Chapter 4

Use of Continuous Simulation for Evaluation of Stormwater Management Practices to Maintain Base Flow and Control Erosion

Raymond T. Guthier, Ronald B. Scheckenberger and William R. Blackport

Maintaining the natural distribution of flow within watercourse ecosystems is desirable for the support of downstream fisheries, and preservation of natural channel forming processes. Increases in impervious land coverage and efficient conveyance systems, associated with urban development, can lead to a decrease in infiltration and associated decrease in base flow to downstream watercourses, along with localized temporal and absolute increases in high flows, which ultimately impact on channel form and associated habitat.

4.1 Background

The Regional Municipality of Hamilton-Wentworth and the City of Hamilton have identified approximately 46 ha of land proposed for prestige industrial development within a headwater area of the Red Hill Creek in southern Ontario (see Figure 4.1). The total contributing subwatershed consists of approximately 133 ha with two tributary watercourses traversing the development area. The present land use within the area is primarily agricultural with a single commercial user and a number of single family residences.

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In order to address these concerns the Regional Municipality of Hamilton-Wentworth (RMHW) and City of Hamilton initiated a comprehensive environmental assessment. The potential impacts of developing the 46 ha industrial land use which were considered in the assessment included: surface water quality, temperature, erosion and baseflow, as well as the effects that these potential impacts may have on the physical and social environment (i.e. fisheries and terrestrial resources, stream morphology, and socio-economic effects). The assessment was undertaken in accordance with requirements of the Ontario Municipal Engineers Association Class Environmental Assessment process (MEA, 1993).

It should be recognized that a dry stormwater management flood control facility is currently under construction approximately 1 km. downstream of the development area. Additionally, the lands located between the development area and the water quantity facility are in public ownership and are used as passive open space, and they are not considered to be subject to damage during flood events. Therefore water quantity control for flood protection has not been required as a key objective in developing a stormwater management strategy, beyond the control of localized flood and erosion impacts within, and immediately downstream of, the development area.

4.2 Baseline Conditions

4.2.1 Soils

The watershed is characterized by rolling topography with average slopes generally ranging from 1.5 to 4.0%. The soils are classified as a mixture of clay and silt loams with drainage characteristics ranging from hydrologic classes BC to D. Most soil types in the western portion of the watershed exhibit class C and D characteristics, with class BC soil types located in the eastern portion of the study area. Notwithstanding, field observations indicate that the infiltration capacity of the soils would likely exceed the infiltration predicted based on the soil class alone.

4.2.2 Fisheries

An assessment of the fish habitat downstream of the Development Area indicates that the stream flow exhibits high temperatures in the summer months and fish species diversity is very limited. Significantly cooler temperatures were however noted at the base of pools, indicating the contribution of base flow to the stream.

4.2.3 Stream Morphology

Watercourse reaches immediately downstream of the developing area are relatively steep and feature cobble substrate with local outcrops of bedrock. Further downstream the watercourse substrate becomes finer, channel slopes are reduced, and critical velocities are lower. Therefore, the downstream sections of the watercourse are considered to be more vulnerable to erosion due to changes in peak flow rates and duration of flow, than the watercourses immediately downstream of the study area.

4.2.4 Geology

The overburden throughout the study area generally consists of a clayey silt or clay till, likely associated with the Niagara Falls Moraine. The overburden thickness varies from approximately 10 m in the southern portion of the study area to bedrock outcropping particularly along portions of Red Hill Creek and its tributaries. Throughout most of the study area the overburden is less than 5 m thick and is commonly oxidized and fractured. The existence of oxidation and fractures gives rise to a more significant potential for movement of water within the overburden as compared to a massive, un-oxidized clay till.

The upper 7 to 10 m of the underlying bedrock consists of a grey brown to dark brown petroliferous dolostone. The upper bedrock appears to be highly fractured and contains numerous vuggy zones. The fractured vuggy nature of the dolostone allows for a significant pathway for groundwater movement. The bedrock topography tends to follow the surficial topography in the study area.

4.2.5 Hydrogeology

Water well records and geotechnical borehole logs indicate that the water table in the shallow bedrock is generally within 5 m below the top of the bedrock. This general water table trend, combined with the oxidized and fractured nature of the overburden, the highly fractured nature of the shallow (upper 10 m) bedrock, and the headwater nature of the study area, supports the following groundwater flow scenario.

Recharge occurs throughout most of the study area. The infiltration into the fractured, thin overburden will be higher than a more massive, thicker overburden. The higher rate of infiltration was confirmed by an observation of a distinct lack of ponded water within the watershed during an early winter snowmelt. The infiltrating water moves vertically to the bedrock and then moves, to a greater degree, horizontally within the shallow bedrock, following the bedrock topography. The horizontal groundwater flow, as it follows the bedrock topography, is directed to an area along Red Hill Creek channel just downstream

of the developing area where it discharges through a relatively small area of highly fractured karstic bedrock (see Figure 4.1).

