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FLORENCE HALL HOUSING DEVELOPMENT

FCS # 0827/76/C

ENGINEERING REPORT

GENERAL DRAINAGE

PREPARED FOR
GORE DEVELOPMENTS LIMITED
2C BRAEMAR AVENUE
KINGSTON 10

NOVEMBER 2008

GORE DEVELOPMENTS LIMITED
FLORENCE HALL

1.0 OVERVIEW

Foreman Chung and Sykes Consultants Limited has been retained by Mr. Christopher Gore of Gore Developments Limited to prepare infrastructure designs including drainage designs for the proposed subdivision.

Gore Developments Limited intends to develop 70.07 hectares of lands east of the Trelawny Multipurpose Stadium as a Housing Estate.

The land borders the North Coast Highway to the south, similar lands to the east, a Jamaica Public Service (JPS) transmission lines and parochial road to the north and the road to Daniel Town to the west of the proposed development. There are three land parcels that fall within the forgoing description that are not included in the development area. This is shown in the attached location plan.

The development lands are located on the slope of the rolling limestone hills that predominate in the area. The area was once farmed but is now overgrown with trees and brush.

Drainage Criteria

DESIGN STORM EVENT

The Jamaica Institution of Engineers recommended “Guidelines for the design and Construction of Housing Infrastructure” Vol 1: 1984 Storm Water Drainage recommends that the design storm frequency of storm sewers be 2 years and for culverts, bridges and flood control projects a minimum of 10 years.

In the Standard Handbook for Civil Engineers by Merritt, Loftin and Ricketts article 14.9 states “Flooding problems and surface drainage as concerns of community and regional planning studies, differ primarily in degree of severity. The principal concern with flooding is the desire to avoid injury and loss of life and reduce property damages caused by major floods (those having a recurrence interval of 25 to 100 years).

Surface-drainage systems on the other hand are primarily concerned with convenience and providing access to property in relatively minor storms (those having a recurrence interval of 2 to 10 years)".

Investigations will be conducted for the 1 in 25 year event for the main drainage channels and 1 in 10 year event for the subdivision drains.

The FHWA HEC 22 recommendation is shown in Table 4-1 below and will be used for the designs.

Table 4-1. Suggested Minimum Design Frequency and Spread.

Road Classification		Design Frequency	Design Spread
High Volume or Divided or Bi-Directional	< 70 km/hr (45 mph)	10-year	Shoulder + 1 m (3 ft)
	> 70 km/hr (45 mph)	10-year	Shoulder
	Sag Point	50-year	Shoulder + 1 m (3 ft)
Collector	< 70 km/hr (45 mph)	10-year	1/2 Driving Lane
	> 70 km/hr (45 mph)	10-year	Shoulder
	Sag Point	10-year	1/2 Driving Lane
Local Streets	Low ADT	5-year	1/2 Driving Lane
	High ADT	10-year	1/2 Driving Lane
	Sag Point	10-year	1/2 Driving Lane

FHWA HEC-22

METHOD OF DETERMINING DESIGN PEAK FLOWS

1. For drainage areas less than 200 acres, the design engineer shall use the Rational Method (Q=CIA) procedure for determining runoff flow. For drainage areas between 200 and 2,000 acres, the design engineer shall use the most recent NRCS Method, for determining runoff rates.

2. **Drains to be sized to** FHWA Hydraulic Engineering Circular No 22 – Urban Drainage Design **HEC –** 22.

3. **Culverts to be sized and conform to** FHWA-NHI-01-020-HDS 5 (Hydraulic Design Series No 5)– Hydraulic design of highway culverts **–** Second Edition.

The calculation for peak runoff using the rational method is set out below:

$Q = C\ i\ A \times 1/Ku$

Where: Q = Flow, m^3/s (ft^3/s)
C = coefficient of runoff (dimensionless)
i = rain intensity mm/hr (in/hr)
A = drainage area, hectares, ha (acres)
Ku = units conversion factor 360 (1 in English units))

The runoff coefficients by hydraulic soil group and slope range used in the rational method were developed by Rawls et al. as shown in table 7.6 in the standard handbook of environmental engineering (by Robert Corbitt).

