GIG HARBOR BASIN PLAN

TECHNICAL MEMORANDUM

Subject: Analysis of flood hazard.

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Introduction

In Phase 1 of the basin planning program, information on existing flooding was compiled. In Phase 2, a mathematical model was developed and used to analyze existing and expected future flood problems. Due to the cost of the data acquisition to develop models, not all streams in Gig Harbor were modeled and only stream basins with the greatest potential for future development and flooding were included in this study. This memorandum describes how the model was developed and provides preliminary results for the following streams:

- Artondale Creek
- Crescent Creek
- Donkey Creek
- Gloodnough Creek
- McCormick Creek
- Nelyaly Creek
- Rosedale Creek
- Sullivan Creek
- Warren Creek
- Wollochet Creek

Selection of Analytical Approach

Most storm water runoff in the Gig Harbor Basin is routed to streams that flow to Puget Sound. Natural drainage patterns remain largely unaltered although many culverts have been built to carry stream flow under roads and driveways. Curbs, gutters and underground storm drains exist only in the more densely developed areas. Storm water runoff in older rural communities and suburban neighborhoods is typically routed to roadside ditches and then on to natural streams. Many streams flow through fairly narrow canyons where streamside properties are generally located a considerable distance above the water level. Where the flood plain is broader, wetlands often exist and are a deterrent to development. As a result of these physical circumstances and careful urban planning, few homes or businesses appear to be located within floodplains. There are no records of damaging flooding in the Gig Harbor Basin. All reported flooding is relatively minor and probably results from temporary blockages of culverts rather than serious drainage deficiencies. However, because urban development in the basin is expected to continue, with a concomitant increase in storm water runoff, flooding could occur in the future. The purpose of the flood hazard analysis described in this memorandum is to predict the location and severity of potential future flooding.

Several different approaches to analysis were evaluated. One approach would be to develop a hydraulic model that would predict the water surface elevation at all points along each stream during design storms. A model of this type is only as accurate as the topographic information that it is based upon. Although two-foot contour interval topography is available for most of the Gig Harbor Basin, the accuracy of the contouring is questionable in the narrow canyons that contain many of the streams. The topography was derived from aerial photographs. Interpretation of aerial photographs can be difficult in heavily wooded areas where the terrain is steep. Topography of sufficient accuracy to serve as the basis for a hydraulic model would have to be obtained by a survey party using conventional surveying instruments. Channel cross-sections would need to be taken at longitudinal intervals of no more than 500 feet and at every major change in channel form. A topographic survey of this type for the Gig Harbor Basin's approximately 100,000 linear feet of stream would cost several hundred thousand dollars. The expense associated with this approach is only justified in circumstances where flood hazard is high. Because of the relatively low risk of flooding in the Gig Harbor Basin this approach was rejected.

To make best use of the available resources an analytical approach was developed that focuses on the hydraulic performance of publicly owned culverts on the major streams that drain the Gig Harbor Basin. Road culverts are the most likely cause of serious flooding in the basin. The Army Corps of Engineer's HEC-HMS model was used to determine whether the existing culverts have sufficient capacity to pass current and expected future stream flows without flooding and whether they are in conformance with Pierce County's storm water design standards. The results of the analysis will serve as a basis for development of a list of capital improvements. The analytical method is described in detail below.

Analytical Method

<u>Overview</u>

The Army Corps of Engineer's HEC-HMS model (Version 2.1.2) was used to generate inflow hydrographs for each culvert. The Soil Conservation Service (SCS) unit hydrograph method was used to develop inflow hydrographs. The model was then used to calculate the maximum upstream headwater depth by routing each hydrograph through a "reservoir" with the culvert acting as an outlet control structure. Two-foot topographic contour data included in the County's geographic information system (GIS) files were used to delineate sub-basins, determine the runoff characteristics of each sub-basin, and calculate storage volumes upstream of each structure. The hydraulic information for culverts (i.e., culvert size, material, the relative elevations of culvert inters and road crests) were based on field survey data collected by URS. The hydraulic information for culverts on McCormick Creek under

SR 16 was derived based on the as-built information from Washington Department of Transportation.

Stage-discharge relationships were developed from US Bureau of Reclamation (USBR) culvert nomographs and from standard weir-flow equations. A combination of field and GIS data were used in conjunction with the nomographs to develop a unique elevation-storage-discharge rating for each structure analyzed. Model runs were performed for the 2-, 25-, and 100-yr SCS Type 1A synthetic design storms.

