Iowa Surface Water Quality Standards Implementation

(WLA Procedure)

Iowa Department of Natural Resources
Water Resources Section

November 2010

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I. Introduction

The purpose of this document is to provide the technical methodologies to develop wasteload allocations and water quality based limits to be protective of surface water quality standards as described in IAC 567 Chapter 61 – Water Quality Standards. A Wasteload Allocation (WLA) is the portion of a receiving water's total maximum daily load that is allocated to one of its existing or future point sources of pollution.

The Iowa Department of Natural Resources (IDNR) is responsible for maintaining and enhancing water quality in the state. To that end, IDNR develops WLAs for facilities that discharge treated wastewater (for example, domestic sewage treatment plants and industrial plants) into waters of the state in order to assure that the permitted effluent limits meet applicable state Water Quality Standards. The calculation of a WLA may be based on conservative assumptions to protect the water quality under worst case scenario conditions. Facilities can submit site-specific information on both the receiving waterbody and the discharge characteristics for consideration.

This document is posted on the Iowa Department of Natural Resources - Water Quality Bureau website (http://www.iowadnr.gov/water/standards/ruleref.html).

II. Determining Water Quality Uses and Criteria

Surface water classifications are described in IAC 61.3(1) as two main categories, General Uses and Designated Uses.

General use segments. These are intermittent watercourses and those watercourses which typically flow only for short periods of time following precipitation and whose channels are normally above the water table. These waters do not support a viable aquatic community during low flow and do not maintain pooled conditions during periods of no flow.

The general use segments are to be protected for livestock and wildlife watering, aquatic life, noncontact recreation, crop irrigation, and industrial, agricultural, domestic and other incidental water withdrawal uses.

Designated use segments. These are water bodies which maintain flow throughout the year or contain sufficient pooled areas during intermittent flow periods to maintain a viable aquatic community.

All perennial rivers and streams as identified by the U.S. Geological Survey 1:100,000 DLG Hydrography Data Map (published July 1993) or intermittent streams with perennial pools in Iowa not specifically listed in the surface water classification of 61.3(5) are designated as Class B(WW-1) and A1 waters unless a Use Attainability Analysis (UAA) proves otherwise.

Designated uses of segments may change based on a use attainability analysis consistent with 61.2(5)"e." Designated use changes will be specifically listed in the surface water classification of 61.3(5).

Designated use waters are specified in different use categories in IAC 61.3(1)”b” as *Primary contact recreational use (Class “A1”), Secondary contact recreational use (Class “A2”), Children’s recreational use (Class “A3”), Cold water aquatic life—Type 1 (Class “B(CW1)”), Cold water aquatic life—Type 2 (Class “B(CW2)”), Warm water—Type 1 (Class “B(WW-1)”), Warm water—Type 2 (Class “B(WW-2)”), Warm water—Type 3 (Class “B(WW-3)”), Lakes and wetlands (Class “B(LW)”), Human health (Class “HH”), Drinking water supply (Class “C”).* Designated uses include:

Primary contact recreational use (Class “A1”). Waters in which recreational or other uses may result in prolonged and direct contact with the water, involving considerable risk of ingesting water in quantities sufficient to pose a health hazard. Such activities would include, but not be limited to, swimming, diving, water skiing, and water contact recreational canoeing.

Secondary contact recreational use (Class “A2”). Waters in which recreational or other uses may result in contact with the water that is either incidental or accidental. During the recreational use, the probability of ingesting appreciable quantities of water is minimal. Class A2 uses include fishing, commercial and recreational boating, any limited contact incidental to shoreline activities and activities in which users do not swim or float in the water body while on a boating activity.

Children’s recreational use (Class “A3”). Waters in which recreational uses by children are common. Class A3 waters are water bodies having definite banks and bed with visible evidence of the flow or occurrence of water. This type of use would primarily occur in urban or residential areas.

Cold water aquatic life—Type 1 (Class “B(CW1)”). Waters in which the temperature and flow are suitable for the maintenance of a variety of cold water species, including reproducing and nonreproducing populations of trout (Salmonidae family) and associated aquatic communities.

Cold water aquatic life—Type 2 (Class “B(CW2)”). Waters that include small, channeled streams, headwaters, and spring runs that possess natural cold water attributes of temperature and flow. These waters usually do not support consistent populations of trout (Salmonidae family), but may support associated vertebrate and invertebrate organisms.

Warm water—Type 1 (Class “B(WW-1)”). Waters in which temperature, flow and other habitat characteristics are suitable to maintain warm water game fish populations along with a resident aquatic community that includes a variety of native nongame fish and invertebrate species. These waters generally include border rivers, large interior rivers, and the lower segments of medium-size tributary streams.

Warm water—Type 2 (Class “B(WW-2)”). Waters in which flow or other physical characteristics are capable of supporting a resident aquatic community that includes a variety of native nongame fish and invertebrate species. The flow and other physical characteristics limit the maintenance of warm water game fish populations. These waters generally consist of small perennially flowing streams.

Warm water—Type 3 (Class "B(WW-3)"). Waters in which flow persists during periods when antecedent soil moisture and groundwater discharge levels are adequate; however, aquatic habitat typically consists of nonflowing pools during dry periods of the year. These waters generally include small streams of marginally perennial aquatic habitat status. Such waters support a limited variety of native fish and invertebrate species that are adapted to survive in relatively harsh aquatic conditions.

Lakes and wetlands (Class "B(LW)"). These are artificial and natural impoundments with hydraulic retention times and other physical and chemical characteristics suitable to maintain a balanced community normally associated with lake-like conditions.

Human health (Class "HH"). Waters in which fish are routinely harvested for human consumption or waters both designated as a drinking water supply and in which fish are routinely harvested for human consumption.

Drinking water supply (Class "C"). Waters which are used as a raw water source of potable water supply.

Numerical criteria for different designated uses are included in IAC 61.3(3) *Specific water quality criteria.*

A waterbody may have multiple uses as described above. WLAs for any point source discharges must protect all the uses of the receiving waterbody and the downstream water uses.

III. WLA Procedures

3.1 Discharge Flow Determination

Wasteload allocations are determined for wastewater treatment facilities or other permitted regulated activitiesthat discharge into waterways in order to assure that applicable state Water Quality Standards are met within the watershed basin. For continuous wastewater discharges that need construction permits, wasteload allocation analyses are performed for the projected design Average Dry Weather (ADW) and Average Wet Weather (AWW) wastewater discharge flows entering a receiving stream which is at the design low stream flow regime.

Design flows are obtained from facility plans, engineering reports, or construction permits. ADW and AWW flows for municipal, industrial, and semi-public wastewater treatment facilities need to be approved by the Iowa Department of Natural Resources Wastewater Review engineers to be used in a wasteload allocations for NPDES permit purposes. The definitions for ADW and AWW flows are provided in Section 14.4.5.1 of the Iowa Wastewater Facilities Design Standards.

For controlled discharge lagoons, the definition of the 180-day AWW flow is provided in Section 18C.4.1.1 of the Iowa Wastewater Facilities Design Standards. WLAs for controlled discharge lagoons are calculated using the drawdown rate that is usually ten times the AWW design flow.

Industrial discharges such as cooling water discharges do not usually have wastewater design flows because no wastewater treatment design is needed. For such discharges, maximum monthly average flow and daily maximum flow are usually provided in the NPDES permit application. In order to be consistent with the flows used for municipal discharges, a maximum monthly average discharge flow and a maximum daily discharge flow are used in the WLA calculations for industrial discharges where wastewater design flows are not applicable. This is consistent with the EPA newly released Permit Writers Manual that says "Permitting authority policy or procedures might specify which flow measurement to use as the critical effluent flow value(s) in various water quality-based permitting calculations."

3.2 Stream Design Low Flow Determination

Water quality criteria are designed to ensure protection of designated uses. Numerical criteria consist of three components, magnitude, duration and frequency. For example, aquatic life criteria are usually defined for four-day and one-hour durations for chronic and acute criteria, respectively. The human health criteria for carcinogens are derived assuming lifetime exposure. Thus, water quality criteria should be appropriately considered in developing wasteload allocations, which are then translated into permit requirements to ensure criteria are met. EPA guidance requires criteria to be protective for an average frequency of excursion of every three years. Duration and frequency are defined in the design stream low flow used to develop WLAs. Table 3.1 shows the applicable design low flows for the implementation of different criteria in WLAs.

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Table 3.1. Design Low Stream Flow Regime

Type of Numerical Criteria	Design Low Flow Regime
Aquatic Life Protection (TOXICS)	
Acute	1Q10
Chronic	7Q10
Aquatic Life Protection (AMMONIA – N)	
Acute	1Q10
Chronic	30Q10
Human Health Protection & MCL	
Non-carcinogenic	30Q5
Carcinogenic	Harmonic mean
CBOD	7Q10

1Q10 means 1-day, 10-year low flow,
7Q10 means 7-day, 10-year low flow,
30Q5 means 30-day, 5-year low flow,
30Q10 means 30-day, 10-year low flow,
Harmonic Mean is calculated by dividing the number of daily flows in the database by the sum of the reciprocals of those daily flows.
CBOD = Carbonaceous Biochemical Oxygen Demand.

Wasteload allocation analysis will be performed on the receiving streams designated as Class A, B, and/or C with existing or proposed wastewater discharges and on the tributaries classified as general use that receive wastewater discharges.

The calculation of low flows on ungaged stream reaches are based on data from Plate 3 and 4 of the USGS publication, “Annual and Seasonal Low-Flow Characteristics of Iowa Streams,” March 1979, as well as the critical low flows at a nearby USGS gage. Low flow at gaged stream locations is obtained from the USGS Open-File Report “Statistical Summaries of Selected Iowa Streamflow Data”. IDNR also developed an Excel program that uses the Log Pearson Type III statistical method to estimate monthly critical low flows such as 30Q10, 7Q10 and 1Q10 values at USGS gaged sites. On ungaged sites, the critical low flows can be extrapolated from a gaged site using methods such as the drainage area ratio method and/or Plate 4 of USGS publication depending on the distance between the USGS gage and the unaged site. A GIS tool has been created by IDNR GIS staff to automatically calculate the critical low flows at ungaged sites in streams.