The proposed development is located south east of the Falmouth Trelawny and the rain data for that town is used to estimate the rain intensity for the project (see **Table 1.0**).

Table 1.0 Rainfall Data

Florence Hall rainfall Data	
24 hr Return	**mm/day**
1 in 2 yr	102
1 in 5 yr	131
1 in 10 yr	159
1 in 25 yr	194
1 in 50 yr	220
1 in 100yr	246

STORM SEWERS

The storm sewer system being the buried drainage conveyance system below the roadway pavement will be designed to convey a 1:10 year storm without surcharging.

The discharge of the storm sewers is mostly to paved drains and positive drainage will be maintained in the design. Minimum cover will be to the manufacturers' specifications.

Open Drains

The open drains will be used where possible and erosion protection using both rigid and flexible linings will be used in the design. CALTRANS Highway Design Manual chapter 860 Open Channels will be used to guide the designs. The maximum velocity for unlined channels in table 862.2 is used to guide the designs.

Table 862.2

Recommended Permissible Velocities for Unlined Channels

Type of Material in Excavation Section	Permissible Velocity (m/s)	
	Intermittent Flow	Sustained Flow
Fine Sand (Noncolloidal)	0.8	0.8
Sandy Loam (Noncolloidal)	0.8	0.8
Silt Loam (Noncolloidal)	0.9	0.9
Fine Loam	1.1	1.1
Volcanic Ash	1.2	1.1
Fine Gravel	1.2	1.1
Stiff Clay (Colloidal)	1.5	1.2
Graded Material (Noncolloidal)		
Loam to Gravel	2	1.5
Silt to Gravel	2.1	1.7
Gravel	2.3	1.8
Coarse Gravel	2.4	2.0
Gravel to Cobbles (Under 150 mm)	2.7	2.1
Gravel and Cobbles (Over 200 mm)	3	2.4

The manning's roughness coefficients to be used in the evaluation of the design depth of flow is guided by table 863.3a shown below.

Table 864.3A

Average Values for Manning's Roughness Coefficient (n)

Type of Channel		**n value**
Unlined Channels:		
	Clay Loam	0.023
	Sand	0.02
	Gravel	0.03
	Rock	0.04
Lined Channels:		
	Portland Cement Concrete	0.014
	Air Blown Mortar (troweled)	0.012
	Air Blown Mortar (untroweled)	0.016
	Air Blown Mortar (roughened)	0.025
	Asphalt Concrete	0.018
	Sacked Concrete	0.025
Pavement and Gutters:		
	Portland Cement Concrete	0.015
	Asphalt Concrete	0.016
Depressed Medians:		
	Earth (without growth)	0.04
	Earth (with growth)	0.05
	Gravel	0.055

Freeboard in the open drains will be guided by table 866.2 of the CALTRANS Highway Design Manual.

Table 866.2

Guide to Freeboard Height

Shape of Channel	Subcritical Flow	SupercriticalFlow
Rectangular	0.1 He	0.20 d
Trapezoidal	0.2 He	0.25 d

Where He = Energy head, in meters
d = Depth of flow, in meters for a straight alignment

For rigid pavements the FHWA HDS 4 guidance will be used and for flexible linings FHWA HEC-15 will be used along with the CALTRANS guidelines.

EROSION CONTROL

CALTRANS Highway Design Manual chapter CHAPTER 870 CHANNEL AND SHORE PROTECTION - erosion control and FHWA HEC 14 hydraulic design of energy dissipators for Culverts and Channels will be used to design the erosion control features.

Drainage Design

PREDEVELOPMENT CONDITION

The proposed development lands are located on the slopes of the rolling limestone hills in the area. The land slopes from the south to the northeastern corner of the property. The North Coast Highway borders the north of the development lands and forms a dyke north of the wetland area. Those wetlands extend further east on lands outside of the development lands. The wetland area with the highway embankment forms a surface water storage area upstream of the highway.

