AF ENGINEERING

14 WORTHINGTON TERRACE, KINGSTON 5, JAMAICA
TEL: 876 920 7144, Mobile: 954 655 0282

GDL WHIM HOUSING DEVELOPMENT

DRAINAGE DESIGNS

CLIENT: GORE DEVELOPMENTS LIMITED
2C BRAEMAR AVENUE
KINGSTON 10

VERSION 1.0
June 25, 2009

Overview

Gore Developments Limited proposes to develop lands west of the Old Harbour Bay Road and South of Highway 2000 known as The Whim as a Housing Estate. The location of the proposed development is shown in **Figure 1**.

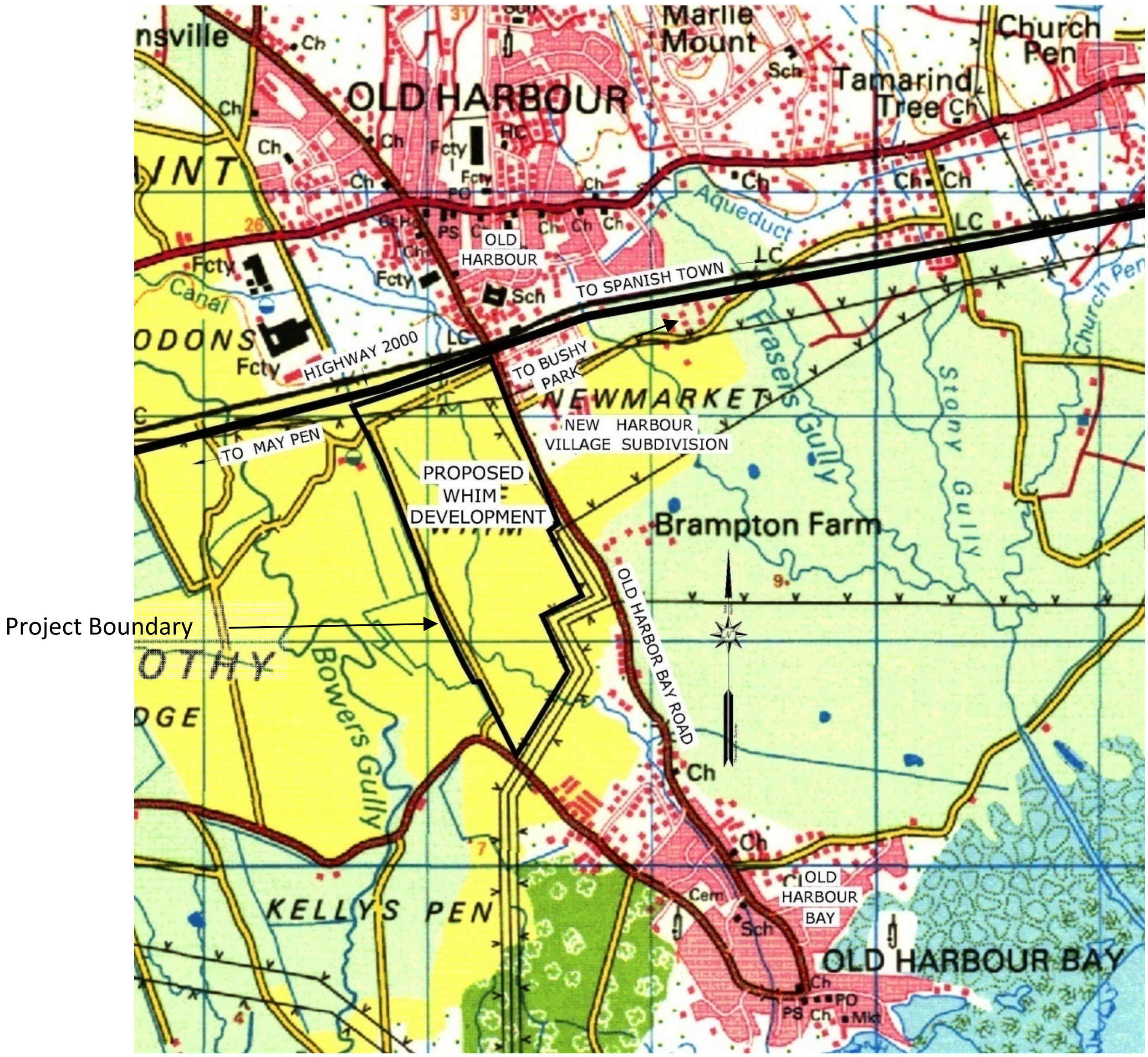


Figure 1 Location plan for the proposed GDL Whim Housing Development

The project lands are part of a flood plain where the Bower's Gully is the major drainage feature in that area. A Hydrology Consultant prepared a hydrologic model of the nearby Bowers Gully where the surface runoff from most of the proposed development will flow. AF Engineering was assigned by Gore Developments Limited to prepare the drainage designs and this report. Parts of this report are taken from a similar report prepared by this author while working for Foreman Chung & Sykes Consultants Limited for a previous version of the Master Plan. This report describes the methods used to develop the drainage designs and the main drainage features that define the drainage scheme. The effect of the storm water runoff from the proposed subdivision on the Bowers is included in the Hydrology Consultant's report prepared for a slightly different layout, where the confluence of the main drain was approximately 400m south of the current central main drain alignment.