Monthly critical low flows in lieu of annual critical low flows may be used for ammonia nitrogen and temperature WLAs due to the fact that both criteria change from month to month. Monthly critical low flows are used for temperature. For ammonia nitrogen WLAs, the Department will explore the use of monthly critical low flows on a case by case basis and will consider the factors such as:

(1) Is the receiving stream a perennial stream;

(2) Is the receiving stream an effluent dominated stream;

(3) Is there a nearby USGS gage that has an adequate flow record to be used to reasonably estimate the monthly low flows at the discharge location.

In addition to the use of critical low flows, the Department may calculate WLAs using stepwise flow approach if a facility chooses to not discharge below a specified stream flow and only discharges when stream flow is high enough to assimilate the discharge. However, the facility must clearly demonstrate that there is sufficient storage available to operate in this manner and must have an accurate means of determining stream flow at the point of discharge.

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3.3 Ammonia Nitrogen

The aquatic life criteria for ammonia nitrogen is a function of pH (for acute criteria) and temperature (for chronic criteria), as presented from Table 3a to Table 3c of IAC 61.3(3)"b", due to the influence of these parameters on the toxic form of ammonia (unionized). Therefore, it is necessary to establish the representative instream and effluent pH and temperature values before the acute and chronic ammonia criteria can be applied. The ammonia criteria is calculated based on representative monthly pH and/or temperature values. As a result, the ammonia WLAs will also be expressed as monthly values.

Instream Background pH, Temperature and Ammonia Nitrogen:

The waterbody background pH, temperature and ammonia nitrogen levels are calculated using Iowa' STORET monitoring data, USGS monitoring data and the Iowa State University Lake Study (the detail calculations are described in Estimation *of Background Temperature and pH Values for Rivers/Streams in Iowa*). The estimated average pH and temperature values based on the monitoring data are selected. They are shown in Table 3.2 and 3.3. Background median ammonia nitrogen concentrations for warmwaters are shown in Table 3.4.

Table 3.2. Default Background Temperature Values for Different Waterbodies (T°C)

Month	WarmWater	Mississippi River		Missouri River	Cold Water	Lakes
		Zone II	Zone III			

Jan.	0.7	0.5	0.4	1.0	5.5	--
Feb.	0.5	0.7	1.0	1.3	4.4	--
Mar.	2.3	3.1	3.8	4.8	6.4	--
Apr.	9.3	9.9	11.1	10.8	9.5	13.6
May	15.3	16.1	16.7	17.3	13.3	18.5
Jun.	19.5	22.2	22.5	22.7	16.8	23.0
Jul.	23.8	25.1	25.4	26.1	18.1	26.7
Aug.	24.7	24.3	25.3	25.6	17.5	26.3
Sep.	20.8	20.3	21.9	20.9	15.1	21.1
Oct.	14.6	12.5	13.8	14.1	11.0	16.3
Nov.	7.7	5.7	7.1	6.9	7.8	--
Dec.	1.8	1.2	1.2	1.5	5.3	--

Table 3.3. Average Background pH Values (s.u.) for Different Waterbodies

Month	Warm Water	Cold Water	Lakes	
	pH	pH	Lab pH	Field pH
Jan.	7.6	8.1	--	--
Feb.	7.9	8.1	--	--
Mar.	7.9	7.9	--	--
Apr.	8.1	8.1	--	--
May	8.1	8.1	8.0	8.3
Jun.	8.0	7.8	8.0	8.3
Jul.	8.1	8.0	7.6	8.3
Aug.	8.2	8.0	8.1	8.2
Sep.	8.2	8.0	7.2	8.6

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Oct.	8.2	8.1	--	--
Nov.	8.2	8.1	--	--
Dec.	8.1	8.2	--	--

Table 3.4. Default Background Ammonia Nitrogen Concentrations for Warmwaters (mg/L)

Month	WarmWater
Jan.	0.04
Feb.	0.08
Mar.	0.16
Apr.	0.03
May	0.03
Jun.	0.03
Jul.	0.02
Aug.	0.02
Sep.	0.02
Oct.	0.02
Nov.	0.02
Dec.	0.03

Statewide Effluent pH and Temperature:

Statewide effluent pH and temperature values were developed for aerated lagoon, mechanical treatment plants, and industrial discharges.

The statewide effluent pH and temperature values for covered lagoons were developed in 2003 based on the data submitted for several facilities by consultants including facilities from several other states. The site specific effluent pH values for floating covered aerated lagoons were extrapolated from the monitoring data (2 years) for these facilities.

Table 3.5 Statewide Effluent pH and Temperature Values for Different Treatment Plants

Months	Aerated Lagoon		Mechanical Plant		Industrial Discharge		Covered Lagoon	
	pH	Temperature	pH	Temperature	pH	Temperature	pH	Temperature
Jan.	7.5	4.5	7.67	12.4	7.9	17.83	7.5	9.6

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Feb.	8	8.1	7.71	11.3	8.1	17.83	8	10.3
Mar.	8.4	8.7	7.69	13.1	8	27.67	8	11.2
Apr.	8.3	14.6	7.65	16.2	8.2	33.89	8	10.8
May	8.5	18.8	7.67	19.3	8.3	35.89	8	18.3
Jun.	8.5	22.8	7.7	22.1	8.2	38.67	8	18.5
Jul.	8.5	25.3	7.58	24.1	8.2	40.61	8	19.4
Aug.	8.6	25.3	7.63	24.4	8.2	39.61	8	19.2
Sep.	8.6	22.2	7.62	22.8	8.3	34.5	8	19.3
Oct.	8.6	16.6	7.65	20.2	8.2	31.89	8	12.4
Nov.	8.6	12.4	7.69	17.1	8.2	29.39	8	11.7
Dec.	8.4	8.4	7.64	14.1	8.1	24.67	8	10.8

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Ammonia Nitrogen Decay Calculations:

Ammonia nitrogen is non-conservative in the environment and it can be oxidized to nitrite and nitrate. The ammonia decay can be accounted for in a wasteload allocation when the effluent flows through a discharge pipe, storm sewer or general use stream before it enters a designated stream. When site-specific field data are available, the ammonia nitrogen decay in a general use segment of a stream is usually estimated by water quality modeling such as QUALIIK. A simpler approach for estimating ammonia nitrogen decay is to use a default decay rate of 1 mg/L loss per mile in a general use segment of a stream (within a 2 mile distance), which is an estimate based on past modeling data. When distances exceed two miles, a first-order decay equation will be used with decay rates based on the default value used in QUALIIK modeling.

Mixing Zone and Zone of Initial Dilution:

The Mixing Zone (MZ) flow and the Zone of Initial Dilution (ZID) flow for ammonia are a function of the dilution ratio of the receiving stream to the effluent. This dilution ratio is defined in IAC 61.2(4)"e"(1) for a specific discharger as the ratio of the critical stream flow to the effluent design flow. The chronic and acute wasteload allocations for ammonia are calculated based on different design low stream flows, i.e. 30Q10 stream flow for chronic WLAs and 1Q10 stream flow for acute WLAs. The dilution ratios for ammonia are calculated using 30Q10 and the effluent discharge flow and are discussed below.

Dilution Ratios

The flow used in the wasteload allocation calculations for the MZ and ZID vary with the type of dilution ratio. The discharger will be separated into one of three types based on the river and discharge flows:

a. Type 1: The ratio of stream flow to discharge flow is less than or equal to 2:1 -
MZ is 100% of the 30Q10 ZID is 5% of the 1Q10

b. Type 2: The ratio of stream flow to discharge flow is less than or equal to 5:1 and greater than 2:1 –
MZ is 50% of the 30Q10 ZID is 5% of the 1Q10

c. Type 3: The ratio of stream flow to discharge flow is greater than 5:1 –
MZ is 25% of the 30Q10 ZID is 2.5% of the 1Q10

Mixing Zone boundary pH and temperatures values used to calculate the water quality standards to meet the chronic criteria are defaulted to the ambient statewide background values unless site specific ambient background pH and temperature values are available. Zone of Initial Dilution boundary pH and temperatures are calculated based on mass balance equations.

Site specific mixing zone study data, either from field studies or modeling such as the use of the CORMIX model, may be submitted to the Department for consideration in lieu of the above default MZ and ZID values.

Calculation of the Wasteload Allocations:

To meet the acute and chronic aquatic life criteria, the ammonia nitrogen wasteload allocations are calculated based on Equation 3.1:

$$C_rQ_r + C_eQ_e = C_s(Q_r + \text{MZ or ZID } Q_e) \qquad (3.1)$$

where:

C_r = Background concentration, mg/l
$(Q_r + \text{MZ or ZID } Q_e)$ = Stream flow in the MZ or ZID, cfs
Q_e = Effluent flow, cfs
C_s = Applicable water quality standard, mg/l
C_e = WLA concentration, mg/l

This equation is solved for C_e, resulting in wasteload allocations for both the acute and chronic criteria.

Final Ammonia Nitrogen WLA:

The acute WLAs calculated from the above equation will be compared with the allowed ammonia nitrogen concentration in order to meet the DO standard calculated from water quality modeling to arrive at the final acute ammonia nitrogen WLAs. The final acute and chronic WLAs for ammonia nitrogen are then carried forward to the Permit Derivation Procedure section to derive the water quality based limits for ammonia nitrogen.

3.4 Toxics (Metals and Other Parameters)

Instream Background Chemical Concentrations:

Iowa Water Quality Standards have defined numerical criteria for 89 priority pollutants. To properly implement these criteria and calculate WLAs for each wastewater discharge, background concentrations of the pollutants in Iowa surface waters have to be established. Two main sources of monitoring data are available. One is the Iowa STORET data, the other is the USGS water quality monitoring data. A brief description of the data from the two sources is as follows:

STORET network data:

The STORET network has 90 monitoring sites for Iowa interior streams. STORET data has been collected since 1999. Sixty six of the 89 priority pollutants that have numerical criteria are among the parameters that the network monitors.

However, the STORET dataset has a large number of observations that are reported as non-detect (ND). Of the 66 parameters of interests, 48 parameters do not have a single monitoring reading above detection limits; 9 parameters have more than 95% of the readings that are below detection limits; 5 parameters have some but less than 95% of readings below detection limits; only 4 parameters do not have readings below detection limits. A large number of NDs in a dataset creates uncertainty.