Culverts selected for analysis

Culverts conveying flow under Pierce County roads were modeled. Culverts to convey flow under private residences, state highways, and within the City of Gig Harbor were excluded from the model. All culverts included in the field survey were assigned an ID consisting of two letters denote to the basin and a number (e.g., WC-01 is located on Wollochet Creek). However, a numbering system to describe a particular location along the stream was not consistently applied in all sub-basins (e.g., WC-01 is furthest downstream, but AD-01 (Artondale basin) is furthest upstream. A list of all culverts analyzed is included in Table 1. They are also shown on the accompanying map.

The HEC-HMS model

The HEC-HMS model (public domain, version 2.1.2, June, 2001) is a newer hydrologic model designed to run on a Windows platform. The model performs all of the functions of the older HEC-1 model.

Design storms

Three synthetic design storms were applied using the SCS type 1A distribution. Rainfall depths were obtained from the Pierce County Storm Water Design Manual (page E-3). Rainfall depths for the 2-, 25-, and 100-yr events are 2.5, 4.0, and 4.8 inches, respectively.

Basin/sub-basin delineation and nomenclature

Basin names were established in Phase I. Basin boundaries were originally delineated from 20-ft contour mapping. In some areas, basin boundaries were found to be not in agreement with the recently obtained 2-foot topographic data, and were re-delineated accordingly. Each basin was further divided into smaller sub-basins using 2-ft topographic data with either a culvert or a natural hydraulic control (such as a lake, a lake systems, or wetland) at the outlet.

Curve number determination

The soils occurring in entire area of study belong to the Harstine Series (USDA Soil Survey of Pierce County, updated 1994). This soil type is described as having a "hardpan" layer that inhibits infiltration. Thus, for a significant storm event, a relatively large proportion of rainfall runs off as surface water, and all sub-basins were assumed to consist of hydrologic group "C" soils.

The distribution of land-use for each sub-basin was obtained from GIS data provided by the County. Each land-use category was assigned an SCS curve number. An area-weighted curve number was calculated for each sub-basin based on the distribution of land use. Curve numbers recommended in the Pierce County Storm Water Design Manual were originally used, but were judged to be too conservative (i.e., the model generated unreasonably high peak flows for the 2-yr return interval; and worse for the less frequent events). Curve number values for each land-use type were reduced to agree with those recommended by SCS (Urban Hydrology for Small Watersheds, 1988, p. 2-5). Values of the initial abstraction (initial loss) are a function of the curve number and were determined for each sub-basin using the relation presented in the same SCS publication.

Time of concentration determination

The time of concentration is the time that elapses from the beginning of a storm event until the entire watershed is contributing to runoff. Time of concentration for each sub-basin was estimated following methodology presented in the Pierce County Storm Water Design Manual. The HEC-1 and HEC-HMS models both require the SCS lag time as input, which is generally accepted to be the time of concentration multiplied by 0.6.

Baseflow

No baseflow was the assumed condition for all streams.

Channel routing

No channel routing algorithms were used in the modeling due to lack of adequate channel cross section data. Thus, multiple inflow hydrographs above culverts were assumed to arrive at the inlet simultaneously, thereby generating a more conservative (higher) peak flow.

Lakes and wetlands, as hydraulic controls

Natural lakes (or a series of lakes) and wetlands were modeled in situations where appropriate. Elevation-storage relationships for each lake or wetland were determined from the 2-ft topographic GIS data. An assumed stage-discharge relationship was developed from available data using either uniform flow or weir-flow equations.

Culvert hydraulic characteristics

All culverts were analyzed assuming inlet control except for several of the most downstream culverts that are subject to tidal influence. It was assumed that these culverts may be submerged or partly submerged during the design storm and so would operate with outlet control. Stage-discharge relationships were determined using the appropriate U.S. Bureau of Reclamation culvert nomograph for each culvert.