Figure 2 Predevelopment Google Earth image of the proposed Whim development site showing key drainage features

Pre Development Drainage

The proposed development lands are part of a flood plain that slopes from north to south. Along the northern boundary of the proposed development is part of the Portmore to Sandy Bay Highway 2000 alignment. The Highway is raised above the natural ground and flows originating north of the highway flow through culverts in the highway embankment to the south. The main flow paths where large flows can cross the highway are the Bowers Gully and the Old Harbour Bay road underpass. Other drainage crossings include box and pipe culverts that discharge storm water onto the proposed development lands. **Figure 2** is a Google Earth image of the area with the main predevelopment drainage features described.

South of the highway the main watercourses are the Old Harbour Bay Road paved drain, the meandering natural drain that is generally centrally located on the proposed development lands and an earth drain on the western boundary of the proposed development. All those drains join the meandering central drain at various points. That drain referred to as the Whim drain currently splits into two watercourses just south of the development lands see **Figure 1**. Those watercourses fall towards Old Harbour Bay to the south and to the southern areas of the Brampton Farm lands to the east.

Post Development Drainage Scheme

The storm water that crosses the Highway 2000 alignment onto the proposed development lands will be collected into a central channel that will also receives storm flows from the proposed development lands and the western boundary drain and those combined flows will be directed toward the Bowers Gully. The western earth drain will be extended to discharge into the Central main drain. The storm water flows originating on the development lands as well as the flows crossing the highway has been redirected from flowing toward the existing community of Old Harbour Bay due to recent drainage upgrading works done in the area. With the proposed change of land use the increased surface flows will mostly be redirected into the Bowers Gully thereby removing the likelihood of flooding the newly built channel. This approach is consistent with the work done to reduce the frequency of flooding in the Old Harbour Bay community.

The most southern part of the proposed development identified as Block A in **Figure 4** is the only part of the proposed development that will continue to discharge storm water flows into the existing watercourse to the south. The flow will be significantly reduced as this is a relatively small catchment compared to the predevelopment condition. The Old Harbour Bay Road drain will not be altered and the existing drain to the south will only receive that flow and the flow from Block A. The drain to the south is now directed to the east across Old Harbour Bay main road onto the southern section of the Brampton Lands.

The design criteria used for this development is guided by the Local requirements for permitting and international standards with regards to similar developments.

Drainage criteria and methods of determining runoff

DESIGN STORM EVENT

The Government of Jamaica (GOJ) Development and Investment Manual, Volume 3 Section 1, Chapter 12, article 12.1, part (ix) set out the design storm return frequency as follows:

- Minor Drainage systems designed to accommodate 1 in 5 year flood event.
- Major Drainage system to be designed to accommodate 1 in 25 year flood event.
- Bridges designed for 1 in 50 year flood 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 surface drainage and inlet sizing is designed as local streets and the design event is the 1 in 5 year frequency.

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. 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.

2. **Drains to be sized to** United States Federal Highway Administration (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 x 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.

$$i = \frac{4.73 \times R}{(12.25+D)^{0.65}}$$

Where R = 24-hour rainfall.

D = Duration of the design rainfall event equal to the time of concentration

The runoff coefficients are taken from the **FHWA HEC 22 Table 3.1.**

The proposed development is located between the Old Harbour town and Old Harbour Bay however the rain data used is for Old Harbour town (see **Table 1**) as this is slightly higher than the other stations and part of the local water shed extends to the Old Harbour town.

Table 1 Rainfall Data

Old Harbour rainfall Data	
24 hr Return	**mm/day**
1 in 2 yr	105
1 in 5 yr	164
1 in 10 yr	203
1 in 25 yr	252
1 in 50 yr	288
1 in 100yr	324

NRCS TR-55 METHOD OF DETERMINING SURFACE RUNOFF PEAK FLOWS

The proposed development was superimposed on the Jamaica Survey Department 1:12,500 topographic map series for the area and the catchments that direct surface runoff toward the proposed development were delineated. Given the data available, catchment sizes and the times of concentration the USDA NRCS Urban Hydrology for Small Watersheds Technical Release 55 most commonly called the TR-55 method of determining the peak surface runoff flow was used to determine the peak 1:10 year and 1:25year flows.