USGS monitoring network data:

USGS has been monitoring water quality parameters in Iowa waters. Compared to STORET data, it has more reported values. However, the datasets do not explicitly report detect limits.

Data used for statewide chemical background levels:

Data from both the STORET Network and the USGS monitoring network were used to derive the statewide background chemical concentrations. Monitoring data for each pollutant will be reviewed for their detection frequency and number of records above detection limit readings.

Statistical Analysis Methods

Most statistical methods require a portion of the dataset to be reported values above detection levels. For a dataset that has 100% NDs or a large percentage of NDs, none of the statistical methods would be applicable or would produce satisfactory results. EPA suggests that in cases when the detection frequency is low (e.g., < 4%-5%) and the number of detected observations is low (<4-6 readings), the project team and the decision makers should make a decision on a site-specific basis.

Datasets having more than 95% NDs will not be analyzed by any statistical method. A zero background level will be assigned to these chemicals.

Datasets with more than 5% of the values being detected will be analyzed by the Kaplan-Meier method, one of the nonparametric procedures. The Kaplan-Meier method is widely recommended for analysis of water quality data for the following reasons:

- Most water quality data does not follow a certain distribution
- Kaplan-Meier methods have been proven to produce best estimates of the upper confidence levels
- Kaplan-Meier method produces better results for smaller sample sizes or highly skewed data.

Table 3.6 shows the estimated background chemical concentrations. It is important to note that for metals, unfiltered measurements are used due to the fact that Iowa's metal criteria are expressed as total concentrations. Median values are used since the Kaplan-Meier method produces unbiased statistical estimations for median values.

Table 3.6 lists the estimated statewide background concentrations for different priority pollutants.

Chemical	Median (ug/l)	Chemical	Median (ug/l)
1,1,1-Trichloroethane	0.00	Endosulfan	0.00
1,1,2-Trichloroethane	0.00	Endothall	--
1,1-Dichloroethylene	0.00	Endrin	0.00
1,2,4-Trichlorobenzene	0.00	Ethylbenzene	0.00
1,2-Dichloroethane	0.00	Ethylene dibromide	--
1,2-Dichloropropane	0.00	Fluoride	200
2,3,7,8-TCDD (Dioxin)	--	gamma-Hexachlorocyclohexane (Lindane)	0.000

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2,4,5-TP (Silvex)	--	Glyphosate	0.03
2,4-D	0.000	Heptachlor	0.00
3,3-Dichlorobenzidine	0.00	Heptachlor epoxide	0.00
4,4' DDT	--	Hexachlorobenzene	0.00
Alachlor	0.000	Hexachlorocyclopentadiene	0.00
Aldrin	0.000	Lead	3.000
Aluminum	700	Mercury (II)	0.000
Antimony	0.00	Methoxychlor	0.00
Arsenic (III)	3.000	Nickel	3.000
Asbestos	--	Nitrate as N	1,800
Atrazine	0.098	Nitrate+Nitrite as N	5,300
Barium	100.0	Nitrite as N	20
Benzene	0.000	o-Dichlorobenzene	0.00
Benzo(a)Pyrene	0.00	Oxamyl (Vydate)	0.00
Beryllium	0.00	para-Dichlorobenzene	--
Bromoform	--	Parathion	0.00
Cadmium	0.00	Pentachlorophenol (PCP)	0.00
Carbofuran	0.00	Phenols	0.00
Carbon Tetrachloride	--	Picloram	0.00
Chlordane	0.00	Polychlorinated Biphenyls (PCBs)	--
Chlorobenzene	0.00	Polynuclear Aromatic Hydrocarbons (PAHs)	--
Chlorodibromomethane	0.00	Selenium (VI)	1.000
Chloroform	190	Silver	0.000

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Chloropyrifos	--	Simazine	0.000
Chromium (VI)	0.00	Styrene	0.000
cis-1,2-Dichloroethylene	0.00	Tetrachloroethlyene	0.000
Copper	10.00	Thallium	0.000
Cyanide	0.000	Toluene	0.030
Dalapon	--	Total Residual Chlorine (TRC)	--
Di(2-ethylhexyl)adipate	--	Toxaphene	0.000
Bis(2-ethylhexyl)phthalate	0.00	trans-1,2-Dichloroethylene	0.000
Dibromochloropropane	--	Trichloroethylene (TCE)	0.000
Dichlorobromomethane	0.00	Trihalomethanes (total)	0.000
Dichloromethane	0.00	Vinyl Chloride	0.000
Dieldrin	0.00	Xylenes (Total)	0.000
Dinoseb	0.00	Zinc	0.000
Diquat	--		

Mixing Zone and Zone of Initial Dilution for Toxics:

The regulatory MZ and ZID are included in IAC 61.2(4)"b"(1). The facility can also provide site specific mixing zone data either by field study or using modeling, such as the CORMIX model. Department staff will use the default mixing zone values unless the applicant provides additional data that demonstrates that the characteristics of the outfall or the discharge location do not match the assumptions used in the development of the WLAs. Other models in addition to CORMIX can be used where appropriate or as they become available.

The Mixing Zone Procedure Section presents the basic field data requirements of a MZ study to be provided by an applicant for recalculation of the local MZ. The purpose of the recalculation is to more closely approximate the local MZ using site specific data instead of statewide data. Contact should be made with the Department's Water Resources Section prior to beginning any field study.

Calculation of WLAs for Toxics:

The calculation of toxic WLAs involves the incorporation of the regulatory MZ and ZID for each wastewater discharge, the effluent flow rates, and the applicable acute and chronic water quality criteria.

As noted in Subrule 61.2(4) of the Water Quality Standards, the chronic criteria must be met at the boundary of the MZ and the acute criteria must be met at the boundary of the ZID. A simple mass balance of pollutants shown in Equation (3.2) is used to meet these boundary conditions.

$$C_rQ_r + C_eQ_e = C_s(Q_e + \text{MZ or ZID } Q_r) \qquad (3.2)$$

where:

C_r = Background concentration, mg/l
(Q_e + MZ or ZID Q_r) = Stream flow in the MZ or ZID, cfs
Q_e = Effluent flow, cfs
C_s = Applicable water quality standard, mg/l
C_e = WLA concentration, mg/l

This equation is solved four times for Ce: one time each for ADW acute, ADW chronic, AWW acute, and AWW chronic. The results are wasteload allocations for the protection of the acute criteria and wasteload allocations for the protection of the chronic criteria. These wasteload allocation values are then carried forward to the Permit Derivation Procedure Section.

3.5 Total Residual Chlorine (TRC)

Total Residual Chlorine (TRC) effluent limits will be calculated for any wastewater treatment facility discharging TRC into or impacting one of the four Class B waters and general uses. The applicable stream standard criteria are listed in Subrule 61.2(5) of the Water Quality Standards.

Calculations

Two types of calculations are available for determining effluent limits: hand calculations, noted above for toxics, and first order decay of TRC. The Department has a spreadsheet available in Microsoft Excel to solve for the TRC decay equation when it is applicable. The TRC decay equation may be used to calculate TRC decay in the general use reach, discharge pipe or tile lines. Background flow, defined as the sum of all upstream flows and any incremental flows along the modeled reach, can be added at one of the reach entries in the Microsoft Excel spreadsheet. The incremental flows should be included at the appropriate distance below the discharge. Most calculations will use the mass balance hand calculations for Toxic Parameters described previously.

In addition to the TRC decay calculations for the general reach, a TRC loss of 300 µg/L is assumed in the Zone of Initial Dilution (ZID) and the Mixing Zone (MZ) of designated streams.

Two sets of example calculations will be shown for TRC: one for a general use water receiving an upstream wastewater treatment plant discharge with a zero background flow, and one for a discharger to a general use water on which a background or upstream flow exists.

TRC Calculations with Zero Background Flow

Two steps are used in the calculation of a TRC Wasteload Allocation (WLA) for a general use water receiving an upstream wastewater treatment plant discharge with a zero background flow. The first step is needed only if the discharge is directly into a designated stream. Both the mass balance equation (including 300 µg/l TRC loss) and the TRC decay equation are used in these situations. The $7Q_{10}$ and $1Q_{10}$ flows will be used in the following examples. The calculation of a TRC WLA will use the applicable design low flow.

First, the $WLA_{chronic}$ and WLA_{acute} values are calculated using the modified TRC mass balance equation in the designated portion of the receiving stream. Second, the more stringent $WLA_{acute\ or\ chronic}$ value is used in the TRC decay equation to calculate the allowable WLA just downstream of the outfall in the general use reach. The overall situation for this type of WLA is shown in the TRC Decay with Zero Background Flow Diagram Examples (Diagrams 1, 2, and 3).

First Step:

The following modified TRC mass balance equation is used for solving for Co.

$$CbQb + CoQo = Cs(Qb + Qo) + 300\ Qo \qquad (3.3)$$

where:
Cb = Background TRC concentration in Class B stream, µg/l
Qb = Stream flow in the mixing zone, zone of initial dilution,cfs
Qo = Effluent flow, cfs
Cs = TRC criteria (acute or chronic, µg/l)
Co = WLA TRC concentration, µg/l

Example of modified TRC Mass Balance Equation:

The modified TRC mass balance equations are calculated for the Mixing Zone (MZ) and Zone of Initial Dilution (ZID) to find the chronic wasteload allocation and the acute wasteload allocation.