A groundwater balance for the area indicates that the infiltration rate throughout the subwatershed may be of the order of 150 to 200 mm/year, which is generally consistent with fractured clay silt tills

The fractured nature of the flow system as presented typically allows for minimal attenuation of contaminated waters entering the subsurface.

4.3 Alternative Stormwater Management Strategies

Based on consideration of baseline environmental conditions, various stormwater management techniques were developed for consideration and evaluation in order to determine the optimum plan for mitigating the impacts due to development. A watershed model was used to evaluate the effects of various stormwater management strategies on baseflow, peak flow and runoff volume. Stormwater management strategies which were considered included:

1. *Future Uncontrolled Development* This alternative involved a traditional type of development (i.e. urban form) with no stormwater management controls. It would not address potential impacts on water quality, erosion or fisheries habitat as required by Provincial and Federal policy; therefore it was considered unacceptable, however it continues to serve as a measure of maximum impact for comparative purposes.
2. *Source Controls* This alternative featured the use of mitigative measures within developing areas on a lot-by-lot basis and included techniques such as: on-site infiltration, reduced lot grading, roof leader discharge to grassed areas, and rural road standards (i.e. roadside ditches).
3. *End-of-Pipe Facility* The end-of-pipe alternative was based on traditional development drainage utilizing an urban road cross section (i.e. curb & gutter) and collection of runoff by storm sewers. Stormwater would then be conveyed to a stormwater management facility (wet pond or constructed wetland) for water quality treatment and additional storage to prevent potential increased incidence and duration of erosive flows.

Two general end-of-pipe alternatives, plus a number of variations were considered and evaluated. The primary difference between these alternatives relates to the conveyance of external drainage through the site:

- 1: no mixing of external and internal stormwater runoff; or
- 2: external and internal stormwater runoff mixes and is conveyed to an end-of-pipe facility.

A potential location for such a stormwater management facility was previously identified (Dames and Moore, 1993) in a former quarry immediately north (downstream) of the development area.

4. *Combination of End-of-Pipe and Source Control Techniques*

This alternative consisted of the use of source controls (i.e. rear yard ponding areas/soakaway pits, reduced lot grading), to promote infiltration from roof tops and landscaped areas in conjunction with an end-of-pipe facility to treat runoff from roads and parking areas, which would be constructed to a full urban standard including storm sewer conveyance.

This option would provide the benefit of promoting the infiltration of “clean” water from rooftop areas, while providing treatment of parking lot, and roadway runoff, typically much higher in suspended sediments, nutrients and heavy metals than roof-top drainage. The combination alternative also included provisions for open watercourse systems to convey external runoff through the developing area without mixing with urban runoff from parking areas and roads.

Based on the proposed land use within the developing area, the use of water quality inlets to pretreat stormwater and provide spill protection for runoff from roads and parking areas, was also considered.

4.4 HSP-F Analysis Approach

In order to determine existing and future land use flows to assess the impact of development on the Red Hill Creek tributary watersheds, a continuous hydrologic simulation technique to determine frequency of flows for system evaluation and design was recommended. Continuous modeling uses historical rainfall data as input to generate a time-series of modeled peak flows. These time-series (i.e. at various key points in the watershed) are then further analyzed for recurrence, based on frequency analysis, thereby associating a peak flow with frequency.

The use of a continuous simulation model offers a significant benefit in terms of allowing evaluation of various water quality and quantity management strategies through a full range of flow conditions including baseflow conditions. This provides a more realistic evaluation of the overall effectiveness of management strategies in replicating the natural hydrologic characteristics of the watercourse.

The Hydrologic Simulation Program - Fortran (HSP-F), developed by Hydrocomp Inc. 1980, for the US-EPA (Bicknell et al. 1993), was selected as a Ministry of Natural Resources - approved continuous simulation technique for use in this study.

4.4.1 Calibration/Verification

The HSP-F model dataset is based on the original calibrated model of the Red Hill Creek which was developed for the *Mountain East-West and North-South Transportation Corridor Study* (Philips Planning and Engineering, 1989) as refined through subsequent design studies (Philips Planning and Engineering, 1993; Bishop and Scheckenberger, 1995). The original model was calibrated to storm flows at a gauge station located at Queenston Road (see Figure 4.1). A portion of the original model was modified and discretized to a greater level of detail as required for the study purposes. The base model was calibrated and validated based on stream flow measurements taken as part of this study. In addition to precipitation and evaporation, snowmelt processes were also simulated in the updated HSP-F model in order to model the performance of stormwater management techniques for the full year as well as assisting in determining water levels within the stormwater management facility.

