedy Grove scheme entrance.

The survey data did not cover the entire study area and the DSM contours were used to fill the gap. The surveyed spot height elevations were compared with the contours derived from the Digital Surface Model. A total of 105 points were surveyed. Ten randomly selected points (10 % of surveyed total) were compared with the 1 m DSM contours. The following table provides information on the differences between surveyed points and the DSM contours.

Table 2: Surveyed Spot Height Elevations vs. DSM Contours

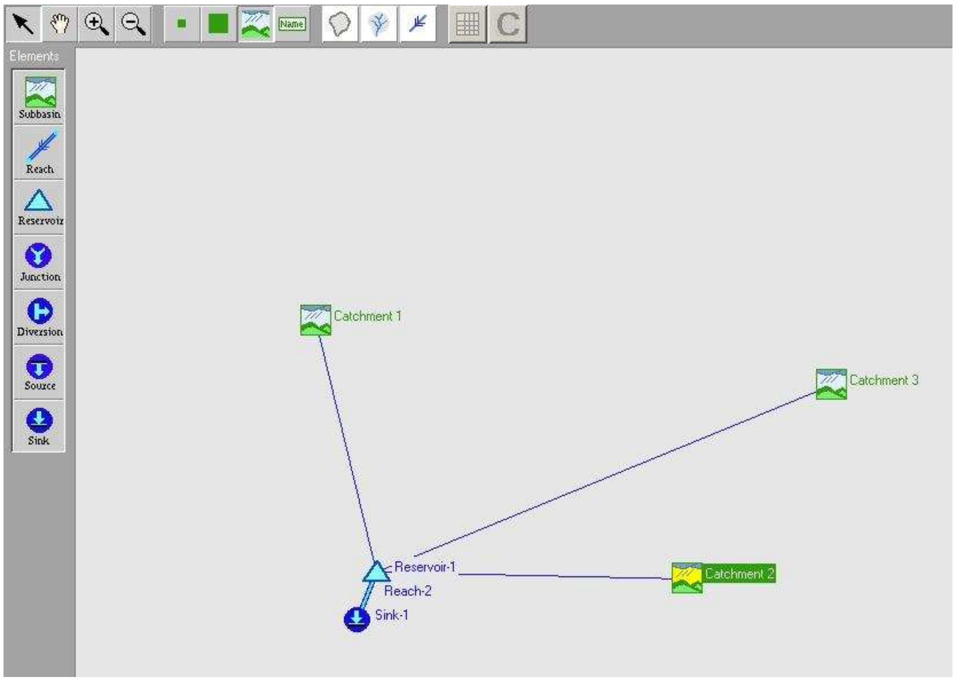
surveyed elevation	elevation based on DSM *	difference in %	
102.26	103.10	0.82	
102.22	102.95	0.71	
100.21	101.05	0.84	
100.98	101.50	0.51	
101.92	102.05	0.13	
101.60	101.80	0.20	
101.56	101.70	0.14	
100.98	101.60	0.61	
97.98	99.60	1.65	
98.07	98.50	0.44	
102.30	101.95	-0.34	
		0.52	mean difference
*(interpolated between 1 m contour line)			

The small difference between the surveyed spot heights and the DSM contours justify the use of the DSM contour for the remaining area.

Model Development for the Drainage Area

From the drainage area determination three major sub-basins were identified as contributing flows into the depression. Figure 2 shows the schematic of the model

Figure 2: Model Schematic



Inflow

For each of the catchments (1, 2 and 3) in the above schematic flows were generated using the SCS rainfall-runoff relation that relates accumulated rainfall depth (P), soil moisture storage deficit (S) and the accumulated runoff (Q_d) through equation:

$$Q_d = \frac{(P - 0.2S)^2}{P + 0.8S} \text{ (Equation 1)}$$

The soil moisture deficit is related to the CN or Curve Number. This is an index of runoff and is a function of the drainage characteristic of the soil group, the land use, soil cover and antecedent moisture conditions.

$$S = \frac{1000}{CN} - 10 \text{ (Equation 2)}$$

The flood hydrograph is the result of a transformation of Q_d using the SCS option of the HEC-HMS model. The major model inputs are the rainfall data and the sub-basin characteristics, represented by the runoff curve number (CN) and the sub-basin area (A). The time lag for each sub-basin was determined by using the time of concentration formula derived by the US Soil Conservation Service

$t_c = \frac{L^{1.15}}{7700 H^{0.38}}$ (Equation 3) where

t_c is the time of concentration in hour
L is the length of the catchment along the mainstream from the basin outlet to the most distant ridge (ft)
H is the difference in elevation between the basin outlet and the most distant ridge (ft)

The required parameters were derived by using the Ikonos image 10e.tif.