WLA chronic Calculation Example (using the MZ):

(This calculation must be done for both ADW and AWW flows.)

where:

using 7Q10 = 20 cfs, 1Q10 = 10 cfs
Cb = 0.0 µg/l
Qb = ¼(7Q10) = 20/4 = 5 cfs
Qo = 10 mgd (15.47cfs)

$Cs = 20$ µg/l chronic criterion

$CbQb + CoQo = Cs(Qb + Qo) + 300\ Qo$

$(0.0)5 + Co(15.47) = 20(5 + 15.47) + 300(15.47)$

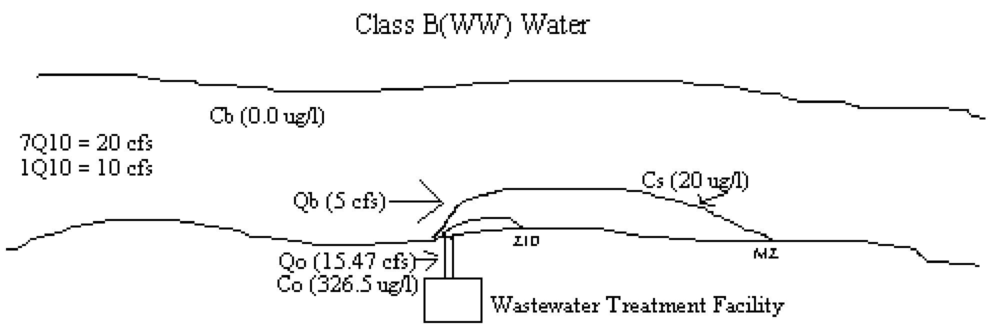
$0 + Co(15.47) = 20(20.47) + 300(15.47)$

$$Co = \frac{20(20.47) + 300}{15.47}$$

$Co = 326.46 = 326.5$ µg/l WLA chronic

WLA chronic Diagram for Shoreline Discharge:
Diagram 1 illustrates a shoreline discharge to a designated stream. The following diagram illustrates the above WLA chronic Calculation Example.

Diagram 1:



KEY:

$7Q_{10}$ = 20 cfs
$1Q_{10}$ = 10 cfs
C_b = Background TRC concentration, ug/l
Q_b = Stream flow in MZ, cfs
Q_o = Effluent flow, cfs
C_s = Water quality standard concentration in the MZ, ug/l
C_o = WLA chronic, ug/l

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WLA $_{acute}$ Calculation Example (using the ZID):

(This calculation must be done for both ADW and AWW flows.)

where:

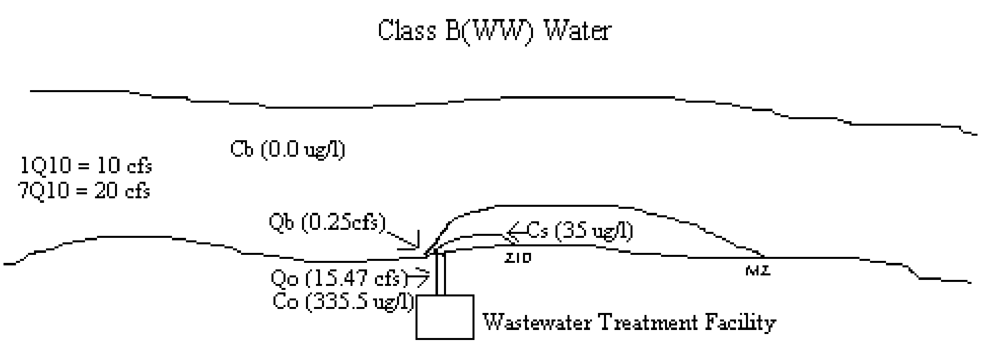
using $1Q_{10} = 10$ cfs
$C_b = 0.0\ \mu g/l$
$Q_b = 1/40(1Q_{10}) = 10/40 = 0.25$ cfs
$Q_o = 10$ mgd (15.47 cfs)
$C_s = 35\ \mu g/l$ acute

$$C_bQ_b + C_oQ_o = C_s(Q_b + Q_o) + 300\ Q_o$$
$$(0.0)0.25 + C_o(15.47) = 35(0.25 + 15.47) + 300(15.47)$$
$$0 + C_o(15.47) = 35(15.72) + 300(15.47)$$
$$C_o = \frac{35(15.72)}{15.47} + 300$$
$$C_o = 335.5\ \mu g/l \text{ WLA }_{acute}$$

WLA $_{acute}$ Diagram for Shoreline Discharge:

Diagram 2 illustrates a shoreline discharge to a designated stream. The following diagram illustrates the above WLA $_{acute}$ Calculation Example.

Diagram 2:



KEY:

$1Q_{10} = 10$ cfs
$7Q_{10} = 20$ cfs
C_b = Background TRC concentration, ug/l
Q_b = Stream flow in ZID, cfs
Q_o = Effluent flow, cfs
C_s = Water quality standard concentration in the ZID, ug/l
C_o = WLA acute, ug/l

Second Step:
The decay model uses a standard first order expression in which the time of travel in the stream reach is incorporated into the calculations. The model expression noted in the EPA's "Technical Guidance Manual for Performing Wasteload Allocations; Book 2, Chapter 3, Toxic Substances" June 1984, Appendix D, is used for TRC decay. The TRC decay equation is used when there is a discharge to a general use water (having zero flow). The decay equation will project the amount of TRC loss along the general use reach. The resulting WLA is more relaxed than the WLA calculated in the mass balance equation for the direct discharge to the designated reach. The following TRC decay equation is used, solving for Cd.

$$Cd = C_o e^{(kt)} \qquad (3.4)$$

where:

Cd = TRC upstream discharge concentration at time t, µg/l
Co = WLA TRC concentration, µg/l
k = Decay rate constant, day –1
t = Time of travel in modeled reach, day

The more stringent of the WLA acute or chronic from the first step is used in the second step. For these examples, the more stringent of the WLA acute or chronic is the WLA chronic value of 326.5 µg/l. This value will be used for Co in the TRC decay with zero background flow example.

TRC Decay with Zero Background Flow Example:

where:
Co = WLAchronic = 326.5 µg/l
k = 20 day –1
t = 0.204 day (1760 ft. upstream at 0.1 ft./sec.)
t = d/v = 1760/0.1 = 17,600 sec.
17,600 sec./86,400 (sec. in a day) = 0.204 day

$$C_d = C_o e^{(kt)}$$
$$= 326.5e(20)(0.204)$$
$$= 326.5(59.145)$$
$$Cd = 19{,}311\ \mu g/l$$

TRC Decay Diagram with Zero Background Flow:
Diagram 3 illustrates TRC decay along a general use stream into a Class B(WW) Water.

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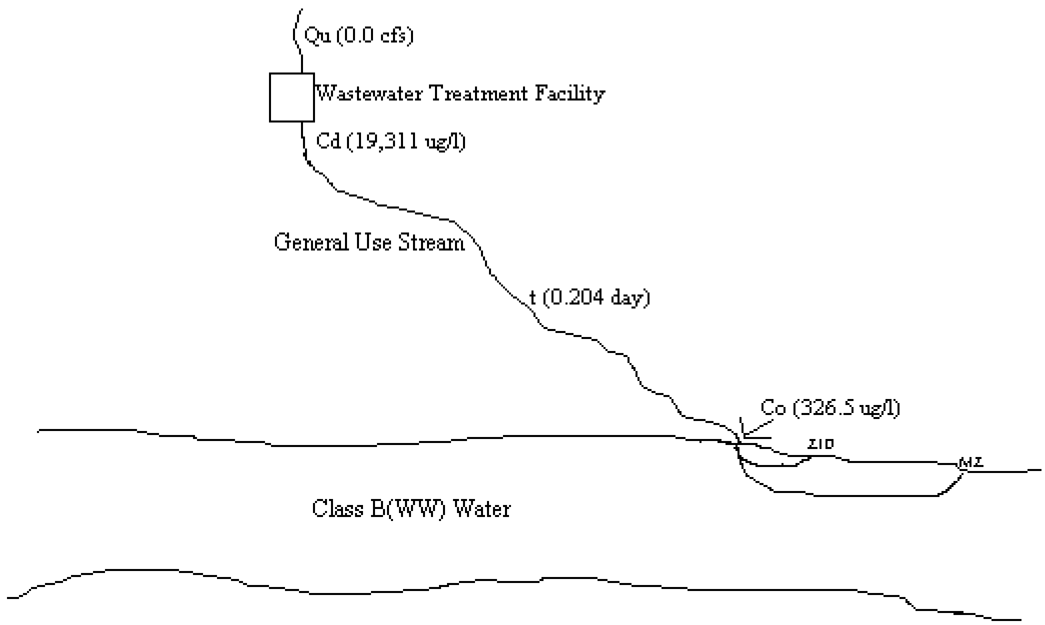
Diagram 3:



KEY:

Q_u = Background or upstream flow, cfs
C_d = TRC upstream discharge concentration at time t, ug/l
C_o = WLA TRC concentration, ug/l
k = Decay rate constant, 20 day $^{-1}$
t = Time of travel in modeled reach, day

TRC Calculations with Background Flow

Three steps are used to calculate the WLA for a discharger to a general use stream on which a background (or upstream) flow exists. Both the modified TRC Mass Balance and the TRC decay equations are used in this situation. First, the $WLA_{chronic}$ and WLA_{acute} values are calculated using the modified TRC Mass Balance equation for the designated portion of the receiving stream. Second, the $WLA_{\text{chronic and acute}}$ for ADW flow and the $WLA_{\text{chronic and acute}}$ for AWW flow are used in the TRC decay equation to calculate the allowable WLA just downstream of the outfall in the general reach. Finally, the actual WLAs for the outfall are calculated using the modified TRC mass balance equation and the upstream flow and concentration. The overall situation for this type of WLA is shown in the TRC Decay with Background Flow Diagram Examples (Diagrams 4,5,6, and 7).

First Step:
The modified TRC mass balance equation in designated water:

$$C_bQ_b + C_oQ_o = C_s(Q_b + Q_o) + 300\ Q_o \qquad (3.5)$$

WLA $_{chronic}$ Calculation with Background Flow Example (using the MZ):

(This calculation must be done for both ADW and AWW flows.)

where:

using $7Q_{10} = 20$ cfs

$C_b = 0.0$ µg/l

$Q_b = ¼(7Q_{10}) = 20/4 = 5$ cfs

$Q_o = \Sigma(Q_D + Q_u)$

Q_D = Discharge flow = 10 mgd (15.47 cfs)

Q_u = Background or upstream flow (1 cfs)

$Q_o = \Sigma(15.47 + 1)$

$Q_o = 16.47$ cfs

$C_s = 20$ µg/l chronic

$C_bQ_b + C_oQ_o = C_s(Q_b + Q_o) + 300\ Q_o$

$(0.0)5 + C_o(16.47) = 20(5 + 16.47) + 300\ (16.47)$

$0 + C_o(16.47) = 20(21.47) + 300\ (16.47)$

$$C_o = \frac{20(21.47)}{16.47} + 300$$

$C_o = 326.07$ µg/l WLA $_{chronic}$

WLA $_{chronic}$ Diagram with Background Flow:

Diagram 4 illustrates a discharge to a general use stream that discharges into a designated stream on which a background (or upstream) flow exists. The following diagram illustrates the previous WLA $_{chronic}$ Calculation with Background Flow Example.