4.4.2 Low Flow Calibration

Low flow measurements were undertaken on June 1, 1995. The resulting baseflow/interflow from the general study area was observed to be approximately 11 l/s (0.011 m³/s) at a location immediately downstream of the development area. The measured flow was determined to occur on the receding limb of the hydrograph caused by a rainfall event of 18 mm on May 28, 1995. The original base model parameterization computed a base flow of 70 l/s for the same event.

An analysis of interflow and soil storage characteristics was undertaken to determine the sensitivity of these various parameters. Hydrogeological investigations which were undertaken to assess interflow conditions in the study area demonstrated that the clay soils exhibit numerous fissures. This attribute, in conjunction with the shallow overburden and fractured bedrock conditions results in rapid infiltration and interflow similar to that exhibited by a sand or gravel soil. On this basis, it is expected that interflow would also recede quickly, in approximately 6 hours following a storm event.

Based on these observations the following parameter changes were incorporated into the model:

1. Mannings "n" (NSUR) for pervious land segment was reduced. This parameter alteration would increase computed peak flows during runoff events.
2. The interflow recession parameter (IRC) which reflects the slope of the interflow recession limit was reduced to 0.25 from 0.5 (base model), to reflect rapid conveyance of interflow to the receiving watercourse.

3. Lower and upper zone nominal soil storage (LZSN and UZSN) values were increased to 150 mm and 12 mm, respectively, from base model assumptions of 10 mm and 1 mm.

The use of these values is consistent with the values obtained using the relation (Crawford and Linsley, 1966):

$$\text{LZSN} = 101.6 + 0.125 (\text{annual precipitation})$$

Note: Annual precipitation was assumed to be 900 mm/yr, and the relation $\text{UZSN} \approx 0.08 \text{ LZSN}$ (Crawford and Linsley, 1966) was utilized to estimate these parameters

The foregoing modifications tended to reduce the computed baseflow and peak flows resulting in a good correlation to baseflow/interflow observed on June 1, 1995; (0.015 m³/s computed, 0.011 m³/s observed) while maintaining a good correlation to peak flow values under the July and October 1962 storm events as compared to the stormflow calibrated base model (see Table 4.1).

Table 4.1 Peak flow (m³/s).

Model	March 12, 1962	July 26, 1962	October 4, 1962
Red Hill Creek 1990 Model (Macro)	0.3	12.0	15.0
Rymal 1996 (Micro)	0.1	11.0	13.7

The July 26, and October 4, 1962 flows generally compare well (within 10%) to the original model results. The March 12, 1962 storm event was observed to exhibit less correlation to the base model. However, peak flow rates for this event are closer to baseflow magnitude and therefore subject to greater variation due to model parameter changes.

4.4.3 Parameterization of Alternatives

The HSP-F datasets developed for existing site conditions were revised to reflect future land use in order to assess the performance of various stormwater management strategies.

4.4.4 Source Controls

The use of source controls primarily relies on providing measures within the context of site development to promote infiltration, thereby reducing runoff

volumes as well as reducing or eliminating direct connections between the impervious surfaces and the storm drainage system.

The following source controls were considered for use in the study area:

1. discharge of roof leaders to pervious areas,
2. reduction in minimum lot grades (slopes) from current municipal standards,
3. rear yard ponding of roof discharge (roof discharge only), and
4. rural cross-sections.

The simulation of these techniques was incorporated into the model through the variation of the following input parameters:

1. Increasing the value of the input upper soil zone nominal storage (USZN) from 12 mm to 15 mm, to reflect greater opportunity for infiltration provided by reduced lot grading.
2. Reduced input impervious area contribution to reflect discharge of roofs and road surfaces to pervious areas. Input impervious coverage was reduced to existing land use levels (i.e. 1% directly connected impervious area). However, input pervious infiltration capacity was also reduced from 2.5 mm/hr to 0.75 mm/hr, in accordance with the amount of input impervious coverage, to reflect the lower opportunity for infiltration. The use of a rural road cross-section was also incorporated into the model through this reduction of input impervious coverage.
3. The simulation of rear yard ponding areas/soakaway pits was incorporated using a single reach reservoir in a lumped approach to simulate storage of runoff from the development area. The use of this technique was considered appropriate in this case for the following reasons:
 - baseflow/interflow discharge to the surface water system occurs less than 500 m. downstream of the development area;
 - groundwater flow is primarily horizontal through the upper bedrock layer, parallel to the surface flow system with less vertical conveyance to the deep groundwater system; and
 - interflow response of the subwatershed is rapid, of the order of 6 to 8 hours.

The foregoing characteristics suggest that attenuation and routing characteristics of the reservoir algorithm adequately represent the relatively rapid and shallow groundwater flow regime.

The volume of storage was based on the 20 mm criteria over the roof top area as recommended by the Ontario Ministry of Environment and Energy (MOEE). Outflow from the reservoir was based on a conservative infiltration estimate of 5 mm/hr.