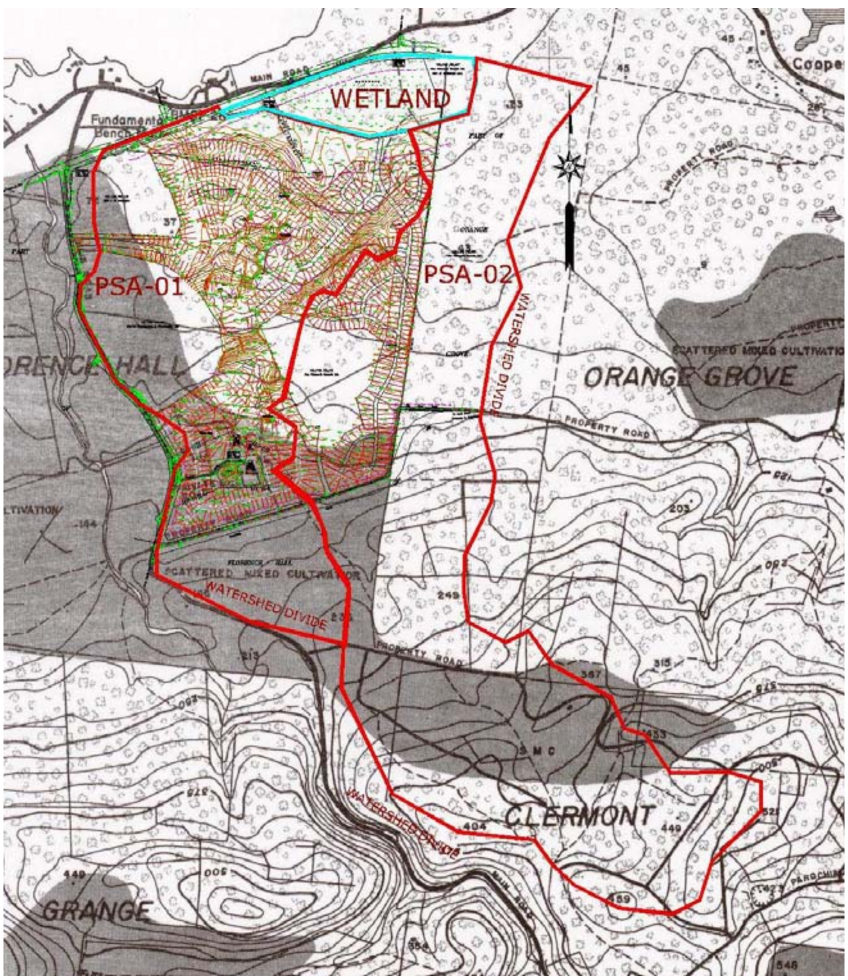

Figure 1 Overall catchments including the project lands

The proposed development is located within two catchments that extend to the south of the project lands. These are identified as PSA-01 and PSA-02. Each sub catchment drains toward the wetland located to the north eastern corner of the project lands.

The existing ground surface includes depressions and caverns at various parts of the site that detain and retain surface runoff.

The land is now overgrown with trees and brush where well fractured limestone and rough limestone outcrops predominates the landscape. The soil type in the area is named Bonnygate and the internal drainage characteristic is described as very rapid. The fractured nature of the limestone and the caverns observed on the site indicates that the existing ground condition will allow for rapid infiltration. Figure 2 shows a fairly recent satellite image of the area that describes the predevelopment land use.