Table 3: Lag Time Determination

	Catchment 1	Catchment 2	Catchment 3
L (ft)	1148.9	2624.67	2460.63
H (ft)	13.12	118.11	98.43
t_c (hr)	0.16	0.18	0.18
t_{lag} (hr)	0.1	0.11	0.11
t_{lag} (min)	5.81	6.52	6.49

The time lag is defined as 0.6 $t_{c.}$

Outflow

For elevations below 101.76 m amsl it was assumed that beside evaporation a small volume of water leaves the pond through the subsoil. Based on the 1990 Water Resources Development Master Plan evaporation for Monymusk/Clarendon has been reported at 133 mm for the month of October. This is an average of approximately 5 mm/day. The evaporation rate was converted into a flow and adjusted to the various pond areas depending on the water levels (Table 1). While the evaporation rate was kept constant with 5 mm/day the losses through the subsoil were variable depending on the pond level. Based on the slow rate of recession it is obvious that the soil has a poor permeability. Without having tested the soil a literature k-value of 10^{-7} m/s representing poorly permeable soil was assumed throughout the pond area. This rate was applied and converted into a flow. The algorithm took into account that with lower pond levels less percolation area is available. Convergence between model and observation was achieved once the modeled levels were within a +/- 1 % band. For elevations above the maximum level at the Kennedy Grove entrance the following approach was taken. The 1.2 m high wall facing the scheme was considered a weir which would allow for an overflow at an elevation of 103 m amsl. Between 101.76 m amsl and 103 m amsl the property gate with a width of 8 m was considered the only outlet. A flow velocity of 0.5 m/s was assumed resulting in an incremental outflow rate of 0.8 m^3/s for each 0.2 m water level.

Rainfall

There are several rainfall stations in the Kennedy Grove environs including May Pen, Hunts Pen, Sevens and Bois Content. However only Bois Content had measured rainfall data for events associated with Hurricane Wilma, Dennis and Emily and was subsequently used for calibration and verification. The rainfall depths for Wilma, Dennis and Emily are shown in tables 4 and 5 and the rainfall stations shown in figure 3

Table 4: Daily Rainfall Hurricane Wilma at Bois Content

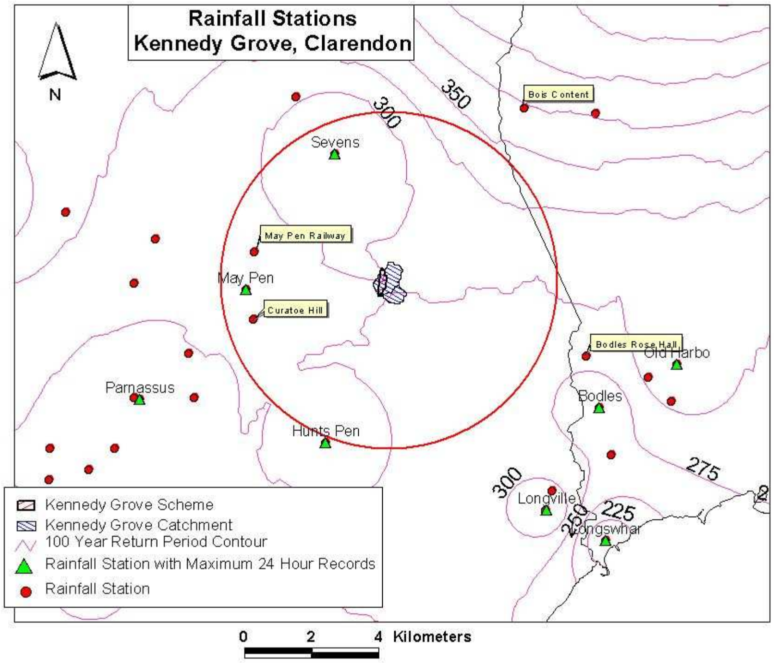
Bois Content rainfall station			
10/14/2005	10/15/2005	10/16/2005	10/17/2005
mm	mm	mm	mm
34.2	74.9	154.3	254.8

Table 5: Daily Rainfall Hurricanes Dennis and Emily at Bois Content

Bois Content (Hurricane Emily and Dennis)				
7/1/2005	7/2/2005	7/6/2005	7/7/2005	7/16/2005
70.1	10	20.9	90.2	61.7