4.4.5 End-of-Pipe Facility

The HSP-F model was revised to incorporate a “wet pond” facility within the vacant quarry located north of Rymal Road (immediately downstream of the development area).

The quarry area was modeled to provide a minimum of approximately 15 mm/ha of storage based on the upstream drainage area, an impervious ratio of 85% and Type 2 habitat as recommended by the Ontario Ministry of Environment and Energy (Marshall Macklin Monaghan, 1994). Given the 46 ha development area, the minimum required storage was calculated to be approximately 6,900 m³. Of the 6,900 m³ of total storage, a minimum of approximately 1,840 m³ would comprise extended detention storage while the remaining 5,060 m³ would be retained as permanent pool storage. In addition to this base scenario, various levels of extended detention storage ranging from 4 mm/ha to 37.5 mm/ha were also simulated and assessed.

Outflow from the quarry was modeled as a multiple objective pond outlet which included a reverse slope primary discharge outlet with an overflow weir and spillway to accommodate major storm events.

4.4.6 Combination of Source Control and End-of-Pipe facility

A third method of providing water quality and erosion protection was modeled by combining infiltration techniques employed with the source control alternative and an end-of-pipe facility which would collect and treat runoff from roads and parking areas. This management technique was incorporated into the model by routing the impervious land segment within the development area, through an end-of-pipe facility and directing runoff from the pervious land segments to the rear yard ponding areas/soakaway pits.

The impervious fraction which is directed to the water quality facility was approximated as 40% of the total development area.

The stormwater management facility was modeled based on 2,026 m³ of permanent pool storage, corresponding to 110 m³/ha as recommended for Type 2 habitat and 85% impervious coverage (Ontario Ministry Environment and Energy, 1994). The facility also includes an extended detention component of 4,534 m³ or 250 m³/ha. The drawdown period from the facility was modeled to occur over approximately three days to provide baseflow augmentation benefits.

4.4.7 External Drainage Consideration

The modeling was carried out for two primary methods of external drainage conveyance:

- external flows conveyed separately to their respective outlets (without mixing with urban runoff), and

- external flows mixed with internal drainage and conveyed to an end-of-pipe facility.

The use of an end-of-pipe facility may involve conveyance of external drainage areas through the facility or provisions to convey external runoff separately without mixing to the downstream watercourse.

The provision of a separate conveyance system to convey external flows to their natural outlet was considered to be preferable for the following reasons:

1. mixing of clean external runoff which requires treatment is avoided thus improving the efficiency of treatment;
2. the conveyance of external flow to its natural outlet will tend to preserve the natural function and habitat features of the receiving watercourse reaches; and
3. the overall storage volume of the end-of-pipe facility and conveyance system is reduced.

4.5 Results

4.5.1 Flow Distribution Analysis

The following water quality management strategies were analyzed with respect to potential impacts on base flows within the downstream watercourse:

1. do-nothing - (no development),
2. future uncontrolled development (no mitigation),
3. source controls only,
4. end-of-pipe facility only, and
5. combination of end-of-pipe and source control techniques.

The *do-nothing* alternative, which reflects present conditions within the development area, was used as a benchmark standard by which to compare the various alternatives. The *future uncontrolled* development represents a *worst-case* scenario to assist in the evaluation of the management alternatives.

A number of different *end-of-pipe facility* options were analyzed to determine the relative impacts of extended detention volume and drawdown period on baseflow rates and off-site erosion potential.

Table 4.2a provides a comparison of the various management alternatives in terms of replicating the natural flow class distribution for a typical year (total annual rainfall), while Table 4.2b provides a description of the various stormwater management alternatives. The results are provided for the outlet of the developing area (Node 90.770) which is located downstream of the end-of-pipe facility and includes all drainage from the development area and external drainage which passes through the development area. The results illustrate the impacts associated with uncontrolled development as well as the computed levels of mitigation achieved through the various techniques.

Table 4.2a Flow distribution (percent of annual) for stormwater management alternatives.
Node 90.770 (outlet of development area).

Flow Class (l/s)	Existing	Future No Control	Source Controls	Combination Source Control and End-of-Pipe	End of Pipe				
					19 mm /3 day	19 mm /7 day	Mix	4 mm /1.5 day	37.5 mm /1 day
0.0-0.5	5.98	6.53	2.76	2.24	5.84	3.87	3.32	5.12	4.78
0.5-1.0	3.94	1.58	2.98	0.58	1.45	0.84	0.21	1.07	1.17
1.0-5.0	37.12	48.51	52.88	43.40	46.77	39.12	31.87	42.47	42.93
5-10	25.13	21.96	24.80	23.49	18.78	12.79	2.76	18.68	14.23
10-50	25.90	16.40	14.57	27.62	21.52	40.76	59.94	28.94	33.14
50-100	0.94	2.29	1.10	1.61	4.13	1.35	0.86	1.54	2.40
100-300	0.80	1.95	0.55	0.76	1.14	0.88	0.61	1.54	1.02
300-500	0.06	0.27	0.12	0.11	0.12	0.13	0.13	0.25	0.13
500-1000	0.06	0.31	0.12	0.10	0.13	0.13	0.15	0.24	0.13
1000-1500	0.04	0.11	0.05	0.04	0.05	0.05	0.06	0.07	0.05
1500-3000	0.03	0.07	0.05	0.05	0.06	0.07	0.07	0.06	0.02
over 3000	0.01	0.02	0.01	0.01	0.02	0.02	0.02	0.02	0.02