This method requires the following inputs

1. Catchment area
2. Time of concentration and time of travel
3. Land use and soil type to determine the curve number CN
4. 24 Hour precipitation for the watershed considered.

[1.]Technical Release 55 (TR-55) presents simplified procedures to calculate storm runoff volume, peak rate of discharge, hydrographs, and storage volumes required for floodwater reservoirs. These procedures are applicable in small watersheds, especially urbanizing watersheds, in the United States.

[1.]The model described in TR-55 assumes a rainfall amount uniformly imposed on the watershed over a specified time distribution. Mass rainfall is converted to mass runoff by using a runoff curve number (CN). CN is based on soils, plant cover, amount of impervious areas, interception, and surface storage. Runoff is then transformed into a hydrograph by using unit hydrograph theory and routing procedures that depend on runoff travel time through segments of the watershed. First issued by the Soil Conservation Service (SCS) in January 1975, TR-55 incorporates current SCS procedures.

[1.] *Extract from the NRCS TR-55 Document available at* *http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/WinTR55.html*

A brief description of the curve number method and the curve numbers for various cover types and hydraulic soil groups were extracted from the NRCS TR-55 document is presented below in this report.

1.

SCS runoff curve number method

The SCS Runoff Curve Number (CN) method is described in detail in NEH-4 (SCS 1985). The SCS runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{[eq. 2-1]}$$

where

Q = runoff (in)
P = rainfall (in)
S = potential maximum retention after runoff begins (in) and
$\mathbf{I_a}$ = initial abstraction (in)

Initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. I_a is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, I_a was found to be approximated by the following empirical equation:

$$I_a = 0.2S \qquad \text{[eq. 2-2]}$$

By removing I_a as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting equation 2-2 into equation 2-1 gives:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{[eq. 2-3]}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and S is related to CN by:

$$S = \frac{1000}{CN} - 10 \qquad \text{[eq. 2-4]}$$

Figure 2-1 and table 2-1 solve equations 2-3 and 2-4 for a range of CN's and rainfall.

Table 2-2a Runoff curve numbers for urban areas [1]

Cover description: Cover type and hydrologic condition	Average percent impervious area [2]	Curve numbers for hydrologic soil group: A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1] Average runoff condition, and $I_a = 0.2S$.
[2] The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3] CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4] Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5] Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4 based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

[1]*. Extract from the NRCS TR-55 Document available at* http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/WinTR55.html

Table 2-2c Runoff curve numbers for other agricultural lands [1]

Cover description		Curve numbers for hydrologic soil group			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing and generally mowed for hay.	—	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 [4]	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	—	59	74	82	86

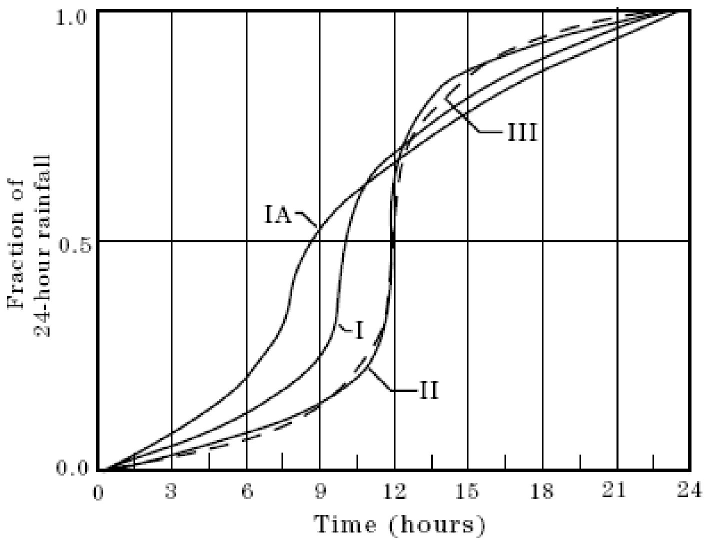
1 Average runoff condition, and $I_a = 0.2S$.
2 *Poor:* <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.
3 *Poor:* <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.
4 Actual curve number is less than 30; use CN = 30 for runoff computations.
5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.
6 *Poor:* Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.
Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Extract from the NRCS TR-55 Document available at
http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/WinTR55.html

[1.] The intensity of rainfall varies considerably during a storm as well as geographic regions. To represent various regions of the United States, NRCS developed four synthetic 24-hour rainfall distributions (I, IA, II, and III) from available National Weather Service (NWS) duration-frequency data (Hershfield 1061; Frederick et al., 1977) or local storm data. Type IA is the least intense and type II the most intense short duration rainfall.