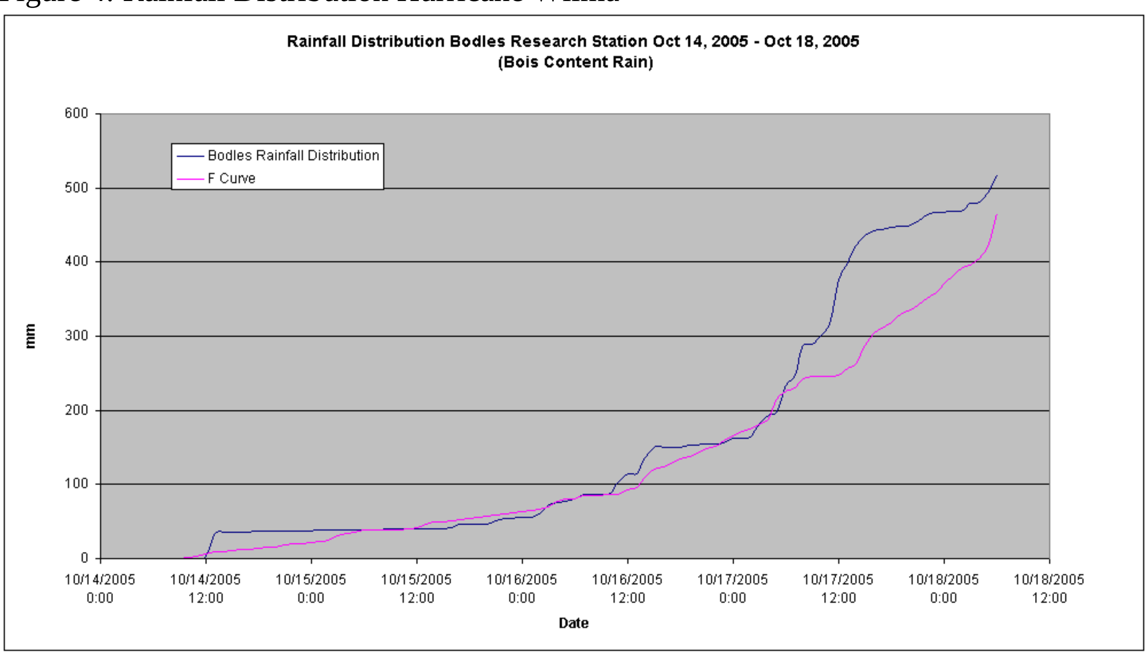
For May Pen, Sevens and Hunts Pen, return periods were determined for the maximum 24 hour rainfall events by the Meteorological Office of Jamaica. These were however not done for Bois Content. The 100 year return period 24 hour maximum rainfall depths plotted in figure 3 show that the catchment lies on the 300 mm contour. The records indicated that the May Pen station has a maximum 24-hour, 100-year return period rainfall of 303 mm. Hence this station was used in the model to simulate the designated water levels in the depression.

Figure 3: Rainfall Stations and 24 Hour Rainfall Contour, 100 Year Return Period



The Bois Content rain for the calibration process was distributed using rainfall distribution from the intensity gauge installed at the Bodles Agricultural Research Station. This is shown in figure 4. Also shown in this figure is the F Curve distribution, which was developed for Jamaica in 1987 for a UNDP/ODP project on Flood Plain Mapping. The F curve was used to distribute the T-year rainfall depths for the May Pen station given the similarity in their distributions.

Figure 4: Rainfall Distribution Hurricane Wilma



Determination of Return Period for Rains Associated with Hurricane Wilma

Table 6: Rainfall Depth with Various Return Periods for May Pen Station

May Pen	
Return Period	mm
2 Y	103
5 Y	157
10 Y	192
25 Y	237
50 Y	270
100 Y	303

On the 17^{th} October 254.8 mm of rain was measured at Bois Content. The maximum 24 hour rainfall is estimated to be 264 mm based on a factor of 1.04 (Ref. 2). This represents approximately a 50 year event from the above table. The maximum probable 24 hour rainfall depth at this station was determined to be 888 mm using the following formula (Ref. 3).

$$PMP = \overline{R} + K_m \sigma$$

where $\overline{R}$ is the mean of the annual maximum rainfall (108 for entire dataset)
K_m an estimated value of 15 (Hershfield, 1961) and
σ is their standard deviation (52 for dataset)

CN Determination

The Curve Number (CN) is a critical parameter for the rainfall runoff model when applying the SCS method. The CN relates soil type to ground coverage and the higher the curve number the greater the degree of impermeability. The following table indicates how the CN was determined. SCS soil tables and CN tables were used to arrive at an average CN for the catchment. CN (III) indicates the curve number under saturated ground conditions.

Table 7: CN Determination

	Site	Area (m^2)	average CN	CN (III)	Weight	Remarks
catchment 1	area west of scheme	26855	79	90	33.9	woodlands, fair, Type D soil
	area west of scheme	51537	89	95	65.1	pasture range poor, Type D soil
	small area within scheme	741	92	97	0.9	road, Type D soil
	Total	79133				
	Weighted CN	**93.32**				
	Site	Area (m^2)	average CN	CN (III)	Weight	Remarks
catchment 2	area southeast of scheme	68817	79	90	87.0	woodlands, fair, Type D soil
	small area within scheme	480	89	95	0.6	pasture range poor, Type D soil
	road outside of scheme	3633	92	97	4.6	road, Type D soil
	Total	72930				
	Weighted CN	**90.38**				
	Site	Area (m^2)	average CN	CN (III)	Weight	Remarks
catchment 3	area east of scheme	240052	36	56	66.5	woodlands, fair, Type A soil
	area west scheme	19782	79	90	5.5	woodlands, fair, Type D soil
	road/roof inside of scheme	29028	92	97	8.0	road, Type D soil
	remainder of scheme	71935	89	95	19.9	pasture range poor, Type D soil
	Total	360797				
	Weighted CN	**68.94**				

Model Calibration and Verification

The calibration process involved a) simulating the water levels of the depression and the environs using the HEC-HMS model b) comparing the simulated water levels with the observed water levels and c) adjusting the CN governing the inflow rate or the percolation governing the outflow rate.

The "observed" maximum water level derived from information received from Mr. Anderson a resident of the Kennedy Grove scheme was 102.26 m amsl which occurred between Monday night and Tuesday morning (Oct. 18, 2005). This was determined by using the surveyed elevation of 101.76 m amsl at the entrance plus 0.5 m depth of water preventing vehicles from entering the scheme. The WRA observations after the event were used to calibrate the recession.