For most flow classes, the use of a combination of *source controls* and an *end-of-pipe* water quality facility best replicated the natural distribution of flow events. However, various end-of-pipe facilities provided varying computed levels of baseflow enhancement through increased computed duration of higher flow class occurrences.

The results of the baseflow analysis indicated that future uncontrolled development would reduce computed baseflow occurrences in the 10 to 50 l/s flow class by approximately 37% and by 15% in the 5 to 10 l/s flow class at this node. A corresponding increase of 31% is noted in the lower flow class range of 1 to 5 l/s.

End-of-pipe water quality facilities provided potential to enhance computed low flow within the 10 to 50 l/s flow class as noted by a 130% increase for an end-of-pipe facility providing conveyance of internal and external drainage to the facility. However, it is noted that computed occurrences of flows within the 5 to 10 l/s and 1 to 5 l/s are reduced by 89% and 14% respectively. This suggests that the effectiveness of end-of-pipe facilities in augmenting computed low flows was largely dependent on the input outflow release rate.

A combination of source control measures such as an end-of-pipe facility to treat road and parking area drainage is noted to be most effective in maintaining the existing baseflow conditions as evidenced by a 17% increase in the computed 1 to 5 l/s flow class, a 6.5% decrease in the 5 to 10 l/s flow class, and a 6.6% increase in the 10 to 50 l/s flow class.

4.5.2 Wet/Dry Year Flow Distribution Analysis

A further flow comparison was undertaken for various land use and management conditions including: existing conditions, future uncontrolled development and the combination alternative for developed land use under representative dry and wet years to verify the effectiveness of the combination of source control and end-of-pipe treatment.

Representative wet and dry years, determined on the basis of high and low annual total rainfall volumes, are represented by the years 1992 and 1963 respectively.

Tables 4.3 and 4.4 provide a summary of flow distribution and total volume of flow at the outlet of development area for representative wet and dry year simulations.

Based on the wet/dry year analysis, the combination alternative appears to be very effective in augmenting computed baseflow during a dry year as evidenced by a 865% increase in the 10 to 50 l/s flow class. During wet/dry years the combination alternative reduces the computed incidence of extreme low flow (0 to 1 l/s) by 23% and 10%, for wet and dry years respectively. Future uncontrolled runoff by contrast, would increase the computed incidence of extreme low flow by 45% and 18% for respective wet and dry years.

Table 4.2b Description of water quality management alternatives.

Alternative	Land Use	Operational Characteristics		
		Storage	Detention Time (Day)	Area Served
Existing	Existing conditions, no change in land use	N/A	N/A	46 ha
Future No Control	Full urban development to 80% impervious coverage with no SWM practices	N/A	N/A	46 ha
Source	Development to 70% impervious coverage utilize on-site infiltration and swale drainage system	On-site storage equal to 200 m ³ /ha (200 mm from rooftop areas only)	1	On-site measures, service 60% of 45 ha development area -Rooftops
Combination	Development to 70% impervious coverage with on-site infiltration of roof drainage and end-of-pipe treatment of parking area and road runoff	On-site storage equal to 200 m ³ /ha (20 mm) from rooftop areas only End-of-Pipe facility Total Storage 360 m ³ /ha Permanent Pool - 110 m ³ /ha Extended detention storage - 250 m ³ /ha (19 mm)	1 3	On-site measures, service 60% of 45 ha development area -Rooftops End-of-Pipe facility services 40% of 46 ha development area - Road and Parking areas
End-of-Pipe 19 mm /3 day	Development to 80% impervious coverage with end-of-pipe facility	Total storage - 300 m ³ /ha Permanent pool - 110 m ³ /ha Extended detention storage - 190 m ³ /ha (19 mm)	3	Services development area only (46 ha)
End-of-Pipe 19 mm /7 day	Development to 80% impervious coverage with end-of-pipe facility	Total storage - 300 m ³ /ha Permanent pool - 110 m ³ /ha Extended detention storage - 190 m ³ /ha (19 mm)	7	Services development area only (46 ha)

Table 4.2b continued Description of water quality management alternatives.