The NRCS recommends the Type III distribution for parts of Florida where intense rainfall is mostly due to tropical storms a similar situation for the south coast of Jamaica. That distribution has been used for this hydrologic evaluation as high intensity rainfall due to tropical weather is a feature of local conditions. The type III hyetograph is shown in Figure B1 taken from the TR55 publication.

Figure B-1 SCS 24-hour rainfall distributions



The time of travel and time of concentration are parameters used to distribute the runoff into a hydrograph. The method is based on velocities of flow through segments of the watershed. Two major parameters are time of concentration (T_c) and travel time of flow through the segments (T_t). These and the other parameters used are the same as those used in accepted hydraulic analyses of open channels.

Travel time (T_t) is the time it takes water to travel from one location to another in a watershed. T_t is a component of time of concentration (T_c), which is the time for runoff to travel from the hydraulically most distant point of the watershed to a point of interest within the watershed. T_c is computed by summing all the travel times for consecutive components of the drainage conveyance system. The times are computed by the product of velocity of flow and the length of the reach of the watercourse considered. The Time of concentration is made up of the sheet flow near the watershed divide, the shallow concentrated flow assumed to occur in this assignment at 15m from the watershed divide and the flow in the drainage channel. Manning's equation is used to estimate the velocity of flow in the drainage path.

The equation below is taken from the appendix of the TR 55 that is used to estimate the stream velocity for shallow concentrated flow in a watershed. Equations are also presented in the TR 55 document to estimate the velocity of the sheet flow that occurs at the watershed divide of the catchments. The Manning's equation shown in the design section of this document is used to estimate the flows in the channel.

Figure 3-1 (average velocities for estimating travel time for shallow concentrated flow):

Unpaved $V = 16.1345\,(s)^{0.5}$

Paved $V = 20.3282\,(s)^{0.5}$

where

V= average velocity (ft/s)

s = slope of hydraulic grade line (watercourse slope, ft/ft)

The Tabular Hydrograph methods was used to estimate the peak flow for the various catchments along this part of the alignment.

[1.] The Tabular method can develop partial composite flood hydrographs at any point in a watershed by dividing the watershed into homogeneous subareas. In this manner, the method can estimate runoff from non-homogeneous watersheds.

[1.] An assumption in development of the tabular hydrographs is that all discharges for a stream reach flow at the same velocity. By this assumption, the subarea flood hydrographs may be routed separately and added at the reference point. The tabular hydrographs are pre-routed hydrographs.

To develop a tabular hydrograph discharge summary; the effect of individual subarea hydrographs are routed to the watershed point of interest. Use ∑ Tt for each subarea as the total reach travel time from that subarea through the watershed to the point of interest. Compute the hydrograph coordinates for selected ∑ Tt's using the appropriate tabular hydrograph unit discharges for type II rainfall distribution. The flow at any time is:

$$q = q_t A_m Q \qquad \text{[eq. 5-1]}$$

where:

q = hydrograph coordinate (cfs) at hydrograph time t

q_t = tabular hydrograph unit discharge from exhibit 5 (csm/in)

A_m = drainage area of individual subarea (mi^2)

Q = runoff (in)

A computer model of each method was used that operates with SI units.

[1]. *Extract from the NRCS TR-55 Document available at* *http://www.wsi.nrcs.usda.gov/products/W2Q/H&H/Tools_Models/WinTR55.html*

ROADWAY DRAINAGE

Caltrans recommends in index 831.4 that concentrations of sheet flow across roadways are to be avoided. As a general rule, no more than $0.003m^3/s$ should be allowed to concentrate and flow across a roadway.

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.

The **GOJ Development Manual**, Volume3, Section 1, Chapter 10, article 10.1.7 parts ii) and iii) recommend minimum easement and freeboard in drains as shown below:

(ii) A minimum easement of 1.22m from each side of the design water way is recommended.

(iii) Bridges and open channels should be designed with a freeboard not less than 25% of the design flow depth.

As recommended in the GOJ Development document all drains will be designed with a minimum 25% of the design depth as freeboard.

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.

Table 873.3D is an example of the limiting of channel velocity based on lining type.

	Table 873.3D		
	Channel Linings		
Mean Velocity (m/s)	Thickness of Lining (mm)		Minimum Reinforcement
	Sides	Bottom	
Portland Cement Concrete or Air Blown Mortar			
< 3	75 -90	90 - 100	152x152-MW19.4 x MW19.4 welded wire Fabric
3 - 4.5	100 -125	125 -150	#15 Bars at300 mm and 450 mm centers
4.5 or more	150 - 200	175 - 200	#10 Bars at300 mm centers both ways

DRAINAGE DESIGN

Pre development condition

The predevelopment land use condition of the proposed development lands is deemed to be brush with mixed grass and weed in fair condition. Figure 2 a recent satellite image shows the existing land use.