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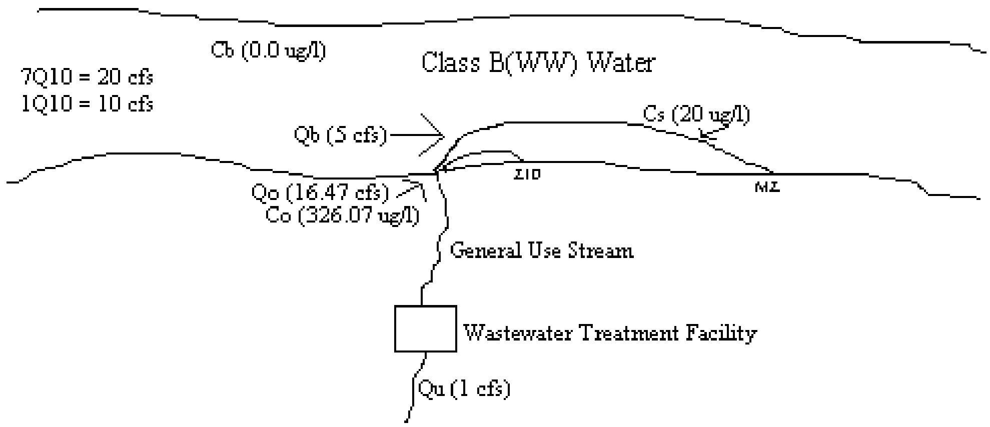
Diagram 4:



KEY:

$7Q_{10}$ = 20 cfs
$1Q_{10}$ = 10 cfs
C_b = Background TRC concentration, ug/l
Q_b = Stream flow in MZ, cfs
Q_o = Sum of discharge flow and background flow, cfs
C_s = Water quality standard concentration in the MZ, ug/l
Q_u = Background or upstream flow, cfs

WLA $_{acute}$ Calculation with Background Flow Example (using the ZID):

(This calculation must be done for both ADW and AWW flows.)

where:

using $1Q_{10}$ = 10 cfs

$C_b = 0.0$ µg/l

$Q_b = 1/40(1Q_{10}) = 10/40 = 0.25$ cfs

$Q_o = \Sigma(Q_D + Q_u)$

Q_D = Discharge flow = 10mgd (15.47 cfs)

Q_u = Background or upstream flow (1 cfs)

$Q_o = \Sigma(15.47 + 1)$

$Q_o = 16.47$ cfs

$C_s = 35$ µg/l acute

$C_bQ_b + C_oQ_o = C_s(Q_b + Q_o) + 300\ Q_o$

$(0.0)0.25 + C_o(16.47) = 35(0.25 + 16.47) + 300\ Q_o$

$0 + C_o(16.47) = 35(16.72) + 300\ Q_o$

$C_o = \frac{35(16.72)}{16.47} + 300$

$C_o = 335.5$ µg/l WLA_{acute}

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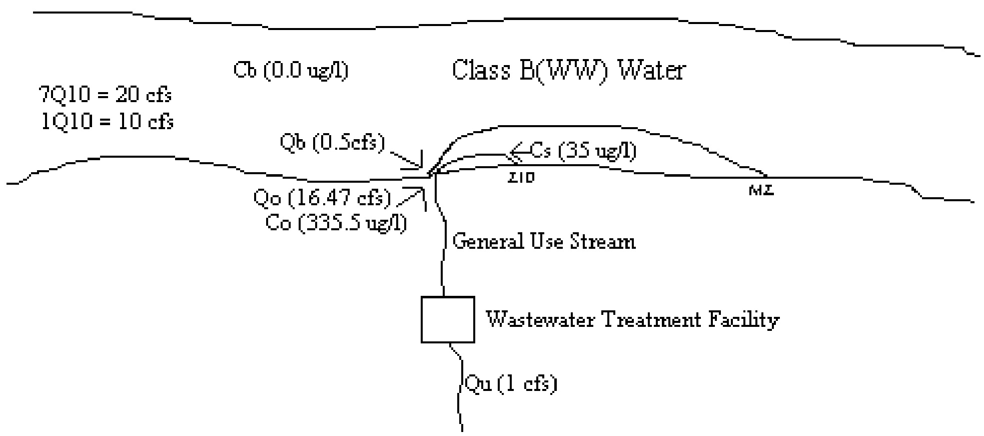
WLA $_{acute}$ Diagram with Background Flow:
Diagram 5 illustrates a discharger to a general use stream that discharges into a designated stream on which a background (or upstream) flow exists. The following diagram illustrates the previous WLA $_{acute}$ Calculation with Background Flow Example.

Diagram 5:



KEY:

$7Q_{10}$ = 20 cfs
$1Q_{10}$ = 10 cfs
C_b = Background TRC concentration, ug/l
Q_b = Stream flow in ZID, cfs
Q_o = Sum of discharge flow and background flow, cfs
C_s = Water quality standard concentration in the ZID, ug/l
Q_u = Background or upstream flow, cfs

Second Step:
The WLA $_{chronic\ or\ acute}$ for ADW flow and WLA $_{chronic\ or\ acute}$ for AWW flow from the above step are used in the TRC decay equation. For this example, the more stringent of the WLA $_{chronic\ or\ acute}$ is the WLA $_{chronic}$ value of 326.07 µg/l. The TRC decay over time "t" is used to calculate the upstream concentration (C_o). The following TRC decay equation for an upstream general waterway with background flow is used for solving for C_{db}.

$$C_{db} = C_o e^{(kt)} \tag{3.6}$$

where:

C_{db} = TRC upstream discharge concentration at time t, µg/l considering background flow (just below outfall)
C_o = WLA TRC upstream concentration, µg/l
k = Decay rate constant, day $^{-1}$
t = Time of travel in modeled reach, day

TRC Decay for Upstream General Waterway with Background Flow Example:

where:
$C_o = WLA_{chronic} = 326.07$ µg/l
k = 20 day $^{-1}$
t = 0.204 day

$C_{db} = C_o e^{(kt)}$
$= 326.07e^{(20)(0.204)}$
$= 326.07(59.145)$
$C_{db} = 19{,}285$ µg/l

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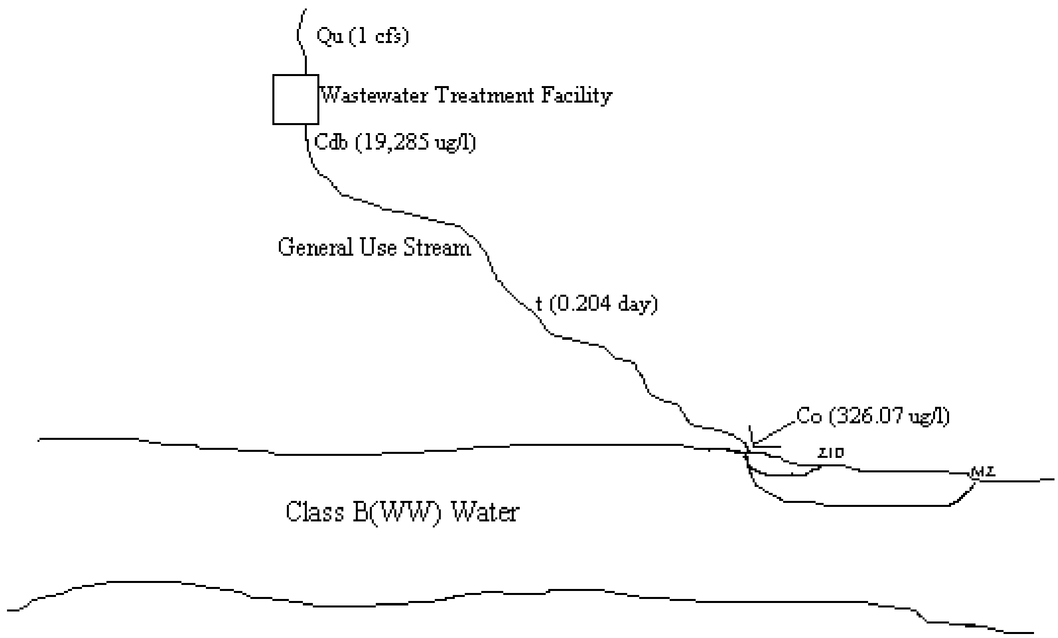
TRC Decay Diagram with Background Flow:
Diagram 6 illustrates TRC decay along a general use stream that discharges into a Class B(WW) water with a background flow.