The storage volume upstream of each culvert was computed in two-foot increments to the estimated road elevation using the two-foot contour interval topographic data. Discharge during an overtopping condition was assumed to consist of a fully flowing culvert and weir-flow over the road.

$$\text{Total discharge} = \text{Full culvert flow at road crest elevation} + 3.33*(\text{assumed road width})*(\text{stage above road crest})^{1.5}$$

Model limitations

Certain limitations affect the accuracy of the hydrology/hydraulics models. They include:

- Local storage was excluded except for storage immediately above culverts and in large lakes and wetlands
- Curve numbers are based on empirical relations and determination of value is somewhat subjective
- Inaccuracies in the available topographic information create some uncertainty with respect to estimated water surface elevations
- The 2-foot topographic data was not available for most of the Donkey and McCormick Creek watersheds. This created more uncertainties in generating storage volume for the culverts in these two watersheds
- No attenuation in channels (no routing)
- Likely errors estimating ratings of natural hydraulic controls

Results

Peak flows and corresponding maximum upstream water levels are shown in Table 1. Water levels are expressed as a "stage" value instead of an elevation because accurate survey information is not available. The table shows the results of modeling the 2-, 25- and 100-year return frequency storms. The design storm for road culverts is the 100-year storm.

The three columns at the extreme right of the table show information for the 100-year storm. The column labeled “estimated freeboard to road crest” indicates the predicted vertical distance between the water surface in the 100-year storm and the crest of the road. Where negative numbers are shown floodwater would be expected to overtop the road.

The next column to the right shows the headwater-to-diameter ratio. The headwater depth is the depth of water over the culvert invert during the 100-yr event. Pierce County storm drainage standards require that the headwater-to-diameter ratio not exceed 1.5. Stated another way, a culvert should not be surcharged by more than half of its diameter. The last column indicates whether individual culverts meet the standard.

Discussion and conclusions

As indicated in the table, the model predicts that road flooding could occur at six culverts. The culverts that could cause road flooding include the following:

- The 12-inch diameter culvert that carries Artondale Creek under Hunt Road (AD-01)
- The 54” diameter culvert crossing at Artondale Drive (AD-02)
- The 24-inch diameter culvert at 144th Street Court on Goodenough Creek (GN-02)
- The 36-inch diameter culvert that carries Nelyaly Creek under 82nd Avenue (NL-01)
- The 36-inch diameter that carries the south tributary of Donkey Creek under Harborview Drive
- The two 36” diameter culverts at Sehmel Drive on McCormork Creek.

A field visit was made to verify that the physical characteristics of each culvert and its surroundings are accurately reflected in the model.

Twenty-five of the 39 culverts analyzed do not meet Pierce County’s current design standards. Three of these culverts would be subject to considerable surcharging during the 100-year storm but road flooding would not occur (i.e., water depth less than 1 foot from top of the road). Although it would be desirable that all culverts met current design standards, the consequences of non-compliance are not expected to be serious. Road fills are not typically designed to function as dams but can be expected to do so successfully on rare occasions and for short periods of time.

In general, our conclusion with respect to the areas of the Gig Harbor Basin that we have analyzed so far is that continued urban development is not likely to cause serious flooding problems. Culverts that could cause road flooding and those that are most seriously surcharged will be included in the capital improvement program. But their priority for replacement is likely to depend more on whether they are in poor condition and represent a barrier to fish passage than on the relatively small flood hazard they pose.

GIG HARBOR BASIN PLAN - ESTIMATED CULVERT ADEQUACY
Pierce Co. Stormwater Design Curve Numbers