The development lands are bounded by a raised highway to the north with box and pipe culverts allowing storm water to cross the alignment, collector roadway (Old Harbour Bay Road) to the east with parallel paved drain, urbanizing lands to the south and an existing unlined earth drain on the western boundary of the proposed development property.

The proposed drainage scheme for the development is to collect the storm flows coming through the highway drainage culverts that impact the site, into a drain centrally aligned through the proposed development that will also receive flows from the various proposed housing blocks and ultimately discharge the flows into the Bowers Gully west of the proposed development.

There are three culverts that cross the highway that directly impact the proposed development, a 2.44m wide by 1.5m ht RC box culvert and 0.914m diameter concrete pipe culvert that cross the highway and discharge flows into a meandering earth drain on the project lands and a 1.22m diameter concrete pipe culvert that crosses the highway and discharges into a straight earth drain on the western boundary of the project lands. The Bowers gully is approximately 500m west of the project lands and seems not to affect the project lands. The location of the existing drainage features are shown in **Figure 2**.

Photos 1 to 4 show the three culverts of concern crossing the Highway and the downstream drain on the western boundary of the proposed development.

Photo 1- 2.4mWide x 1.5mHt RC Box Culvert below H2k

Photo 2 - 0.914m Dia. RC pipe culvert below H2k

Photo 3 - 1.22m Dia. RC pipe below H2k

Photo 4 - Existing drain downstream of the 1.22m Dia. culvert

The Proposed Developed Condition

For the detailed drainage design the culverts constructed across the Highway were taken into account and the likely peak discharge assessed.

An estimate of the upper limit of flow that the culverts that cross the Highway 2000 embankment was estimated by assuming that inlet control will be the limiting factor and that the inlets to the culverts are submerged by 2m.This is to assume that the top water level above the invert of the inlet to the 1.5m height box culvert is 3.5m. That being the approximate height the water level will possibly rise to overtop the ridge of the watershed draining to the culvert and flow into the adjoining Bowers gully watershed to the west or the Old Harbour Bay road watershed to the East neither of which convey flow onto the site. The flow crossing the highway was determined in the TR-55 model by modeling the crossings as a weir.

The western boundary drain is proposed to remain in its preconstruction state with a boundary wall or earth berm incorporated in the western access road reserve located along the western boundary of the proposed development. That western drain will be connected with the main drain and those flows included with the flows from the subdivision and beyond directed into the Bowers Gully.

The estimated maximum storm flows that will cross the Highway are shown below in table 2.

Table 2

Estimate of maximum storm flow through culverts crossing the H2k road			
Item	Description	Headwater (m)	Estimated max flow (m^3/s)
1	2.4m W x 1.5m Ht Box Culvert	3.75	17.45
2	0.9m Diameter RC Culvert	3.75	3.28
3	1.2m Diameter RC Culvert	3.45	5.725