Table 6 shows that the simulated peak in fact occurred at about 1000 hours on the 18th of October with a maximum water level of 102.32 m amsl and a peak inflow rate of 2.13 m^3/s The simulated water level is in line with Mr. Anderson's observation.

The observations made by the WRA were used for comparison with the simulated rate of fall. The WRA visited the site on November 1, 2005 and noted that the water levels had receded by approximately 3 m. This information was obtained by surveying the elevation difference between the highwater mark (102.3 amsl) at one of the houses and the then present water level. The water level on November 1, 2005 was approximately 99.3 m amsl. On November 9, 2005 it was noted that water levels had receded by a further 1 m to reach an elevation of approximately 98.3 m amsl.

Table 8: Modeled and Observed Water Levels

18-Oct-05		
observed water level (m)	modelled level (m)	difference %
102.26	102.32	-0.06
1-Nov-05		
observed water level (m)	modelled level (m)	difference %
99.30	98.68	0.62
9-Nov-05		
observed water level (m)	modelled level (m)	difference %
98.30	97.78	0.53

The calibration run was considered acceptable as the water levels on the descending limb differed by 0.62 % for the 1st of November and 0.53 % for the 9th of November, 2005.

Verification of the model was done using the Dennis/Emily rainfall information. Mr. Anderson, the resident from Kennedy Grove indicated that during the Dennis/Emily event water levels rose to about knee height along the third house on Cedar Ave. This house is opposite of the housing units located at the lowest elevation in the scheme. The contour information indicates a ground elevation of 100.5 amsl. Assuming a water depth of 0.6 m representing knee height the maximum water level was 101.1m amsl. The HEC-HMS model determines a maximum water level of 101.43 m amsl which differs by 0.3 m or 0.32 %. The model setting has been accepted as adequate as it represents closely the actual water levels.

Simulation Runs for Interventions

Intervention 1

Intervention 1 represents the ‘do nothing’ option which entails leaving the depression area undisturbed, allowing natural drainage to take place. It involved using the calibrated model to simulate water levels for the designated return period rainfall and the probable maximum precipitation.

Intervention 2

Intervention 2 was represented in the model by the addition of two soakaways. The outflow rate through the soakaways was determined during the draining of the pond by pumping.. The percolation rate of the soakaways was assumed to be greater than or equal to the pumping rate stated on the pumps, given the fact that there was no ponding in the soakaways during pumping. This rate was 31 l/s. The assumption in the model was that the second soakaway to the west of the scheme and the sinkhole at the bottom of the pond which is to be cleaned have the same infiltration rate. The maximum outflow through the two soakaways and the sinkhole would then be 93 l/s or 0.09 m^3/s. This rate would be attained above the 98 m contour line as the invert of the soakaways is located at this elevation. Below this rate a percolation of 0.03 m^3/s was assumed.

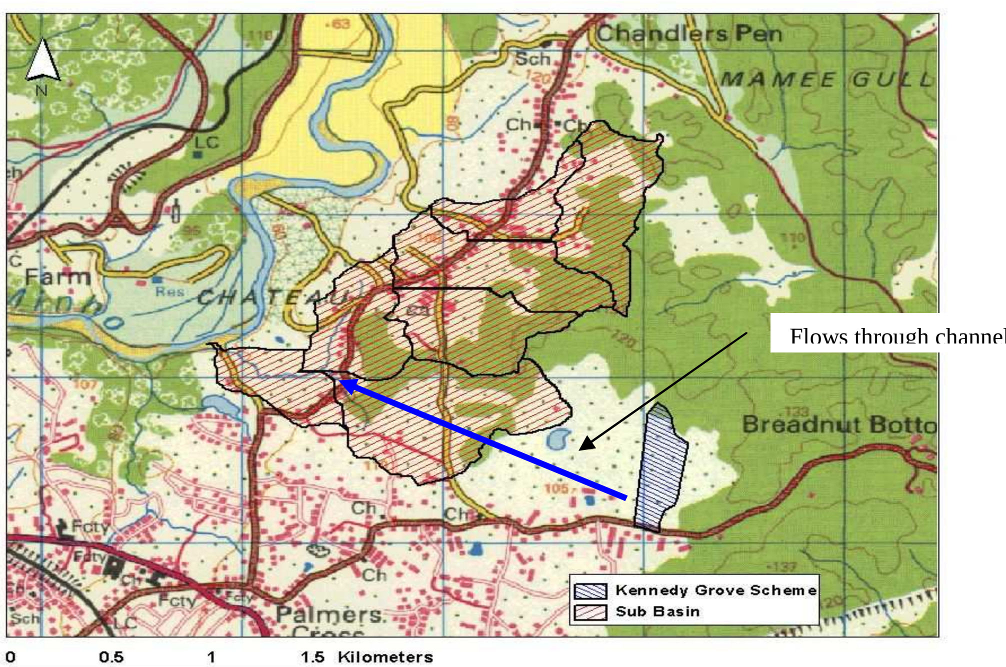
Intervention 3

Intervention 3 was represented in the model by the rehabilitated sinkhole including the additional two soakaways. An estimate of the percolation rate of the rehabilitated sinkhole was made based on the results of a limestone aquifer recharge study conducted in the 1980's in the Innswood area (Ref.1). This indicated that the average percolation/absorption rate of a single sinkhole was 4 cfs or 0.11 m^3/s. This rate was used in the model in addition to the 0.06 m^3/s of the two soakaways.