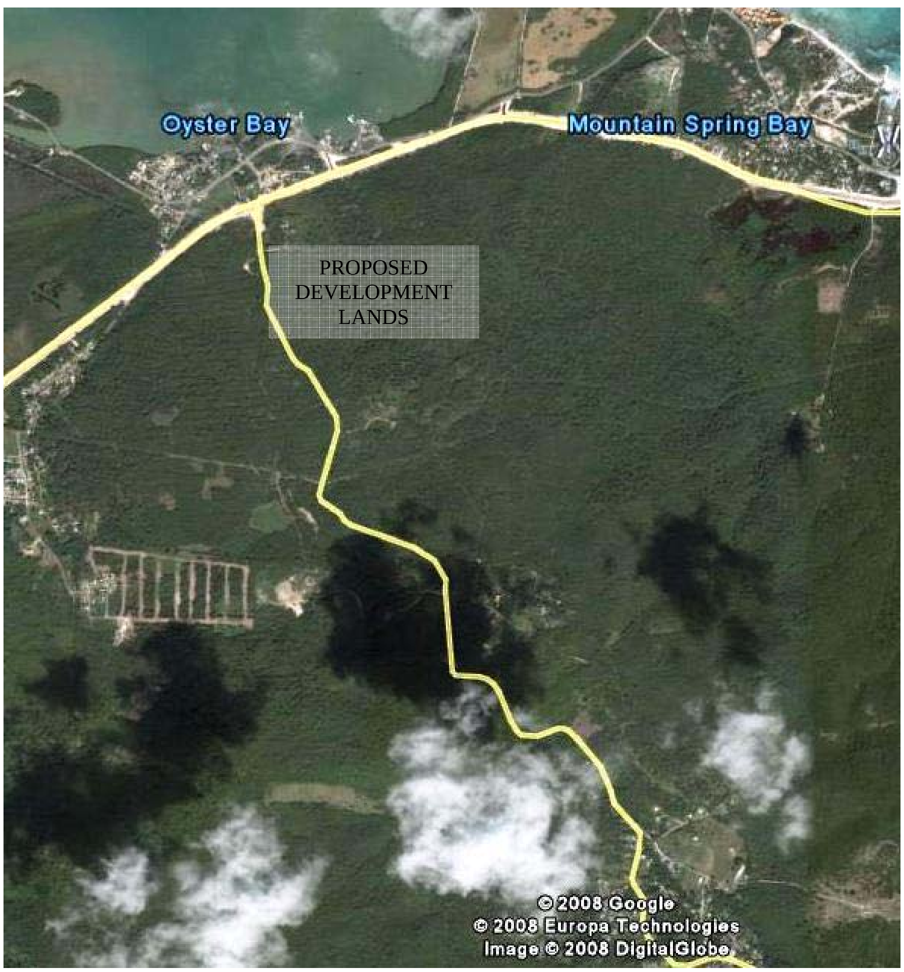

Figure 2 Fairly recent satellite image showing predevelopment land use

ESTIMATE OF PREDEVELOPMENT SURFACE RUNOFF

The predevelopment model used assumed that this region has type A hydraulic soil group and that the ground cover was brush weed and mixed grass with poor ground cover. A summary of the predevelopment sub-catchments and drainage characteristics is shown in Table 1.1.

Table 1.1

FCS	Florence Hall Pre Development Evaluation			
	Region:	Trelawny	Locale:	Falmouth
	Sub-Area Summary Table			
Sub-Area Identifier	Drainage Area (ha)	Time of Concentration (hr)	Curve Number	Receiving Reach
PSA-01	79.8	0.584	49	R3
PSA-02	118.6	0.734	48	R3
WETLAND	9.5	0.1	98	R3
Total	Area:	207.9	(ha)	

The runoff is routed through the wetland detention area where the outlet has been modeled as a 1.8m pipe assumed to be equivalent to the four 1.2m diameter pipes that currently cross the North Coast Highway. The estimated runoff from the subareas and discharge through the pipe from the wetland is shown in Table 1.2.

Table 1.2

FCS	Florence Hall Pre Development Evaluation			
	Region: Trelawny		Locale: Falmouth	
	Hydrograph Peak/Peak Time Table			(Trial #2)
Sub-Area or Reach Identifier	Peak Flow and Peak Time (hr) by Rainfall Return Period			
	10-Yr (cms) (hr)	25-Yr (cms) (hr)	50-Yr (cms) (hr)	100-Yr (cms) (hr)
SUBAREAS				
PSA-01	4.12	7.5	10.36	13.44
	12.28	12.26	12.25	12.24
PSA-02	4.74	8.84	12.35	16.21
	12.42	12.38	12.35	12.36
WETLAND	5.32	6.5	7.38	8.26
	11.92	11.93	11.92	11.93
REACHES				
R3	9.24	16.72	23.08	30.04
	12.35	12.33	12.32	12.32
Down	2.31	3.83	5.13	6.35
	13.54	13.43	13.37	13.36
OUTLET	2.31	3.83	5.13	6.35

The pre development evaluation includes an estimate of the height of water detained in the wetland area south of the North Coast Highway Table 1.3 shows that it is anticipated that the road will not be affected by high runoff events.