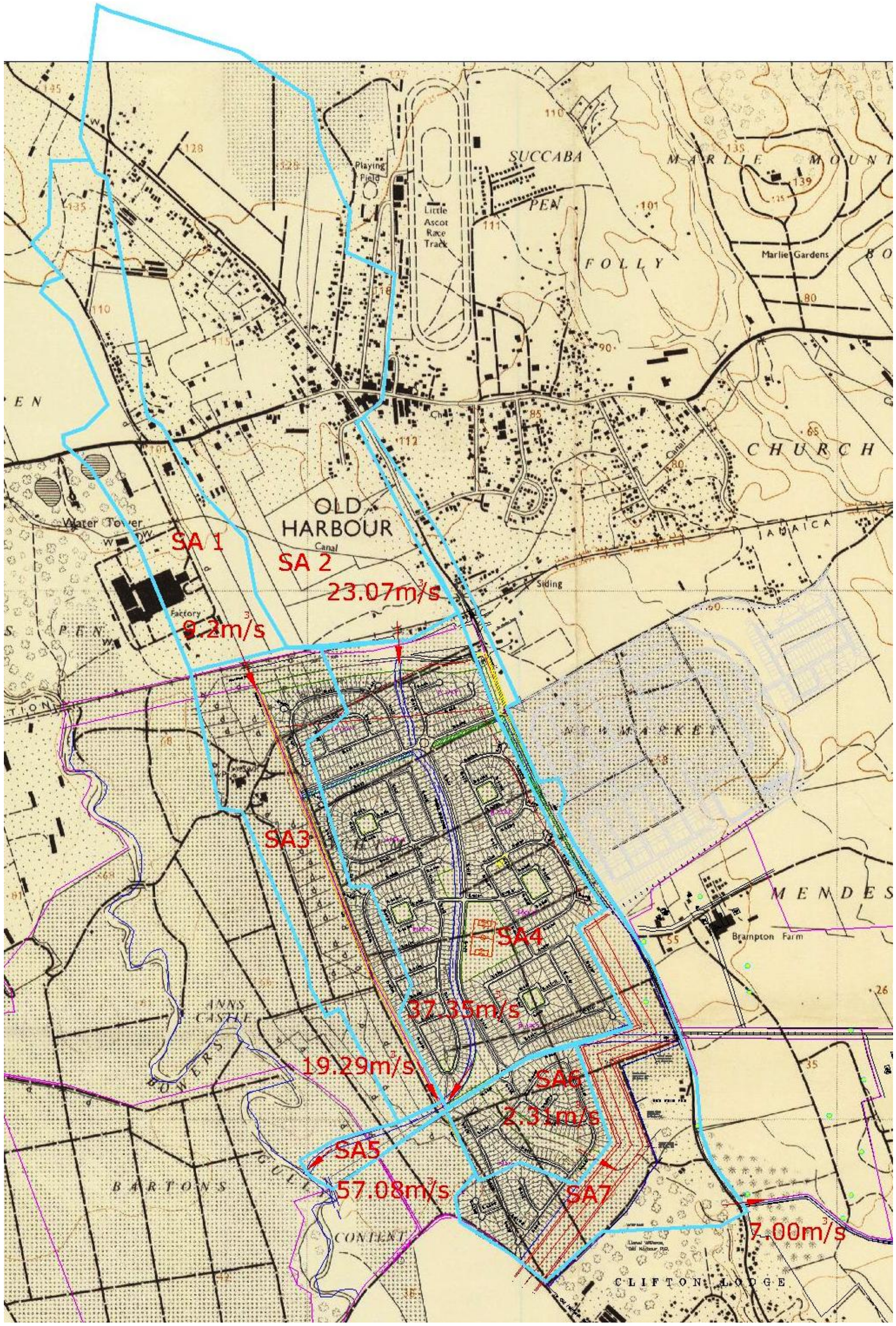


Figure 3 shows Main drainage sub areas and routed downstream flows if area is fully urbanised and taking into account the effect of the Highway 2000 culverts.

Figure 4 Proposed Whim Development Layout.

With the flows crossing the highway evaluated the flows from the various development blocks were added at discrete points along the central main drain alignment to determine the flow in the main drain. This method was used to estimate the flows in the main drain at various points along the drain alignment or reaches of the drain. The size of the reaches was determined using the Manning's method of determining the velocity of flow in a channel. The block layout is shown in **Figure 4** and the estimates of flows from the various blocks are shown in **Table 6**.

The developed scenario was modeled in TR-55 and where the catchments were divided into seven sub areas as shown in figure 3. Sub areas 1,2,3,4 and 5 are all connected and the flows from there are directed into to Bowers Gully. The details of the sub areas hydrologic condition is shown in **Table 3**.

Table 3

AF ENG	**GDL WHIM DEVELOPED SCENARIO**				
	Type III rain distribution				
Region: OLD HARBOUR			Locale: SAINT CATHERINE		
	Sub-Area Land Use and Curve Number Details				
Sub-Area Identifier	Land Use	Condition	Hydrologic Soil Group	Sub-Area Area (ha)	Curve Number
SA1	Open space; grass cover < 50%	(poor)	C	6.6	86
	Residential districts (1/8 acre)		C	26.4	90
	Total Area / Weighted Curve Number			33	89
SA2	Open space; grass cover < 50%	(poor)	C	43.28	86
	Residential districts (1/4 acre)		C	64.92	83
	Total Area / Weighted Curve Number			108.2	84
SA3	Residential districts (1/8 acre)		C	31.2	90
SA4	Open space; grass cover < 50%	(poor)	C	2.3	86
	Residential districts (1/8 acre)		C	68	90
	Total Area / Weighted Curve Number			70.3	90
SA5	Open space; grass cover < 50%	(poor)	C	1.5	86
	Residential districts (1/8 acre)		C	1.8	90
	Total Area / Weighted Curve Number			3.3	88
SA6	Residential districts (1/8 acre)		C	12.2	90
SA7	Open space; grass cover < 50%	(poor)	C	18	86
	Residential districts (1/8 acre)		C	21.9	90
	Total Area / Weighted Curve Number			39.9	88