Alternative	Land Use	Operational Characteristics		
		Storage	Detention Time (Day)	Area Served
End-of-Pipe Mix	Development to 80% impervious coverage with study area	Total storage - 104 m ³ /ha Permanent pool - 110 m ³ /ha Extended detention storage - 190 m ³ /ha (19 mm)	6	Development area and external drainage areas are conveyed to end-of-pipe facility
End-of-Pipe 4 mm/ 1.5 day	Development to 80% impervious coverage with end-of-pipe facility	Total storage - 150 m ³ /ha Permanent pool - 110 m ³ /ha Extended detention storage - 40 m ³ /ha (4 mm)	1.5	Services development area only (46 ha)
End-of-Pipe 37.5 mm /1 day	Development to 80% impervious coverage with end-of-pipe facility	Total storage - 485 m ³ /ha Permanent pool - 110 m ³ /ha Extended detention storage - 375 m ³ /ha (37.5 mm)	1	Services development area only (46 ha)

During wet year conditions, the combination alternative is also noted to be effective in maintaining the computed natural flow distribution throughout the range of flow classes. Future uncontrolled development would result in increased computed incidence of both extreme high and extreme low flow occurrences.

The volumetric comparison also indicates that the combination alternative is effective in augmenting computed baseflow/interflow at non-erosive flow rates (see Table 4.5), while mitigating erosive flow durations to approximately existing undeveloped land use conditions.

4.5.3 Source Control Infiltration Assessment

The use of on-site source control measures was assessed by relating approximate annual recharge to annual total rainfall. The assessment was based on a duration analysis of the computed outflow rate (infiltration) from the modeled source control measure in order to calculate a total volume of the infiltration attained throughout the year through the on-site source control

Table 4.3 Wet year analysis - Node 90.770 (outlet of development area).

Flow Class (l/s)	Wet Year - 1992 - Percent of Year in Flow Class (%)			Wet Year - 1992 - Total Volume in flow class (m ³)		
	Existing	Future	Combination	Existing	Future	Combination
0 - 1.0	12.04	17.52	9.30	1 898	2 762	1 466
1.0 - 5.0	16.57	19.30	16.97	15 676	18 259	16 054
5 - 10	25.90	26.57	23.43	61 258	62 843	55 416
10 - 50	43.02	32.83	45.69	407 003	310 598	432 263
50 - 1 00	1.59	2.91	3.55	37 606	68 827	83 964
100 - 500	0.83	3.24	0.95	78 524	306 529	89 877
500 - 1000	0.02	0.34	0.04	4 730	80 416	9 460
1000 - 1500	0.01	0.07	0.02	3 942	27 594	7 884
1500 - 3000	0.02	0.06	0.02	14 191	42 573	14 191
3000+	0.02	0.05	0.02	18 921	47 304	18 921
Total Flow Volume below Erosive Threshold (500 l/s)				601 968	769 820	679 043
Total Flow Volume above Erosive Threshold (500 l/s)				41 785	197 887	50 456

Table 4.4 Dry year analysis - Node 90.770 (outlet of development area).

Flow Class (l/s)	Dry Year - 1963 - Percent of Year in Flow Class (%)			Dry Year - 1963 - Total Volume in Flow Class (m³)		
	Existing	Future	Combination	Existing	Future	Combination
0 - 1.0	44.17	52.27	39.90	6 965	8 241	6 291
1.0 - 5.0	36.84	41.75	37.61	34 854	39 499	35 582
5 - 10	17.82	2.37	11.40	42 148	5 605	26 963
10 - 50	1.13	1.62	10.91	10 690	15 326	103 217
50 - 100	0.02	0.64	0.07	473	15 137	1 656
100 - 500	0.01	1.09	0.04	946	103 122	3 784
500 - 1000	0.01	0.15	0.01	2 365	35 478	2 365
1000 - 1500	0.00	0.03	0.00	0	11 826	0
1500 - 3000	0.00	0.05	0.00	0	35 478	0
3000+	0.00	0.01	0.00	0	9 460	0
Total Flow Volume below Erosive Threshold (500 l/s)				96 076	186 930	177 493
Total Flow Volume above Erosive Threshold (500 l/s)				2 365	56 764	2 365

Table 4.5 Erosive threshold flows.

HSP-F Node	Lower (m ³ /s)	Upper (m ³ /s)
90.770	0.5	-
30.031	0.3	8.0
80.090	1.5	16.9

facilities. The analysis was based on results of three typical years 1962-average year, 1992-wet and, 1963 dry year according to total annual rainfall volume. The following analysis was based on assessment of the development area only:

Total rainfall depth (average for three years)	= 771 mm
Total rainfall volume (46 ha development area)	= 354 859 m ³
Average natural recharge (150-200 mm)	= 175 mm
Average total recharge volume	= 80500 m ³
Enhanced Volume of recharge through source control measure (HSP-F simulation)	= 43343 m ³
Average Natural Volume of recharge - additional pervious areas (20% of 46 ha)	= 16100 m ³
Average recharge volume (source controls and pervious areas)	= 59 443 m ³
Average recharge (source controls and pervious areas)	= 129 mm

An indication of the effectiveness of source controls in mitigating computed baseflow reductions due to development in the context of the greater watershed area is provided in Table 4.6.