Table 1.3

FCS		Florence Hall Pre Development Evaluation			
		Region: Trelawny	Locale: Falmouth		
		Structure Output Table			
Reach	Identifier	Peak Flow (PF), Storage Volume (SV), Stage (STG) by Rainfall return Period			
Structure	Identifier	10-Yr	25-Yr	50-Yr	100-Yr
Reach:	R3				
Pipe :	Wetland				
1500(cm)					
PF	(cms)	1.91	3.14	4.19	4.92
SV	(ha m)	2.91	4.78	6.38	8.25
STG	(m)	0.31	0.5	0.67	0.87
1800(cm)					
PF	(cms)	2.31	3.83	5.13	6.35
SV	(ha m)	2.65	4.39	5.88	7.54
STG	(m)	0.28	0.46	0.62	0.79

ESTIMATE OF POST DEVELOPMENT SURFACE RUNOFF

For the post development evaluation the subdivision was divided into various subareas. It is anticipated that only 20% of the eastern predevelopment subarea PSA-02 (called SA6 in post development evaluation) will be developed in this effort. Most of the western predevelopment subarea PSA-01 is either developing presently or will be developed except for a central park area that will have a berm constructed on the perimeter to detain surface water. That park is identified as subarea Detention.

Table 1.4

FCS	**Florence Hall Post Development Evaluation**			
	Region:	Trelawny	Locale:	Falmouth
	Sub-Area Summary Table			
Sub-Area Identifier	Drainage Area (ha)	Time of Concentration (hr)	Curve Number	Receiving Reach
SA1	13.3	0.169	48	R4
SA6	117.8	0.671	54	R3
WETLAND	9.5	0.1	98	R3
SA2	30	0.209	77	R1
SA3	12.3	0.104	77	R3
SA4	8.9	0.117	76	R3
SA5	12.8	0.141	77	R3
Detention	2.75	0.1	68	R2
Total	Area:	207.35	(ha)	

The present ground condition is such that cavities exist in many areas and where practical some will be filled. Where life is absent and the cavities exist in a green area surface water will be allowed to enter and retained for infiltration into the ground. The cavity volumes have been omitted and would add storage volume in the actual condition in this evaluation. The results can be deemed as conservative due to the abstractions that those features will allow.

Table 1.5

FCS	Florence Hall Post Development			
	Region: Trelawny		Locale: Falmouth	
	Hydrograph Peak/Peak Time Table			(Trial #2)
Sub-Area or Reach Identifier	Peak Flow and Peak Time (hr) by Rainfall Return Period			
	10-Yr (cms) (hr)	25-Yr (cms) (hr)	50-Yr (cms) (hr)	100-Yr (cms) (hr)
SUBAREAS				
SA1	1.24	2.2	3	3.86
	12.03	12.03	12.02	12.01
SA6	8.26	13.59	17.94	22.49
	12.34	12.31	12.3	12.28
WETLAND	5.32	6.5	7.38	8.26
	11.92	11.93	11.92	11.93
SA2	10.18	13.49	15.94	18.42
	12.02	12.01	12.02	12.01
SA3	4.87	6.45	7.64	8.82
	11.94	11.93	11.93	11.93
SA4	3.35	4.47	5.3	6.14
	11.94	11.94	11.94	11.94
SA5	4.76	6.31	7.47	8.63
	11.96	11.96	11.95	11.95
Detention	0.83	1.17	1.42	1.68
	11.93	11.94	11.93	11.93
REACHES				
R1	11.31	15.59	18.87	22.21
	12.02	12.02	12.02	12.02
Down	4.6	6.1	6.38	6.68
	12.21	12.21	12.22	12.24
R2	4.75	6.34	7.06	7.53
	12.19	12.12	12.04	12.02
Down	4.75	6.34	7.05	7.53
	12.21	12.14	12.06	12.03
R3	22.41	30.97	37.88	44.87
	11.95	12.02	12.02	12.01
Down	4.61	6.41	7.11	7.86
	13.28	13.3	13.42	13.53
R4	1.24	2.2	3	3.86
	12.03	12.03	12.02	12.01
Down	1.24	2.2	3	3.86
	12.07	12.05	12.04	12.04
OUTLET	4.61	6.41	7.11	7.86

Table 1.5 show the peak flows from the various subareas and outlet flows from the detention areas.