Intervention 4

Intervention 4 involves the simulation of the flows through the proposed channel and an assessment of the impacts of the backwater effect in the Rio Minho-Chateau reach (Figure 7) using the HEC-RAS hydraulic analysis. It is assumed that peak inflow rate into the channel equals the peak outflow rate from the depression. The backwater effect is assessed based on flows from the Chateau-Rio Minho tributary catchment **and** flows from the proposed channel. The invert of the channel in the scheme was set to 98 m amsl representing the base of the house at the lowest elevation. The 100 year peak flow of 3.4 m^3/s was considered a conservative design flow.

Figure 5 : Location Map of Catchment and Kennedy Grove Scheme



Flows from the tributary catchment were estimated using the rainfall runoff model described above. The catchment area for the tributary was determined using the DSM contours. The Curve Number was calculated using land use and soil classes found in the catchment.

Table 9: CN Determination for Catchment

Landuse Type from TFT document	Area m^2	Soil type SCS Table	CN SCS Table	Area m^2	% of total area
SC, SF	707359.00	A	56.00	538719.00	23.77
		D	91.00	168640.00	7.44
FC	3516.00	D	97.00	3516.00	0.16
CS	1553127.00	A	62.00	1100000.00	48.54
		D	95.00	453127.00	19.99
BA	2219.00	D	97.00	2219.00	0.10
Total Area of Catchment	2266221.00				100.00
weighted CN	**69.42**				

Table 10: Catchment Characteristic

Catchment Characteristic			
Longest flow path	Height Difference	tc	lag time
m	m	hr	hr
2960	58	0.72	0.4

The 24 hour rainfall depths for May Pen (Table 6) formed the input into the HEC-HMS model to generate flows from the catchment.

The flows generated from this catchment by the HEC-HMS model are shown in table 11.

Table 11: Simulated Peak Flows in the Chateau to Rio Minho Tributary

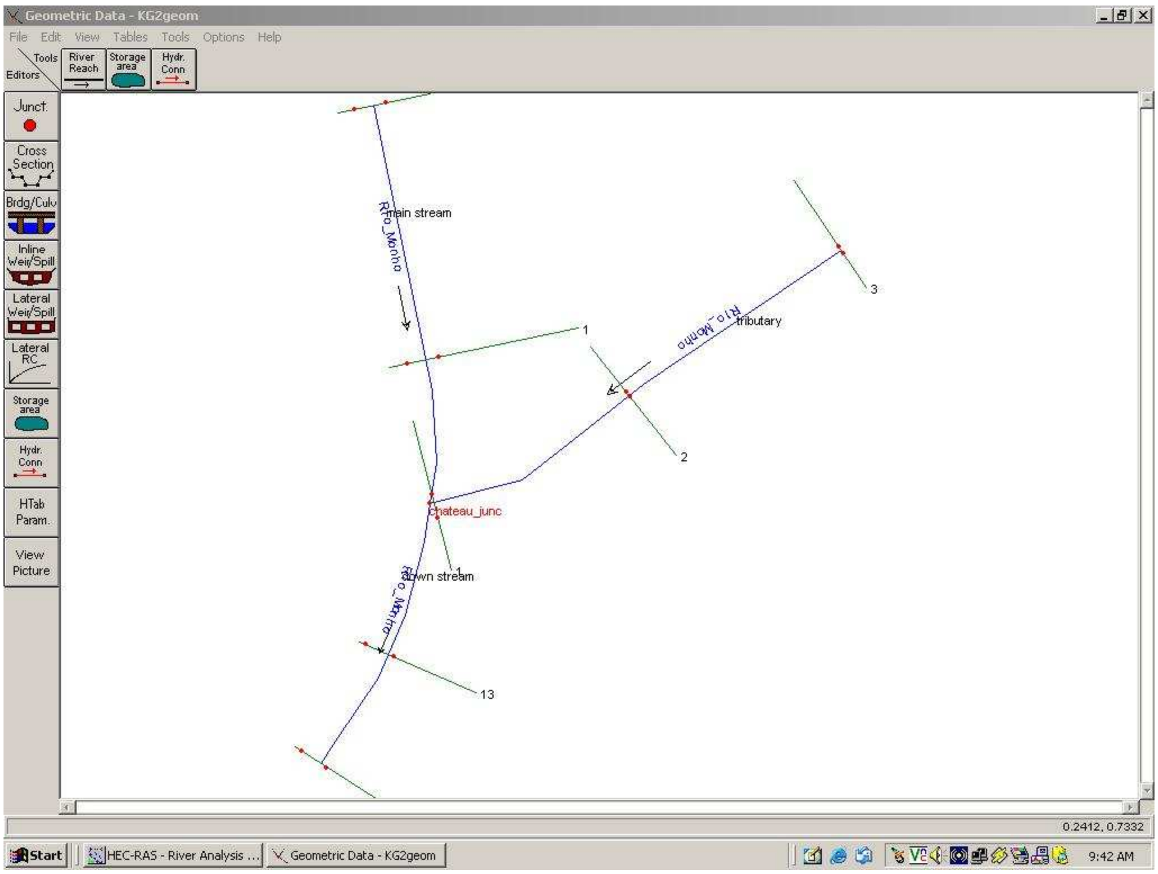
Return Period (y)	5	10	25	50	100	PMP
	Peak Flows [m^3/s]					
catchment	6.6	8.7	11.4	12.42	14.23	45.55
Kennedy Grove S	1.71	2.18	2.81	3.01	3.4	10.7
Total	**8.31**	**10.9**	**14.2**	**15.43**	**17.63**	**56.25**