Table 4.6 suggests that source control measures and landscaped areas on development lots would provide 74% of the average naturally occurring infiltration within the developing area. Accordingly, source control measures alone would contribute approximately 57% of the original natural infiltration function within the developing area, providing an effective means of maintaining the recharge function.

The results were considered to be conservative based on the following considerations:

1. Evaporation losses are explicitly included in the reservoir routing function which was been utilized to model the infiltration facilities. Therefore evaporation was subtracted from the infiltrating waters. This was considered to be consistent with the physical characteristics of rear yard ponding areas.
2. The infiltration rate of the soil was conservatively assumed to be approximately 5 mm/hr. It was anticipated that the real soil infiltration rate would be greater than this value providing additional infiltration storage capacity and less frequent occurrence of overflow.

Table 4.6 Infiltration assessment for average annual rainfall (mm) - 1962.

Location	HSP-F Node	Existing Land Use	Future Development No Mitigation	Change in Infiltration over watershed No Mitigation (mm) [%]	Future Land Use with Mitigation	Mitigation of Impacts (% of Objective value)
Developing Area (46 ha)	90.030	175	52	-122 [-70%]	129	74%
Outlet Development Area and External Drainage (133 ha)	90.770	175	133	-42 [-24%]	159	91%
Confluence with West Tributary (403 ha)	30.031	175	161	-14 [-8%]	170	97%

- 3. Additional contributions to baseflow/interflow regime would occur within local swales and natural watercourse systems. The volume of runoff contribution through this process was not included in the assessment.
- 4. Seepage into the fractured limestone within the stormwater management facility was also expected to increase baseflow/interflow to the downstream watercourse system.

The foregoing considerations are difficult to quantitatively assess, however it is anticipated that the additional contributions would provide a significant amount of additional infiltration.

4.5.4 Erosion Analysis

The potential impacts of urban development on erosion potential were assessed on a preliminary basis using a duration analysis technique for a typical year. The erosion analysis was undertaken at three locations corresponding to:

- 1. development area outlet (Node 90.770),
- 2. confluence with Western Tributary, 500 m downstream of development area (Node 30.031), and
- 3. Stonechurch Road, 1000 m downstream of development area (Node 80.090).

The analysis results are based on the number of hours during which stream flows are above the erosion threshold flow at each location. The lower threshold reflects the flow rate resulting in a critical velocity, above which channel erosion would be expected to occur. An upper threshold flow corresponding to the bankfull discharge was determined at which point flood flows would spill on to the flood plain and channel velocities would reach a maximum. (An upper threshold flow and velocity was not selected at Node 90.770 due to the steep channel slope, hence all flows above the critical threshold were assessed in the erosion analysis.)

A critical velocity of 1.0 m/s was selected for Nodes 90.770 and 30.031, while a critical velocity of 0.6 m/s was selected for Node 80.090 based on observed channel sloughing and finer grained soils in this vicinity. Table 4.5 provides a summary of the limiting erosive threshold levels for the three locations.

Table 4.7 provides a summary of the erosion analysis results for two cases:

1. Future land use within the development area only, existing land use applied to external areas.
2. Future land use within the development area, as well as future land use applied to external areas (ultimate development conditions).

The results of the duration analysis indicate that future uncontrolled development would significantly increase the erosive flow duration immediately downstream of the development area. Increases in erosive duration were calculated at 263%, 37% and 9% at Node 90.770, 30.031, and 80.090 respectively.

Generally, use of a combination of source controls and an end-of-pipe water quality facility best mitigates increases in erosion impacts at the development area outlet (Node 90.770). However all methods tested were noted to be effective in mitigation of erosion impacts.

It is noted that the various end-of-pipe facilities are also effective in mitigating increases in erosive flows. However their effectiveness is slightly reduced in comparison to source controls or a combination of source controls and an end-of-pipe facility. Of the end-of-pipe facilities evaluated, a 37.5 mm level of extended detention storage in conjunction with a 24-hour drawdown period was computed to be the most effective in mitigating increased erosive flows with a 78% level of effectiveness in mitigating erosion impacts at Node 90.770.

Results for Node 30.031 and 80.090 illustrate similar findings, however the effectiveness of the various stormwater management strategies is influenced by the larger external drainage area outside the development area.

Table 4.8 illustrates the results of an assessment of the combination alternative on the basis of wet, dry and average years combined.