Table 1.6

FCS	Florence Hall Post Development				
	Region: Trelawny	Locale: Falmouth			
Structure Output Table					
Reach	Identifier	Peak Flow (PF), Storage Volume (SV), Stage (STG) by Rainfall return Period			
Structure	Identifier	10-Yr	25-Yr	50-Yr	100-Yr
Reach:	**R1**				
Pipe:	**Det1**				
1220(cm)					
PF	(cms)	3.62	4.31	4.52	4.74
SV	(ha m)	1.06	1.51	1.9	2.31
STG	(m)	0.53	0.75	0.95	1.16
1500(cm)					
PF	(cms)	4.6	6.1	6.38	6.68
SV	(ha	0.93	1.3	1.63	1.99
STG	(m)	0.47	0.65	0.82	1
Reach:	**R3**				
Pipe:	**Wetland**				
1500(cm)					
PF	(cms)	3.7	4.92	5.42	5.94
SV	(ha m)	5.63	8.25	10.6	13.05
STG	(m)	0.59	0.87	1.12	1.37
1800(cm)					
PF	(cms)	4.61	6.41	7.11	7.86
SV	(ha m)	5.29	7.72	9.88	12.19
STG	(m)	0.56	0.81	1.04	1.28

The anticipated flows into and out from the wetland area are shown in Table 1.6. The estimated raise in water level would not rise to the road surface at the road sag even in a 50year event. Highway design allows for sags to be inundated to 30cm in the 100yr event and the estimate of that event show that this would not be compromised.

METHOD OF DETERMINING DESIGN PEAK FLOWS FOR INTERNAL DRAINAGE SYSTEM

4. For drainage areas less than 200 acres, the design engineer shall use the Rational Method (Q=CIA) procedure for determining runoff flow. For drainage areas between 200 and 2,000 acres, the design engineer shall use the most recent NRCS Method, for determining runoff rates. For drainage areas greater than 2,000 acres, or (800 hectares) the design engineer shall use the most recent WRA Regression methods or HEC HMS to estimate runoff rates.

5. **Drains to be sized to** United States Federal Highway Administration (FHWA) Hydraulic Engineering Circular No 22 – Urban Drainage Design **HEC –** 22.

6. **Culverts to be sized and conform to** FHWA-NHI-01-020-HDS 5 (Hydraulic Design Series No 5)– Hydraulic design of highway culverts **–** Second Edition.

The calculation for peak runoff using the rational method is set out below:

$Q = C\ i\ A \times 1/Ku$

Where: Q = Flow, m^3/s (ft^3/s)
C = coefficient of runoff (dimensionless)
i = rain intensity mm/hr (in/hr)
A = drainage area, hectares, ha (acres)
Ku = units conversion factor 360 (1 in English units))

Rain data is taken from the National Meteorological Service's estimates of maximum 24 hour rainfall for selected return periods. This is converted to rainfall intensity by the following equation.

Table 1.7 Regression equations for rainfall intensity (i) in mm/hr for Sangster International Airport for durations less than 30mins.

Return period	Regression equation for i
1 in 10 yr	89764.9/((t+40.735)^1.594)
1 in 25 yr	113157.6/((t+40.618)^1.609)
1 in 50 yr	113382.4/((t+40.618)^1.618)

Where t = rainfall duration (assumed equal to the time of concentration)

Table 1.8 Estimate of runoff flow from sud drainage areas throughout the subdivision.