Diagram 6:



KEY:

Q_u = Background or upstream flow, cfs
C_{db} = TRC upstream discharge concentration at time t, ug/l (just below outfall)
C_o = WLA TRC concentration, ug/l
k = Decay rate constant, 20 day $^{-1}$
t = Time of travel in modeled reach, day

Third Step:
The discharge flow, upstream TRC concentration, upstream flow in the general reach, and the calculated C_{db} from above will be used in the basic mass balance equation to calculate the amount of TRC for the outfall. In the mass balance equation the effluent concentration (WLA) is noted as C_d.

$$C_uQ_u + C_dQ_d = C_{db}(Q_u + Q_d) \qquad (3.7)$$

where:

C_u = Background TRC concentration in General Use stream, µg/l
Q_u = Background or upstream flow in the general reach, cfs
Q_d = Effluent flow, cfs
C_{db} = Discharge TRC concentration, µg/l considering background flow
C_d = TRC discharge (outfall) concentration at time “t”, µg/l

TRC Mass Balance Equation at Outfall Location Calculation Example:

where:

C_u = 0.0 µg/l
Q_u = 1 cfs
Q_d = 10 mgd (15.47cfs)
C_{db} = 19,285 µg/l

$$C_uQ_u + C_dQ_d = C_{db}(Q_u + Q_d)$$
$$(0.0)1 + C_d(15.47) = 19{,}285(1 + 15.47)$$
$$0 + C_d(15.47) = 19{,}285(16.47)$$
$$C_d = \frac{19{,}285(16.47)}{15.47}$$

C_d = 20,500 µg/l (20.5 mg/l) TRC discharge (outfall) concentration at time ‘t”

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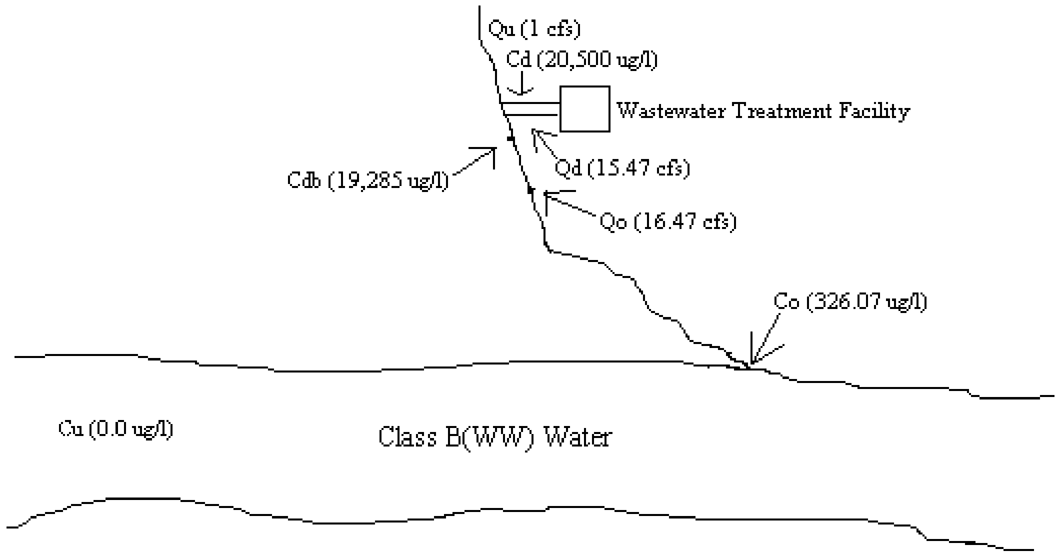
TRC Mass Balance Equation Diagram at Outfall Location:

Diagram 7 illustrates the amount of TRC WLA for the outfall. The following diagram illustrates the previous mass balance equation.

Diagram 7:



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KEY:

Q_u = Background or upstream flow, cfs
C_d = TRC discharge (outfall) concentration at time t, ug/l
Q_d = Effluent flow, cfs
C_{db} = Discharge TRC concentration, ug/l considering background flow
Q_o = Sum of discharge flow and background or upstream flow, cfs

3.6 Bacteria

IAC 61.3(3)"a"(1) shows the *E. coli* criteria Table that are applicable to designated Class "A" waters. Waters which are designated as Class "A1," "A2," or "A3" in subrule 61.3(5) are to be protected for primary contact, secondary contact, and children's recreational uses, respectively. The general criteria of subrule 61.3(2) and the specific *E. coli* criteria apply to all Class "A" waters. The implementation of *E. coli* criteria are presented for two different discharges, continuous discharges and intermittent discharges.

Continuous Discharges:

After Iowa adopted the current *E. coli* criteria in 2003, EPA published the BEACH Act Rule in 2004 (69 FR 67217, November 16, 2004). In the BEACH rule, EPA indicated that it expected that the single sample maximum values would be used for making beach notification and closure decisions. EPA recognized, however, that States and Territories also use criteria in their water quality standards for other purposes under the Clean Water Act in order to protect and improve water quality. Other than in the beach notification and closure decision context, the geometric mean is the more relevant value for ensuring that appropriate actions are taken to protect and improve water quality. The geometric mean is generally more relevant because it is usually a more reliable measure of long term water quality, being less subject to random variation, and more directly linked to the underlying studies upon which the 1986 bacteria criteria were based.

Accordingly, Iowa revised Chapter 62 regarding the implementation of the sample maximum criterion, which became effective on October 14, 2009. The revised Chapter 62 states "…that the daily sample maximum criteria for *E. coli* set forth in Part E of the 'Supporting Document for Iowa Water Quality Management Plans' shall not be used as an end-of-pipe permit limitation."

Therefore, based on the BEACH Rule regulation and the revision of Chapter 62, only the geometric mean limit of 126 org./100 mL applies to continuous discharges.

Intermittent Discharges:

The use of the geometric mean limit of 126 org./100 mL makes establishing limits somewhat difficult for intermittent discharges such as controlled discharge lagoons and CSOs, since there may not be adequate sample data to calculate a geometric mean due to their limited duration of discharge (current rules require no less than five samples evenly distributed during a 30-day period to calculate the geometric mean). Thus, a single sample maximum value based on the same risk levels or same level of protection as the geometric mean value is used for these types of discharges.

The current *E. coli* geometric mean criterion of 126 org/100 mL was developed based on a risk level of 8 illnesses per 1000 people. In order to derive an equivalent sample maximum value corresponding to the same risk level as the geometric mean of 126 org/100 mL, the variability of *E. coli* levels should be taken into account. Based on the EPA 1986 Bacteria Document, when a site-specific coefficient of variance is not available, a default log coefficient of variance of 0.4 can be used for freshwaters. Based on the default coefficient of variance of 0.4 and the geometric mean value of 126 org/100 mL, the sample maximum values at different confidence levels can be calculated as shown in Table 8.1. The 99th percentile value of 1,073 org/100 mL is selected as the sample maximum value since it is consistent with Iowa's current permit derivation procedure and it ensures the geometric mean is protected 99% of the time for the same risk level (0.8%). Thus, an effluent sample maximum value below 1,073 org/100 mL will ensure that there is a greater than 99% probability that the waterbody will meet a geomean criterion of 126 org./100 mL. Correspondingly, an effluent sample maximum value of 5,365 org./100 mL is used to meet Class A2 uses.

Table 3.7 Water Quality Criteria for Bacteria for Fresh Recreational Waters

Indicator	Geometric Mean Density	Single Sample Maximum Allowable Density*			
		75th percentile	90th percentile	95th percentile	99th percentile
E coli	126	235	410	576	1,073

* Sample maximum values are calculated based on a log coefficient of variance of 0.4

***E. coli* Decay Rate:**

The *E. coli* decay equation is used when there is a discharge to a non-Class A water. The decay equation will project the amount of *E. coli* loss along the non-Class A stream reach. The decay model uses a traditional relationship in which time of travel in the non-Class A designated stream reach is incorporated into the calculations. The model formulated in the EPA publication "Rates, Constants and Kinetics Formulation in Surface Water Quality Modeling" (Second Edition), June 1985, is used for *E. coli* decay. The resulting WLA is more relaxed than the WLA calculated for a direct discharge to the designated reach. The following *E. coli* equation is used when there is zero background flow in the non-Class A water, solving for Cd.

$$C_d = C_o e^{(kt)} \qquad (3.8)$$

where:

Cd = *E. coli* discharge concentration, # org./100 ml
Co = Water Quality Standard, # org./100 ml – Geometric Mean (GM)
k = Decay rate constant, day $^{-1}$
t = Time of travel in modeled reach, day

The *E. coli* criteria value from the water quality standard in the designated segment for Class A must be used in the above equation.

The EPA 1985 document titled “Rates, constants and kinetics formulations in surface water quality modeling. 2nd ed. EPA/600/3-85/040” summarized about twelve decay rates for streams and rivers, and six decay rates for lakes and ponds. The decay rates came from studies conducted from 1920’s to 1980’s.

The Department analyzed the decay rate data published in the 1985 EPA document and focused on both stream/river and pond decay rates.

The Analysis of the Original Decay Rates:

The mean and median decay rates based on the original decay rates are shown in Table 3.8:

Table 3.8. Summary Statistics of the Original Decay Rates

k(1/day)	Rivers & Streams	Lakes & Ponds
Mean	3.37	2.31
Median	1.03	1.46

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The Analysis of the Log-transferred Decay Rates:

The decay rates were first log-transferred since the decay rates are log normally distributed. The decay rates were re-analyzed using the log-transformed data. The purpose of using log-transferred data is to calculate the confidence interval values for the median values.

All of the bacteria decay study data in the EPA document are in-situ studies (that is, they are field study data, not laboratory data), and they are decay rates for coliforms. Table 3.9 shows the summary statistics of the original data assuming a log-normal distribution for the decay rates:

Table 3.9. Summary Statistics Using Log-transferred Decay Rates

k(1/day)	Rivers & Streams	Lakes & Ponds
Mean	2.90	2.57
Median	1.30	1.15
95% Confidence Interval	0.85 – 1.97	0.62 – 2.15

The statistical analysis of the log transformed data for streams and rivers show the median value is about 1.30/day, and the 95% confidence intervals for the median value are 0.85/day and 1.97/day. The statistical analysis of the log transformed data for lakes and ponds show the median value is about 1.15/day, and the 95% confidence intervals for the median value are 0.62/day and 2.15/day. The recalculated median values using log-normal distribution are a little different than the median values calculated using original data. To minimize the data transfer errors, the median values based on the original data shown in Table 3.8 are used as the bacteria decay rates for streams and ponds. The decay rates for streams and ponds are1.03/day and 1.46/day, respectively.

3.7 Chloride and Sulfate

The chloride and sulfate criteria are included in IAC 61.2(4)"b"(1). Both criteria are hardness dependent. Chloride criteria also depend on sulfate concentrations and sulfate criteria are a function of chloride levels. Thus, it is necessary to determine the water chemistry parameters before the criteria can be applied.

Statewide Default Water Chemistry Values

Chloride and sulfate toxicity are both heavily dependent on water hardness. To a lesser degree chloride toxicity is dependent on the sulfate concentration of the waters, while sulfate is dependent on the chloride concentration of the waters. For those situations where site-specific water chemistry may not be available, statewide default water chemistry values were developed. The values were determined by analyzing DNR ambient water monitoring data from 2000 to 2007. The statewide default background concentrations are presented below:

- Hardness – 200 mg/L as $CaCO_3$
- Sulfate – 63 mg/L
- Chloride – 34 mg/L

Utilizing the above values, the water quality criteria for chloride results in an acute concentration of 629 mg/L and a chronic concentration of 389 mg/L. For sulfate, the default water quality criterion for aquatic life protection is 1,514 mg/L.