The absence of a gauge at this tributary prevents the calibration using observed flow data however the flows shown in table 11 were accepted based on the following considerations. The Rio Minho and its sub basins have been calibrated for the Rio Minho Flood Plain Mapping project. The Pindars River sub basin although much larger (78 km^2) than the catchment to be modeled has been calibrated with real flows at the Rio Minho @ Danks station. The catchments have a similar condition with soils having moderate permeability and landuse consisting of fields and disturbed broadleaf. The CN for the

Pindars River sub basin has been set to 71. The weighted CN for the sub basin to be modeled was calculated at 69.4 (Table 9).

Water surface elevation was simulated from the flows in table 11 using the HEC-RAS hydraulic model developed by USACE. The basic schematic is shown in Figure 6.

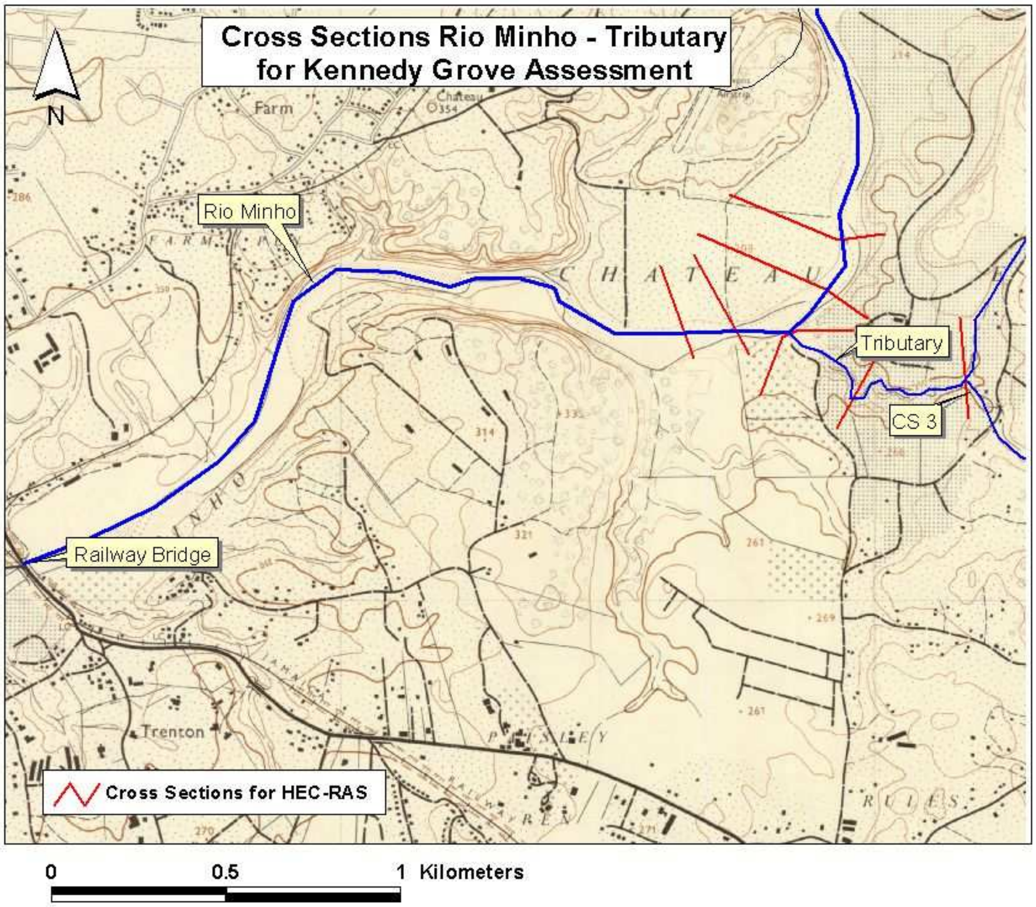
Figure 6: Model Schematic HEC-RAS



The designated flows (table 11, last row) were routed through the tributary channel resulting in water levels along the channel. In the absence of observed water surface elevations at the confluence water surface elevations at the railway bridge of 12 m was used to represent the starting water levels in the Rio Minho.

The absence of surveyed cross section information for the tributary required us to determine the channel geometry using digitized cross sections from the 1:12,500 topographic map sheet.