Gore Florence Hall Drainage runoff evaluation									
Description	**Return period**	**mm/day**	**in/day**	**Tc**	**Area msq**	**Ha**	**c**	**I mm/hr**	**cms**
DA 1	1 in 10 yr	159	6.26	7	21,147.25	2.11	0.65	189.25	0.73
DA 2	1 in 10 yr	159	6.26	7	9,220.96	0.92	0.65	189.25	0.32
DA 3	1 in 10 yr	159	6.26	8	29,452.75	2.95	0.65	183.10	0.98
DA 4	1 in 10 yr	159	6.26	10	22,762.15	2.28	0.65	171.73	0.71
DA 5	1 in 10 yr	159	6.26	10	16,162.13	1.62	0.65	171.73	0.51
DA 6	1 in 10 yr	159	6.26	7	4,091.61	0.41	0.65	189.25	0.14
DA 7	1 in 10 yr	159	6.26	7	8,378.35	0.84	0.65	189.25	0.29
DA 8	1 in 10 yr	159	6.26	7	21,416.67	2.14	0.65	189.25	0.74
DA 9	1 in 10 yr	159	6.26	10	28,940.94	2.89	0.65	171.73	0.90
DA 10	1 in 10 yr	159	6.26	10	12,329.16	1.23	0.65	171.73	0.39
DA 11	1 in 10 yr	159	6.26	7	20,457.73	2.05	0.65	189.25	0.70
DA 12	1 in 10 yr	159	6.26	7	11,526.33	1.15	0.65	189.25	0.40
DA 13	1 in 10 yr	159	6.26	7	13,198.74	1.32	0.65	189.25	0.45
DA 14	1 in 10 yr	159	6.26	7	14,808.68	1.48	0.65	189.25	0.51
DA 15	1 in 10 yr	159	6.26	6	6,580.11	0.66	0.65	195.75	0.23
DA 16	1 in 10 yr	159	6.26	10	28,785.37	2.88	0.65	171.73	0.90
DA 17	1 in 10 yr	159	6.26	7	11,862.38	1.19	0.65	189.25	0.41
DA 18	1 in 10 yr	159	6.26	15	76,122.00	7.61	0.65	147.84	2.05
DA 19	1 in 10 yr	159	6.26	7	11,273.76	1.13	0.65	189.25	0.39
DA 20	1 in 10 yr	159	6.26	15	51,433.22	5.14	0.65	147.84	1.38
DA 21	1 in 10 yr	159	6.26	10	32,125.87	3.21	0.65	171.73	1.00
DA 22	1 in 10 yr	159	6.26	7	7,726.15	0.77	0.65	189.25	0.27
DA 23	1 in 10 yr	159	6.26	10	13,562.06	1.36	0.65	171.73	0.42
DA 24	1 in 10 yr	159	6.26	10	20,491.79	2.05	0.65	171.73	0.64
DA 25	1 in 10 yr	159	6.26	10	29,032.49	2.90	0.65	171.73	0.91
DA 26	1 in 10 yr	159	6.26	10	19,837.94	1.98	0.65	171.73	0.62
DA 27	1 in 10 yr	159	6.26	7	5,868.17	0.59	0.65	189.25	0.20
DA 28	1 in 10 yr	159	6.26	7	6,906.81	0.69	0.65	189.25	0.24
DA 29	1 in 10 yr	159	6.26	7	8,465.74	0.85	0.65	189.25	0.29
DA 30	1 in 10 yr	159	6.26	7	21,385.52	2.14	0.65	189.25	0.74
SA 6S	1 in 25 yr	194	7.64	20	641,234.06	64.12	0.3	128.92	6.94

The location of each sub drainage area (DA) in table 1.8 is shown in Figure 3

Table 1.9 Maximum flow into drainage storm culverts of various sizes

Q_{max}	Units	Min Storm Drain size at 0.5% slope
0.42	m^3/s	60cm Dia HDPE pipe
0.77	m^3/s	75cm Dia HDPE pipe
1.25	m^3/s	90cm Dia HDPE pipe
2.7	m^3/s	120cm Dia HDPE pipe
4.9	m^3/s	150cm Dia HDPE pipe