WLAs for Chloride and Sulfate:

When site-specific water chemistry data for ambient and/or effluent are either not available or not adequate to develop site-specific water chemistry values, statewide default water chemistry values will be used to calculate the chloride and sulfate criteria, and to develop the wasteload allocations.

Wasteload allocations for point source dischargers in regard to chloride and sulfate will be calculated in the same manner as those pollutants listed in Table 1 (Iowa Administrative Code – 567, Chapter 61, Water Quality Standards). The acute wasteload allocation will be calculated with the use of the 1Q10 stream flow and be applied at the boundary of the zone of initial dilution (ZID). The chronic wasteload allocation will use the 7Q10 stream flow in its calculation and be applied at the end of the mixing zone (MZ). Due to the fact that sulfate criterion is a single value criterion, it will be applied at both the end of MZ and the ZID.

A simple mass balance of pollutants will be used to meet these boundary conditions.

$$CrQr * MZ \text{ or } ZID + CeQe = Cs(Qb + Qe) \qquad (3.9)$$

where:
Cb = Background concentration, mg/L
Qb = Stream critical low flow, cfs
Qe = Effluent flow, cfs
Cs = Applicable water quality standard, mg/L
MZ or ZID = Mixing Zone or Zone of Initial Dilution (0 - 1)
Ce = WLA concentration, mg/L

This equation is solved four times for Ce: one time each for ADW acute, ADW chronic, AWW acute, and AWW chronic. The results are wasteload allocations for the protection of the acute criteria and wasteload allocations for the protection of the chronic criteria. These wasteload allocation values are then carried forward to the Permit Derivation Procedure section.

3.8 Thermal Discharges (Temperature WLA)

DEFINITION OF TERMS USED IN THIS SECTION

c	specific heat (BTU/lb/°F)
D	dilution ratio (Qe + Qr MZ)/Qe, unitless
H_e	facility heat rejection (million BTU/day)
H_{Tmax}	daily maximum heat rejection limit
$H_{\Delta T\uparrow}$	monthly average heat rejection limit
$H_{\Delta T/hour}$	allowable heat rejection rate in million BTU/hour
m	mass of body gaining or losing heat (lb)
MZ	allowed mixing zone percentage divided by 100
Q	flow rate, cfs
Q_{7-10}	seven-day, 10 year low flow (cfs)
Q_e	facility discharge flow in million gallons per day (MGD)
$Q_e + Q_r$ MZ	stream flow downstream of facility discharge in cubic feet per second
Q_r	design stream flow in cubic feet per second (cfs)
ΔT	temperature change
$\Delta T\uparrow$	allowable temperature increase
Te	facility discharge temperature (ºF)
$T_{e\text{-}average}$	average allowable effluent temperature
Tmax	maximum allowable downstream temperature after mixing zone(ºF)
$T_{e\text{-}max}$	maximum allowable effluent temperature
$T_{e\text{-}max1\%}$	allowed effluent temperature for 1% of the hours in 12-months period, °C
$Te_{\text{-}rate}$	maximum allowable effluent rate of temperature change (°F/ hr)

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T_{in}	facility intake temperature (°F)
T_r	upstream stream temperature (°F)
T_{II} or T_{III}	Maximum allowed river temperature in the Mississippi River Zone II or III defined in WQS is shown in Table 7.1

SUMMARY OF TEMPERATURE CRITERIA

The temperature water quality criteria are included in 567 IAC 61.3(3)b(5). The following is a summary of the criteria.

(1) $\Delta T\uparrow$: temperature shall not be increased more than 3°C (5.4 °F) for warm water streams and 2°C (°F) for cold water streams
(2) T_{max}: maximum temperature cap. For all warm waters except the Mississippi River and for cold waters, the temperature caps are 32°C and 20°C respectively. These criteria apply at all times. For the Mississippi River, the temperature cap values vary by month and there are two temperature cap values; one is the absolute temperature cap never to be exceeded, and the other prohibits exceedance for more than 1% of the hours (86.4 hours) in the 12-month period ending with any month.
(3) ΔT/hour: rate of change. The rate of temperature change shall not exceed 1°C/hour.

In addition, 567 IAC 61.2(5)"a" and IAC 61.2(5)"b" include the following statements:

a. The allowable 3°C temperature increase criterion for warm water interior streams, 61.3(3)"b"(5)"1," is based in part on the need to protect fish from cold shock due to rapid cessation of heat source and resultant return of the receiving stream temperature to natural background temperature. On low flow streams, in winter, during certain conditions of relatively cold background stream temperature and relatively warm ambient air and groundwater temperature, certain wastewater treatment plants with relatively constant flow and constant temperature discharges will cause temperature increases in the receiving stream greater than allowed in 61.3(3)"b"(5)"1."

b. During the period November 1 to March 31, for the purpose of applying the 3°C temperature increase criterion, the minimum protected receiving stream flow rate below such discharges may be increased to not more than three times the rate of flow of the discharge, where there is reasonable assurance that the discharge is of such constant temperature and flow rate and continuous duration as to not constitute a threat of heat cessation and not cause the receiving stream temperature to vary more than 3°C per day.

The following describes how the department will implement the temperature criteria.

HEAT TRANSFER THEORY AND HEAT-BASED LIMITS

In any heat transfer situation, the amount of heat gained or lost may be mathematically defined as:

$$H = mc\Delta T \qquad (3.10)$$

For simplicity in water quality calculations, the mass (m) of the stream or wastewater is usually expressed as a flow rate (Q) and is expressed in terms of million gallons per day (MGD) or cubic feet per second (cfs). The specific heat (c) of water can be assumed to be 1 since a BTU or British Thermal Unit is defined as the amount of heat required to raise one pound of water by 1°F. Incorporating a flow rate instead of a mass yields results in terms of the rate of heat transfer or heat rejection rate. Equation 3.10 incorporating appropriate conversion factor becomes:

$$H = Q\Delta T \qquad (3.11)$$

Thus, in any closed system, the amount of heat gained or lost may be determined from the heat transfer equation:

$$(Q\Delta T)\text{ gained} = (Q\Delta T)\text{ lost}$$

For the purpose of establishing effluent limits, all thermal discharges fall into one of two categories based upon the source of cooling water, as illustrated by Figure 1. Case 1 situations are those where the source of cooling water is the receiving stream upstream of the point of discharge. Case 2 situations are those where the source of cooling water is not the receiving stream, such as a municipal water system, a well, or from a different waterbody. The form of the heat transfer equation to be used to calculate effluent limits differs for Case 1 and Case 2 situations as explained in the following sections:

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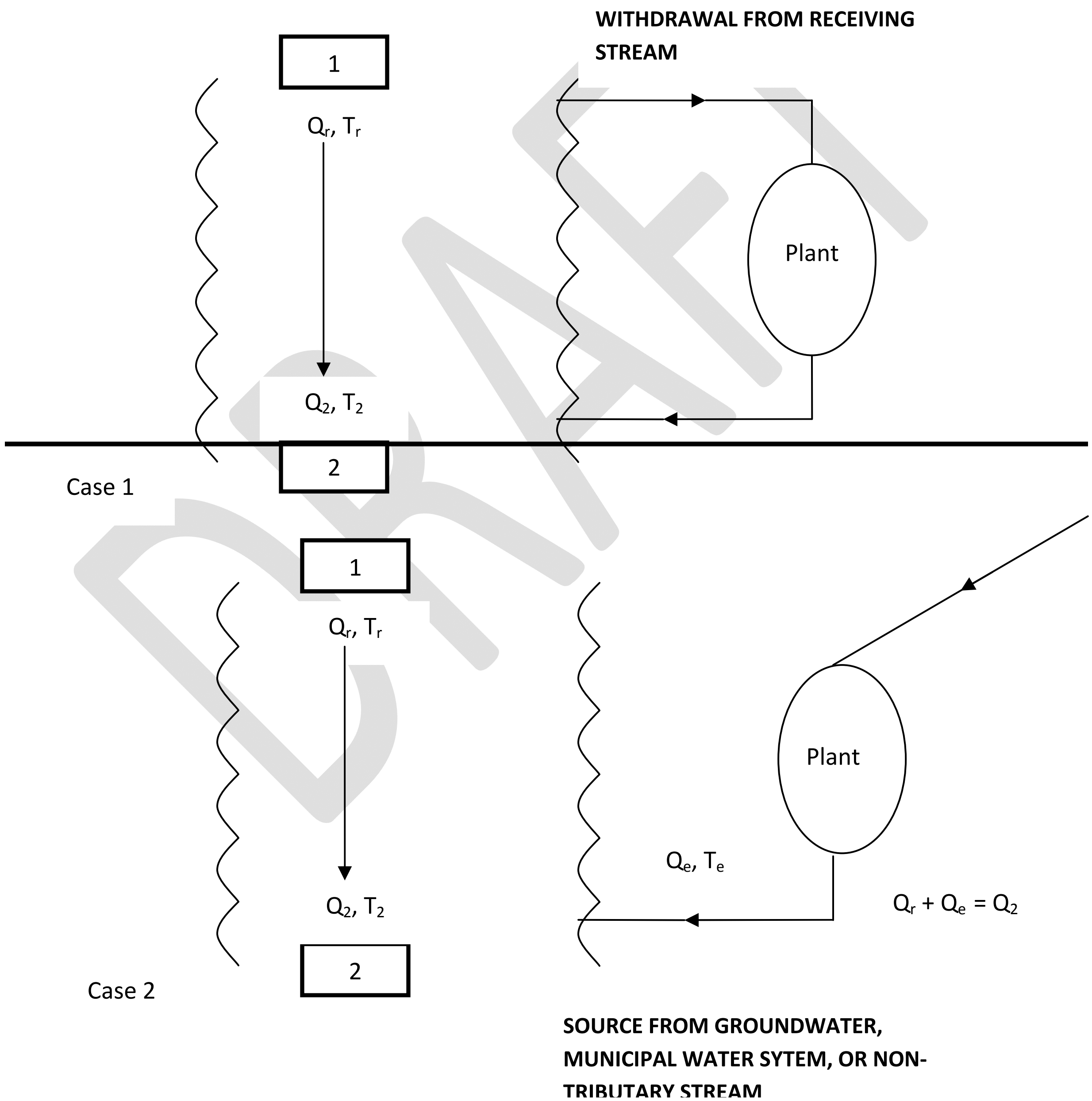
FIGURE 1



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Case 1 - Withdrawal From Receiving Stream

When using the following formulas if million gallons per day (MGD) are used as the units of stream flow instead of cfs, the conversion factor is 8.34 instead of 5.39.