The results indicate that the combination alternative has a 91% effectiveness in mitigating computed increases in erosive flow occurrences.

Table 4.7 Erosion duration analysis. Number of hours above erosive threshold velocity for management alternatives
- average rainfall year (1962).

Node	Case	Existing	Future No Control	Source Control	Combination Source Control and End-of- Pipe	End-of-Pipe				
						19 mm /3 day	19mm /7 day	Mix	4 mm /1.5 day	37.5 mm /1 day
90.770		12.3	44.7	20.1	17.5	22.8	23.7	26.3	34.2	19.3
30.031	1	90.2	123.3	77.1	85.0	93.0	90.3	90.3	111.0	89.0
	2	90.2	296.1	238.5	244.0	251.4	247.9	252.3	265.4	248.8
80.090	1	56.1	61.0	50.3	51.0	54.5	54.0	54.8	57.5	53.8
	2	56.1	118.3	112.0	111.0	117.4	115.6	114.8	118.3	116.5

mm = mm of storage over developing area

day = facility drawdown period

Table 4.8 Duration analysis for wet (1992), dry (1963), and average (1962) years (based on total annual rainfall); hours above erosive threshold velocity.

Location	HSP-F Node	Existing	Future Uncontrolled	Net Change in Duration (hours) [%]	Future Combination Alternative	Mitigation Effectiveness on Erosion Impacts (%)
Outlet of Development Area	90.770	6.4	37.1	30.6 [478%]	9.1	91

4.6 Preferred Stormwater Management Strategy

Based on the technical evaluation process, the stormwater management alternative which incorporates the use of a combination of source controls, as well as an end-of-pipe facility was recommended. This alternative best maintains the computed hydrogeologic, erosion and water quality regime. In addition, opportunities are provided to enhance terrestrial resources in the area and maintain habitat functions through protection and enhancement of on-site watercourses.

Key components of the preferred solution include:

1. Rear yard ponding areas or soakaway pits to promote infiltration. Each ponding area or soakaway pit should provide storage equivalent to 20 mm of runoff from the contributing roof top area.
2. Provisions for an open naturalized watercourse system through the study area. The watercourse system should be designed to incorporate objectives of natural channel design. The watercourse system should be utilized to convey external drainage through the study area, provide a major system outlet for the developing area, and should discharge to its present outlet location, north of Rymal Road. The provision of open natural watercourses will also serve to preserve flood storage throughout the development area.
3. Roads and parking areas within the study area should drain to an end-of-pipe facility via a storm sewer system and should be constructed using an urban curb and gutter cross section. The storm sewer system should be sized according to the drainage requirements for roads and parking areas only, thereby reducing the overall cost of the storm sewer, and concentrating the capture of contaminated runoff, thereby improving water quality treatment.
4. Supplemental use of water quality inlets (such as manhole separators installed within the storm sewer system) within development sites to provide pre-treatment of runoff and spill protection.

5. An end-of-pipe facility (stormwater management pond). This facility should be located in the former quarry location and should include an open water component as well as a shallow wetland area. The pond should be divided into two storage zones consisting of a sediment forebay and a naturalized wetland/open water area. The water storage sizing has been based on results of this study and the Ministry of Environment and Energy Guidelines. The stormwater management facility should provide water quality treatment, erosion control, low flow augmentation, and passive recharge.

Note: If the natural configuration and elevations within the former quarry are utilized, the permanent pool storage provided would be well in excess of the minimum storage requirements.

6. The end-of-pipe facility should include native wetland plantings such as cattails, and sedges to enhance the natural features of the site. This natural design approach should provide a high aesthetic value in the context of the adjoining environmentally sensitive area (ESA) and watercourse valley owned by the Hamilton Region Conservation Authority.

4.7 Conclusions

The preferred stormwater management strategy includes two primary measures: enhanced infiltration of relatively clean rooftop drainage and treatment of more highly polluted runoff from roads and parking in an end-of-pipe facility.

The preferred stormwater management solution for the 46 hectares of prestige industrial development, was developed using the Municipal Engineers Association (1993) - Class Environmental Assessment process.

The existing hydrogeologic function of a watershed is often difficult and/or resource-intensive to assess directly. The evaluation methodology, as well as the hydrologic modeling techniques used to simulate the linkage between hydrogeologic and surface flow regimes, provided a simple assessment approach for this unique subwatershed location. The study of the various hydrologic and hydrogeologic aspects of the headwater area determined a significant linkage between surface water interflow and the relatively shallow groundwater flow regime. It has been determined that significant contributions to base flow immediately downstream of the study area occur through this linkage.

The HSP-F modeling approach proved to be an effective tool to evaluate various stormwater management practices with respect to their effectiveness in augmenting base flow/interflow linking the hydrologic and hydrogeologic function of the watershed.

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