The runoff from subarea 2 is controlled by the box culvert and that feature is added to the model by including a weir structure at the downstream end of the subarea and is identified as weir NE in table 5. The anticipated flows in the main drainage features from the subdivision are shown in **Table 4**

TABLE 4

AF ENG	GDL WHIM DEVELOPED SCENARIO			
Type III rain distribution				
Region: OLD HARBOUR		Locale: SAINT CATHERINE		
	Hydrograph Peak/Peak Time Table			
Peak Flow and Peak Time (hr) by Rainfall Return Period				
Sub-Area or Reach Identifier	(hr)	10-Yr (cms)	(hr)	25-Yr (cms)
SA1	12.58	7.26	12.58	9.2
SA2	12.72	20.6	12.69	26.61
SA3	12.32	9.48	12.31	11.97
SA4	12.36	19.97	12.37	25.26
SA5	12.16	1.27	12.17	1.61
South Drainage				
SA6	12.21	4.34	12.21	5.48
SA7	12.48	9.8	12.46	12.45
REACHES	(hr)	10-Yr (cms)	(hr)	25-Yr (cms)
R3	12.41	15.39	12.41	19.49
Down	12.54	15.23	12.53	19.29
R4	12.51	28.76	12.51	37.38
Down	12.54	28.74	12.55	37.37
R5	12.54	44.4	12.53	57.23
Down	12.55	44.28	12.54	57.08
R2	12.72	20.6	12.69	26.61
Down	12.99	17.29	12.96	23.07
OUTLET to Bowers Gully		44.28		57.08
REACHES SOUTH				
R1	12.21	4.34	12.21	5.48
Down	12.44	2.84	12.5	3.06
R2	12.48	12.63	12.48	15.5
Down	12.98	6.29	13.1	7
R3	12.98	6.29	13.1	7
Down	13.02	6.29	13.14	7
OUTLET to Brampton Drain		6.29		7

TABLE 5

Reach Idendifier	Stage (m)	Pool Storage (ha m)	Flows(cms) @ Weir Length Length (2.4m)
WEIR NE	0	0	0
	0.15	0.15	0.221
	0.3	0.3	0.624
	0.61	0.61	1.766
	1.52	1.52	6.98
	3.05	3.05	19.742
	6.1	6.1	55.84
Reach Idendifier	Stage (m)	Pool Storage (ha m)	Flows (cms) @ Pipe Diameter 900mm
PIPE 1	0	0	0
	0.38	0.11	2.625
	0.75	0.22	2.821
	1.5	0.45	3.176
	3.75	1.13	4.06
	7.5	2.25	5.21
	15	4.5	6.961
Reach Idendifier	Stage (m)	Pool Storage (ha m)	Flows (cms) @ Pipe Diameter 1220mm
PIPE 2	0	0	0
	0.46	0.37	4.746
	0.91	0.73	5.187
	1.83	1.46	5.973
	4.57	3.66	7.874
	9.14	7.31	10.288
	18.28	14.62	13.913

It is proposed to include detention features in the southern drainage areas of the site that discharge onto the Brampton lands. A detention feature is proposed in Block A and a second within the JPS reserve. The ponds are created by converting deep ravines from the existing Whim drain by constructing embankments with pipes that cross the embankments to control the downstream peak discharge. **Table 5** shows the stage and flows that are expected to flow through the culverts from the detention features given the size of detention feature. The down stream flows are shown in **Table 4.** The inclusion of these features reduces the downstream flow and increases the infiltration potential thereby enhancing the possibility of aquifer recharge.

Table 6

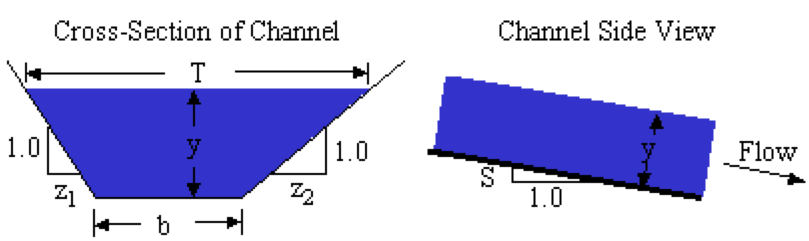
GDL WHIM - CENTRAL MAIN DRAIN SIZES							
No.	**REACH**	**Description**	**Bottom Width** m	**Height** m	**depth of flow**	**Q m3/s**	**slope**
1	0+000 - 0+620	Rectangular Reinforced Concrete	6.00	1.50	1.03	32	0.64%
2	0+620 - 1+320	Rectangular Reinforced Concrete	6.00	1.85	1.47	37	0.30%
3	1+320 - 1+420	Rectangular Reinforced Concrete	9.00	1.80	1.43	57	0.30%
4	1+420 - Bowers Gully	Trapezoidal Gravel lined	9.00	2.50	1.95	57	0.20%

N.B. All dimensions are internal dimensions

The central collector drain flows and drain sizes are shown in **Table 6**. The drain sizes were determined using the Manning's method of determining channel velocity and the method is outlined in the following section of the report. The Housing Block drainage is presented in the subdivision drawings. The micro catchments were delineated and the flows estimated using the rational method. The pipe culverts and open drains presented in the drawings submitted were sized to meet the criteria set out in this document.