Table 1
HEC-HMS Modeling Results

Culvert Name	Culvert Diameter (in)	Culvert Type	Location / Description	Vert. Dist. U/S Invert to Road Bed (ft)	U/S invert (ft)	2-Yr Peak Flow (cfs)		Max Stage Above Invert (ft)	Est. Freeboard to Road Crest (ft)	25-Yr Peak Flow (cfs)		Max Stage Above Invert (ft)	Est. Freeboard to Road Crest (ft)	100-Yr Peak Flow (cfs)		Max Stage Above Invert (ft)	Est. Freeboard to Road Crest (ft)	Headwater/Diameter Ratio	Meets Design Std?
						Inflow	Outflow			Inflow	Outflow			Inflow	Outflow				
AD-01	12	12" CSP	Hunt St.	7	170	14	13	7.05	0.0	43	42	7.32	**-0.3**	67	66	7.48	**-0.5**	7.5	NO
AD-02	54	54" CMP	Artondale Dr.	12	28	50	49	3.15	8.9	162	145	7.67	4.3	238	184	11.71	0.3	2.6	NO
AD-04	72	6'X6' Box	Wollochet Dr.	30 (c)	4	62	62	2.42	27.6	168	168	4.92	25.1	215	215	5.82	24.2	1.0	YES
CR-01**	72	6'X6' Box	96th Ave	30	0	131	131	6.44	23.6	377	375	9.04	21.0	539.0	523.0	11.96	18.0	2.0	NO
CR-02	72	5'x12' Box	Crescent Val. Dr.	7	54	98	98	2.18	4.8	287	287	4.49	2.5	413.0	413.0	5.91	1.1	1.0	YES
CR-05	48	4'X12' Box	Crescent Val. Dr.	5	78	58	58	1.42	3.6	162	160	3.03	2.0	231	228	3.92	1.1	1.0	YES
CR-09*	72	6' CMP	Crescent Lk. Outlet*	7	163	43	19	3.45	3.6	148	46	4.09	2.9	217	63	4.47	2.5	0.7	YES
GN-01	24	24" CSP	144th St@51st Ave	20 (c)	294	38	35	5.59	14.4	109	58	14.52	5.5	153	69	20.06	**-0.1**	10.0	NO
GN-02	24	24" CSP	144th St Ct	9	282	35	29	4.15	4.9	58	55	9.07	**-0.1**	69	63	9.13	**-0.1**	4.6	NO
GN-05	42	42" CMP	54th Ave	15 (a)	200	51	51	3.74	11.3	128	110	10.03	5.0	175	134	13.89	1.1	4.0	NO
GN-06	48	48" CMP	SR 16	15 (a)	136	63	63	3.54	11.5	139	137	7.94	7.1	169	162	12.14	2.9	3.0	NO
GN-08	48	48" CSP	Good Enough Dr.	30	28	79	79	4.19	25.8	166	164	9.42	20.6	193	191	11.94	18.1	3.0	NO
GN-09	48	54" CSP	Purdy Dr	40	14	86	86	4.0	36.0	185	184	6.83	33.2	218	216	10.25	29.8	2.6	NO
NL-01	36	36" CSP	82nd Ave.	5	12	21	21	1.91	3.1	74	65	4.9	**0.1**	110	109	5.34	**-0.3**	1.8	NO
RD-01	48	48" CSP	Rosedale St.	7	10	9	9	2.24	4.8	28	28	2.75	4.3	40	40	3.07	3.9	0.8	YES
SL-01	30	30" CSP	East Bay Dr.	60 (c)	18	82	60	6.86	53.1	205	98	13.71	46.3	285	106	17.19	42.8	6.9	NO
SL-02**	30	30" CSP	Sullivan Dr.	25 (c)	6	61	61	7.93	17.1	102	100	16.47	8.5	110	109	19.18	5.8	7.7	NO
WC-01	48	(2) 48" CSP	East Bay Dr.	15 (c)	16	71	71	2.95	12.1	139	139	3.86	11.1	163	163	4.32	10.7	1.1	YES
WC-02	60	60" CMP	Wollochet Dr.	20 (c)	36	69	69	3.56	16.4	135	135	5.82	14.2	157	157	6.86	13.1	1.4	YES
WC-04	36	36" CSP	Spruce Ln.	15 (c)	58	64	42	3.27	11.7	168	82	6.45	8.6	231	94	8.49	6.5	2.8	NO
WC-05	30	30" CMP	57th St.	12 (a)	100	34	24	2.99	9.0	106	46	6.26	5.7	152	56	8.18	3.8	3.3	NO
WC-06	48	48" CMP	Hunt St.	25 (c)	148	19	19	1.69	23.3	61	61	3.37	21.6	87	87	4.41	20.6	1.1	YES
WR-01	24	24" CSP	Warren Dr. @ 92 Ave	50 (c)	48	30	22	3.58	46.4	91	32	8.75	41.3	129	35	11.16	38.8	5.6	NO
DK-02	30	30" CSP	96 ST NW	7	120	4	4	0.58	6.4	13	13	1.8	5.2	18	17	2.28	4.7	0.9	YES
DK-03	72	6' X 6' Box*	Harborview Dr.	20 (c)	40	54	54	2.26	17.7	155	153	4.74	15.3	221	216	6.03	14.0	1.0	YES
DK-04**	36	36 " CSP*	Harborview Dr.	20 (c)	30	54	50	5.73	14.3	153	100	14	6.0	216	119	18.76	1.2	6.3	NO
DK-05&06**	36	36 " CSP*	Harborview Dr.	10 (c)	30	38	24	4.24	5.8	83	38	6.85	3.2	108	44	8.27	1.7	2.8	NO
MC-12	30	30" CSP	Bujacich Rd.	30 (c)	308	5	4	0.43	29.6	18	10	1.12	28.88	27	14	1.59	28.41	0.64	YES
MC-04&5	24	24" & 18" CMP	53rd Ave. NW	6 (c)	170	11	10	1.07	4.9	37	25	2.78	3.22	54	33	3.76	2.24	1.88	NO
MC-04&5	18	24" & 18" CMP	53rd Ave. NW	6 (c)	170	11	10	1.07	4.9	37	25	2.78	3.22	54	33	3.76	2.24	2.51	NO
MC-06&7	36	2- 36" CSP	Burnham Dr.	15 (c)	145	16	16	0.79	14.2	41	39	1.94	13.06	52	49	2.26	12.74	0.75	YES
MC-WDOT1	30	30" CSP	SR 16	20 (b)	160	12	12	1.45	18.6	32	31	3.25	16.75	43	43	4.28	15.72	1.71	NO
MC-WDOT2	54	54" CSP	SR16	25 (b)	125	27	27	2.03	23.0	70	70	3.4	21.6	92	92	4.2	20.8	0.93	YES
MC-WDOT3	54	54" CMP	SR16	25 (b)	118	34	34	2.25	22.8	91	90	4.15	20.85	121	121	5.2	19.8	1.16	YES
MC-60CSP	60	60" CSP	SR16	25 (b)	116	34	34	2.11	22.9	90	90	3.86	21.14	121	121	4.68	20.32	0.94	YES
MC-24	24	24" CMP	SR16	25 (b)	104	14	14	2.09	22.9	39	30	7.65	17.35	54	36	11.6	13.4	5.80	NO
MC-60CMP	60	60" CMP	SR16	22 (b)	102	69	69	3.56	18.4	199	193	8.15	13.85	269	245	11.46	10.54	2.29	NO
MC-01 &2	36	2- 36" CSP	Sehmel Dr./Burnham	6	94	69	68	2.82	3.2	193	190	6.28	**-0.28**	245	244	6.61	**-0.61**	2.20	NO
MC-03	60	60" CSP	Woodhill Dr.	25 (c)	82	69	69	3.22	21.8	195	191	6.7	18.3	250	250	8.63	16.37	1.73	NO