Figure 7: Location of Cross Sections



RESULTS

Intervention 1

Table 12 shows that maximum simulated water levels range between 101.26 m amsl and 102.52 m amsl for the 5 year to 100 year return period rainfall and 103.2 m amsl for the probable maximum precipitation event. The model indicates that it will take at least 20 days before the water levels recede to the base of the house at the lowest elevation (98 m amsl).

Table 12: **Simulated Water Levels and Duration of Flooding under Various Intervention Scenarios**

Intervention #	Measures		Return Period (years) 5	10	25	50	100	PMP
1	'do nothing'	max. water level (m amsl)	101.26	101.66	102.17	102.36	102.52	103.2
		duration of flooding	20 d	21 d	21 d	21 d	21 d	21 d
2	2 working soakaways and sinkhole	max. water level (m amsl)	101.09	101.56	102.06	102.28	102.44	103.08
		duration of flooding	3 d 19 hr	4 d 17 hr	5 d	5 d	5 d	5 d
3	two soakaways and cleaned sinkhole	max. water level (m amsl)	100.8	101.35	101.85	102.18	102.37	103.05
		duration of flooding	1 d 15 hrs	2 d 4 hrs	2 d 14 hrs	2 d 16 hrs	2 d 16 hrs	2 d 18 hrs
4	cut channel for flow of 3.4 m^3/s	max. water level (m amsl)	98	98	98	98	98	102.91
		duration of flooding	0	0	0	0	0	10 hrs
5	two soakaways and cleaned sinkhole, no inflow from road	max. water level (m amsl)	100.24	100.8	101.38	101.67	102.01	102.95
		duration of flooding	1 d 1 hrs	1 d 13 hrs	2 d 4 hrs	2 d 13 hrs	2 d 16 hrs	2 d 16 hrs

Intervention 2

Table 12 shows that water levels range from 101.09 m amsl to 102.44 m amsl for the 5 year to 100 year return period rainfall and 103.08 m amsl or the maximum probable precipitation which is similar to the ‘do nothing’ intervention. However the duration of flooding is reduced significantly by 16 days.

Intervention 3

Table 12 shows that water levels range from 100.8 m amsl to 102.37 m amsl for the 5 year to 100 year return period rainfall and 103.05 m amsl or the maximum probable precipitation which is similar to the ‘do nothing’ intervention. The duration of flooding is reduced by 18 days when compared with the ‘do nothing’ intervention.

Intervention 4

Based on the assessment of intervention 4 the proposed channel should be able to convey a discharge of 3.4 m^3/s to maintain a water level of 98 m amsl within the depression. The backwater assessment shows that for the reach of the Chateau-Rio Minho tributary all flows would generally be contained within the channel. This concurs with information received from Mr. Blake of the Clarendon Parish Council. Table 13 shows specifically the water levels at the cross section nearest to the Chateau parochial road.

Table 13.: Water Levels along the Tributary to the Rio Minho

Return Period (year)	Confluence water level in (m amsl)	cross section 3			
		water level (m amsl)			bank overflow
		without KG flow	combined flow	difference (m)	
5	68.07	74.19	74.27	0.08	no
10	68.07	74.29	74.38	0.09	no
25	68.07	74.4	74.49	0.09	no
50	68.07	74.44	74.54	0.1	no
100	68.07	74.5	74.61	0.11	no
PMP	68.07	75.22	75.39	0.17	no
High Water Level Rio Minho (simulated): 68 m amsl					

Intervention 5

Based on anecdotal evidence water enters the scheme not only from the north but also via the parochial road from the south. In developing the drainage model (figure 1) it was noted that the natural drainage from catchment 1 is towards the south and not into the scheme, likewise a part of catchment 2 south of the road representing approximately 20 % of this catchment. Intervention 5 considered the removal of the runoff contribution from these two sub sections. Table 12 shows that water levels range from 100.24 m amsl to 102.01 m amsl for the 5 year to 100 year return period rainfall and 102.95 m amsl for the maximum probable precipitation, which is similar to the ‘do nothing’ intervention. The duration of flooding for return periods less than 50 years is further reduced by approximately 10 hours.

IMPACT DETERMINATION

Intervention 1, 2 and 3

Figure 8 shows the maximum extent of flooding under a 100-year return period rainfall. 78 housing units and more than 800 m of road network would be impacted. The maximum depth of water along the western road within the scheme is 5.12 m. Table 14 shows the level of impact under the various interventions and for the designated return periods of rainfall. At water levels greater than 102.3 m amsl the main entrance will not be accessible and an alternative route will have to be established. The sewage pump located close to the existing pond will always be impacted by flood waters under any of the return period rainfall events.