Manning's Open Channel drain design method

The Manning's method of determining velocity of channel flow is used to size the drain cross sections. An outline of the method is shown below.



$$Q=VA \qquad V=\frac{k}{n}R^{2/3}S^{1/2} \qquad R=\frac{A}{P} \qquad A=\frac{y}{2}(b+T)$$

$$P=b+y\left(\sqrt{1+z_1^2}+\sqrt{1+z_2^2}\right) \qquad T=b+y(z_1+z_2)$$

$$F=V\sqrt{\frac{T}{gA\cos\theta}} \qquad \theta=Tan^{-1}(S)$$

Variables [] indicates dimensions

A = Flow cross-sectional area, determined normal (perpendicular) to the bottom surface [L^2]

b = Channel bottom width [L].

F = Froude number. *F* is a non-dimensional parameter indicating the relative effect of inertial effects to gravity effects.

Flows with $F<1$ are low velocity flows called subcritical. $F>1$ are high velocity flows called supercritical. Subcritical flows are controlled by downstream obstructions while supercritical flows are affected by upstream controls. $F=1$ flows are called critical.
g = acceleration due to gravity = 32.174 ft/s^2 = 9.8066 m/s^2. *g* is used in the equation for Froude number.
k = unit conversion factor = 1.49 if English units = 1.0 if metric units.

n = Manning coefficient

P = Wetted perimeter [L]. *P* is the contact length between the water and the channel bottom and sides.

Q = Discharge or flowrate [L^3/T].

R = Hydraulic radius of the flow cross-section [L].

S = Slope of channel bottom or water surface [L/L]. Vertical distance divided by horizontal distance.

T = Top width of the flowing water [L].

V = Average velocity of the water [L/T].
y = Water depth measured normal (perpendicular) to the bottom of the channel [L

z_1, z_2 = Side slopes of each bank of the channel.

The manning friction coefficients used are shown in the table below.

Material	**Manning n**	**Material**	**Manning n**
Natural Streams		*Excavated Earth Channels*	
Clean and Straight	0.030	Clean	0.022
Major Rivers	0.035	Gravelly	0.025
Sluggish with Deep Pools	0.040	Weedy	0.030
		Stony, Cobbles	0.035
Metals		*Floodplains*	
Brass	0.011	Pasture, Farmland	0.035
Cast Iron	0.013	Light Brush	0.050
Smooth Steel	0.012	Heavy Brush	0.075
Corrugated Metal	0.022	Trees	0.15
Non-Metals			
Glass	0.010	Finished Concrete	0.012
Clay Tile	0.014	Unfinished Concrete	0.014
Brickwork	0.015	Gravel	0.029
Asphalt	0.016	Earth	0.025
Masonry	0.025	Planed Wood	0.012
		Unplaned Wood	0.013
Corrugated Polyethylene (PE) with smooth inner walls			0.009-0.015
Corrugated Polyethylene (PE) with corrugated inner walls			0.018-0.025
Polyvinyl Chloride (PVC) with smooth inner walls			0.009-0.011

Conclusion

The primary drainage feature for this development is the Central Main Drain. That central drain collects storm water flows originating upstream of the proposed development and the flows from the development blocks and convey them to the Bowers Gully.

The development blocks will be graded to fall toward storm sewers or minor paved drains that will fall toward the central drainage system. If the drainage system for the development sub areas called blocks is obstructed by debris the finished ground is to be graded to allow for storm water to overtop the drainage infrastructure and to flow toward the Central Main Drain and out to the Bowers Gully. This feature will limit the extent of flooding during storm events that exceed the design runoff event.

The main drainage feature in the area is the Bowers Gully and with its fairly large estuary will naturally improve storm water quality before discharge into the Sea.

A separate hydrologic evaluation has been prepared for the effects the flows from the gully may have on the proposed development and the effect of the flow from the development on the Gully.

The data presented and the designs forwarded in this report ought to provide an adequate description of the drainage features required for the successful development of this property as a housing estate.

Prepared by:

Ivan Andrew Foreman P.E.

AF ENGINEERING

DATE: June 25, 2009