Notes:

(a) Distance estimated from 2-ft. topographic maps

(b) Distance estimated from Washington DOT "As Constructed" Plans for Swede Hill Interchange

(c) Distance estimated in field

(d) All headwater depths computed assuming inlet control except at CR-09 as noted in (e).

(e) Culvert CR-09 is believed to be half full for all initial conditions. Peak discharges and elevations are for d/s channel cross section - See project notes.

(f) Pierce County design standard states that H/D not to exceed 1.5 for culverts over natural streams, where H is u/s headwater elevation relative to invert and D is the culvert diameter (or depth, if rectangular).

** Culvert inlet and outlet size inconsistent*
*** Outlet subject to tidal influence*

U/S = Upstream
D/S = Downstream

Culvert ID	Culvert Type	100-Yr Peak Flow (cfs)	Estimated Base Flow* (cfs)	Ratio of Base Flow Vs. 100-Yr Peak Flow
Artondale Basin				
AD-01	12" CSP	72	1.2	1.67%
AD-02	54" CMP	249	1.2	0.48%
AD-04	6'X6' Box	232	1.2	0.52%
Crescent Basin				
CR-01*	6'X6' Box	584	4.5	0.77%
CR-02	5'x12' Box	453	4.5	0.99%
CR-05	4'X12' Box	243	4.5	1.85%
CR-09	6' CMP	237	4.5	1.90%
McCormick Creek Basin				
MC-12	30" CSP	29	0.8	2.76%
MC-04&5	4" & 18" CM	63	0.8	1.27%
MC-04&5	4" & 18" CM	63	0.8	1.27%
MC-06&7	2- 36" CSP	59	0.8	1.36%
MC-WDOT	30" CSP	45	0.8	1.78%
MC-WDOT	54" CSP	100	0.8	0.80%
MC-WDOT	54" CMP	154	0.8	0.52%
MC-60CSP	60" CSP	152	0.8	0.53%
MC-24	24" CMP	57	0.8	1.40%
MC-60CMP	60" CMP	313	0.8	0.26%
MC-01 &2	2- 36" CSP	283	0.8	0.28%
MC-03	60" CSP	288	0.8	0.28%

Note: Base flow rate estimated based on the average of minimum discharges measured for months from June to September.

C:\DOWNLOADS\barbaraann\[Appendix E - base flow comparison.xls]Table 1