Figure 8: Extent of Inundation with a 100 Year Rainfall Event

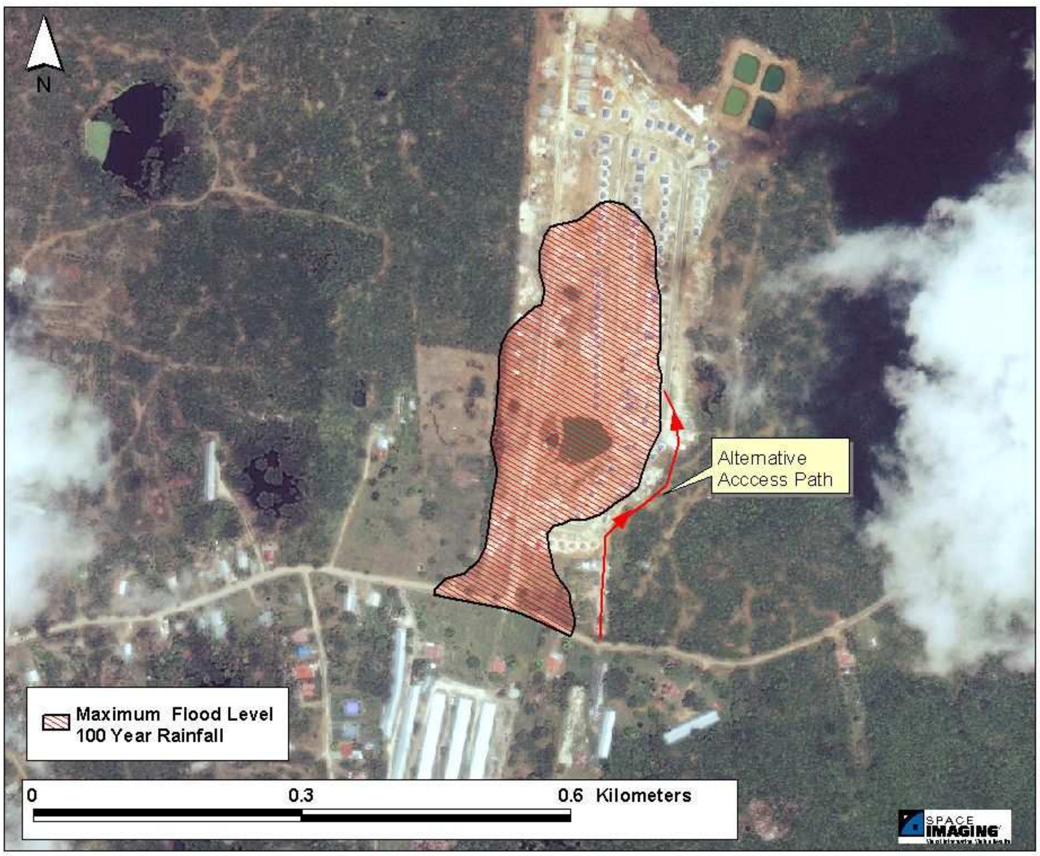


Table 14: Level of Impact

Intervention #	Measures		Return Period (years) 5	10	25	50	100	PMP
1	'do nothing'	# of houses impacted	30	51	73	78	78	> 78
		road length impacted (m)	420	550	> 800	> 800	> 800	> 800
		maximum depth of water (m)*	3.86	4.26	4.77	4.96	5.12	5.8
2	2 working soakaways	# of houses impacted	25	51	67	78	78	> 78
	and sinkhole	road length impacted (m)	390	540	> 800	> 800	> 800	> 800
		maximum depth of water (m)	3.69	4.16	4.66	4.88	5.04	5.68
3	two soakaways and	# of houses impacted	13	32	54	73	78	> 78
	cleaned sinkhole	road length impacted (m)	320	530	670	> 800	> 800	> 800
		maximum depth of water (m)	3.4	3.95	4.45	4.78	4.97	5.65
4	cut channel for flow of 3.4 m^3/s	# of houses impacted	0	0	0	0	0	> 78
		road length impacted (m)	0	0	0	0	0	> 800
		maximum depth of water (m)	0	0	0	0	0	5.51
5	two soakaways and cleaned	# of houses impacted	9	13	32	51	66	>78
	sinkhole, no inflow from road	road length impacted (m)	250	320	530	550	720	> 800
		maximum depth of water (m)	2.84	3.4	3.98	4.27	4.61	5.55
	* determined at lowest elevation along western abandoned road							

CONCLUSION

Interventions 1 –3 producing roughly the same results with respect to water levels would necessitate the relocation of at least 78 houses for rainfall events with a return period greater than 50 years (Table 12). The sewage infrastructure will have to be redesigned as the sewage lift pump needs to be relocated to a higher elevation. The main entrance to the Kennedy Grove scheme becomes inundated at water levels greater than 101.76 m amsl. An alternative access road needs to be established either to the west or the east of the main entrance.

Intervention 4 indicates that drainage of the floodwaters via the proposed channel is the most effective option for reducing the flood levels in the scheme. This channel should be designed to discharge peak flow of 3.4 m^3/s corresponding to the 100-year rainfall. It has to be considered that this will necessitate extensive cutting through rock material over a length of approximately 1.5 km. From a water resources point of view this might not be the most suitable option as the depression in the scheme acts as a recharge to the limestone aquifer and conveying water out of the scheme would result in loss of recharge.

The selection of the most suitable option should therefore be guided by a socio-economic assessment.

References

1. Botbol March 1981--Lower Rio Cobre Limestone Aquifer Artificial Groundwater Recharge Progress Report No. 3. An unpublished report of the Underground Water Authority

2. National Meteorological Service 1989- Flood Plan Mapping Project, Transfer of Technology.

3. Shaw, Elizabeth 1983 Hydrology in Practice