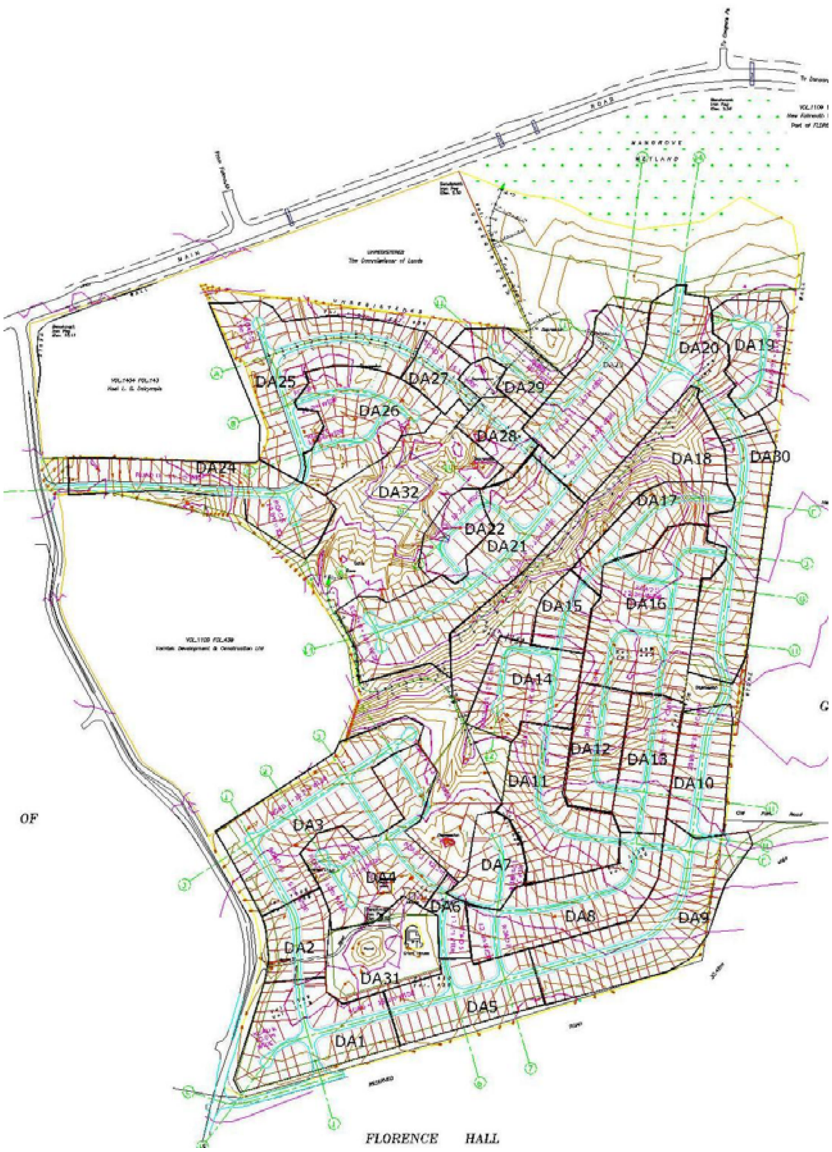

Figure 3 Drainage sub areas flowing to drain inlets or channels.

Conclusion

The drainage approach to allow for detention in some recreational areas in the development during infrequent storm events allow for a reduction in downstream peak flows and maintain the level of service for the North Coast Highway during heavy rainfall associated with such storms.

The highest culvert invert crossing the North Coast Highway is 0.634m as measured by the project Commissioned Land Surveyors. The height of water level rise in the wetland for the 1:50 year flow is estimated to be 1.04m making the estimated 1:50 water level in the wetland 1.67m. The surveyor measured the minimum road level as 2.10 or approximately 0.37m above the estimated top water level in the wetland.

The sewage treatment plant infrastructure will have a minimum tank wall height 0.75m above the 50year estimated top water level in the wetland area. Flows from the sewage treatment plant are planned to be discharged into the wetland at a peak rate 17.7Lps.

Prepared by:

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Foreman Chung and Sykes Consultants Limited

DATE: November 24, 2008