T_{max} criterion implementation:
Figure 1 illustrates the physical layout for a typical Case 1 situation. In the Case 1 situation, effluent limits should be expressed as heat rejection rates, usually in million BTUs/day, unless intake temperature is not available using this equation:

$$H_{Tmax} = Q_r (T_{max} - T_r) * MZ * 5.39 \qquad (3.12)$$

$\Delta T\uparrow$ criterion implementation:
To calculate effluent limits based on the $\Delta T\uparrow$ criterion replace (T_{max} - T_r) in equation 3.12 with the $\Delta T\uparrow$ criterion of 3 ºC (or 5.4 ºF):

$$H_{\Delta T_\uparrow} = Q_r*(5.4\ ºF) *MZ * 5.39 \qquad (3.13)$$

Rate of change calculation: Effluent limits based on the ΔT criterion of 1 ºC/hour (or 1.8 ºF/hour) are calculated using equation 3.14.

$$H_{\Delta T/hour} = Q_r*(1.8\ ºF/hour) *MZ * 5.39/24 \qquad (3.14)$$

A default mixing zone (MZ) of 25% of either the annual or monthly $Q_{7\text{-}10}$ flow for interior streams, and 10% of either the annual or monthly $Q_{7\text{-}10}$ flow for the Mississippi and Missouri Rivers will be used unless site-specific mixing zone study data are available. For Mississippi and Missouri River side channels, sloughs and backwaters 10% of the volume of water that flows through the side channel will be used as the default mixing zone. For interior stream side channels, 25% of the volume of water that flows through the side channel will be used as the default mixing zone.

The default upstream temperature values found in the Default Statewide Water Chemistry Values section of this document will be used unless site-specific upstream temperature data are available.

Determining Compliance:

For all Case 1 situations, the facility discharge flow Q_e, the discharge temperature T_e, and the intake temperature T_{in} may be monitored at least hourly to determine compliance with the allowable heat rejection limit. The discharge heat rejection rate is calculated as follows:

$$H_e= Q_e (T_e - T_{in}) * 8.34 \qquad (3.15)$$

To determine compliance with the rate of change limit ($H_{\Delta T/hour}$), the discharge heat rejection (H_e) needs to be converted from million BTU/day to million BTU/hour by dividing by 24.

The $H_{\Delta T\uparrow}$ limit will be used as the monthly average permit limit, and the H_{Tmax} limit will be used as the daily maximum limit. The $\Delta T\uparrow$ allowable temperature increase is for the purpose of maintaining a well-rounded population of warmwater fishery, and to protect fish that are acclimated to the warmer temperature as a result of the discharge from cold shock due to rapid cessation of heat source from the discharge and resultant return of the receiving stream temperature to natural background temperature. T_{max} is the upper incipient temperature allowable for fish to survive.

In Case 1 situations, permit limits based on heat rejection rate (million BTU/day) should be established whenever possible because these limits provide a facility more flexibility in achieving compliance. Permit limits expressed as a fixed temperature (ºF) must be calculated using a facility's discharge flow rate whereas permit limits expressed as a heat rejection rate are not. When calculating fixed temperature limits the department must assume that the maximum discharge flow rate can occur when stream flow is at a minimum resulting in permit limits that are more restrictive whenever the discharge flow is less than the flow used in the calculations or when stream temperature or stream flow are different from the values used in the calculations.
Temperature-based limits in Case 1 situations may be considered under the following conditions:

1. Facility discharge flow rates can be accurately and reliably measured;
2. Facility discharge flow rates do not vary substantially within each month, or; the permittee does not object to a permit that specifies limits that may overly restrictive.
3. The facility cannot conduct the monitoring that is required to calculate the heat rejection rate (intake temperature, facility discharge flow, and discharge temperature).

Case 2 - Withdrawal From Source Other Than Receiving Stream

The Case 2 situation is different from Case 1 in that the intake temperature is likely not the same as the receiving stream temperature upstream of the discharge. Figure 1 presents the physical layout for a typical Case 2 situation.

For case 2 situations, the temperature of the intake water is not known, the heat rejection for the effluent cannot be calculated. In these cases, permit limits must be expressed based on the temperature of the discharge water, and compliance monitoring is accomplished by measuring the temperature of the discharge water. The limitations of this method are as listed in the previous section, in that the adequacy of the permit limits depends on an accurate facility flow balance, and the analyst must assume conservative or maximum facility flow rates. The derivation of temperature limits is shown in the following section.

TEMPERATURE-BASED LIMITS CALCULATIONS

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Temperature-based permit limits will be calculated using a mass balance calculation of the thermal inputs. Formulas for calculating permit limits for the T_{max} and $\Delta T\uparrow$ criteria are as follows:

$$T_{max}: \quad (Q_e + Q_r\ MZ)*T_{max} = Q_r *MZ *T_r + Q_e*T_e \qquad (3.16)$$

$$\Delta T\uparrow: \quad T_e = 3^{o}C * D + T_r \qquad (3.17)$$

Warm Water Interior Streams and the Big Sioux River: Monthly average, daily maximum and rate of temperature change effluent limits will be calculated based on IAC Chapter 61.3(3)"b"(5) temperature criteria. The statewide average background temperatures for streams designated as warm waters described in the Default Statewide Water Chemistry section of this document are used in the calculations.

Monthly Average Limits: WQS requires that "no heat shall be added to interior streams or the Big Sioux River that would cause an increase of more than 3°C". This criterion applies at the end of the regulatory mixing zone. The default mixing zone is 25% of the $Q_{7\text{-}10}$ flow in the receiving stream and will be used in the calculations unless site-specific mixing zone data are available. The calculation is described by equation 3.18:

$$T_{e\text{-average}} = T_r + 3°C *D \qquad (3.18)$$

Daily Maximum Limits: WQS requires that "in no case shall heat be added in excess of that amount that would raise the stream temperature above 32 °C". The same mixing zone and AWW effluent flow will be used to calculate monthly average limits and daily maximum limits. The calculation is described by equation 3.19:

$$T_{e\text{-max}} = T_r + (32\ °C - T_r) *D \qquad (3.19)$$

Rate of Change (ΔT) Limits: WQS requires that "the rate of temperature change shall not exceed 1°C per hour". The same mixing zone and effluent flow will be used to calculate the monthly average limits and daily maximum limits will be used to calculate ΔT limits. The calculation is described by Equation (3.20):

$$T_{e\text{-rate}}\ (°C/\ hr) = 1\ (°C/hr) *D \qquad (3.20)$$

Equation (3.20) can be used for large heat sources when there are continuous temperature monitoring data. For smaller heat sources, the following special conditions may be included in the wasteload allocations to implement the rate of change criteria.

Special conditions: cessation of thermal inputs to the receiving water by the discharge shall occur gradually so as to avoid fish mortality due to cold shock during the winter months (November through March). The basis for this requirement is to allow fish associated with the discharge-heated mixing zone for the discharge to acclimate to the decreasing temperature. The decrease in temperature at the end of the calculated mixing zone shall not exceed 1ºC per hour.

Cold Water Streams: The procedures for calculating temperature limits for discharges to cold water streams are the same as those for warm waters streams except for the following:

(1) Background temperature values: The statewide background temperature values derived for cold water streams described in the Default Statewide Water Chemistry section of this document are used.
(2) Criteria: IAC Chapter 61.3(3)"b"(5)"2" states that "No heat shall be added to streams designated as cold water fisheries that would cause an increase of more than 2°C. The rate of temperature change shall not exceed 1°C per hour. In no case shall heat be added in excess of that amount that would raise the stream temperature above 20°C." The 3°C $\Delta T\uparrow$ and 32°C T_{max} criteria for warm water streams are replaced by 2°C and 20°C in the formulas.

The Missouri River: The procedures for calculating temperature limits for discharges to the Missouri River are the same as for warm water streams except for the mixing zone. For the Missouri River, the default mixing zone is 10% of the Q_{7-10} flow instead of 25% and this default flow will be used unless site-specific mixing zone data are available.

The Mississippi River: Monthly average, daily maximum and rate of change limits will be calculated according to the temperature criteria described in IAC 61.3(3)"b"(5)"5". An additional criterion for the Mississippi River is that the water temperature shall not exceed the maximum limits in Table 3.10 during more than 1 percent of the hours in the 12-month period ending with any month. The average background temperatures for the Mississippi River Zone II (from Iowa north border to Wisconsin – Illinois border) and Zone III shown in the Default Statewide Water Chemistry section of this document are used in the calculations.

Monthly Average Limits: WQS requires that "no heat shall be added to the Mississippi River that would cause an increase of more than 3°C". This criterion applies at the end of the regulatory MZ. The default mixing zone is 10% of the Q_{7-10} flow for the Mississippi River at the discharge location unless site-specific mixing zone data are available. The calculation is described by equation 3.21:

$$T_e = T_r + (3°C) * D \qquad (3.21)$$

Daily Maximum Limits: WQS requires that at no time shall the water temperature exceed the maximum limits in table 3.10 by more than 2°C. The same mixing zone and AWW effluent flow will be used to calculate the monthly average limits and daily maximum limits. The calculation is described by equation 3.22:

$$T_{e\text{-}max} = Tr + [2 + (T_{II} \text{ or } T_{III}) - Tr] * D \qquad (3.22)$$

Table 3.10: Maximum Allowed River Temperature Set for Mississippi River Zones II & III (River temperature not to exceed the maximum values in the table below for more than 1 percent of the hours in a 12-month period)

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Month	Zone II	Zone III
	Temperature *(°C)	Temperature *(°C)
Jan.	4	7
Feb.	4	7
Mar.	12	14
Apr.	18	20
May	24	26
Jun.	29	29
Jul.	29	30
Aug.	29	30
Sep.	28	29
Oct.	23	24
Nov.	14	18
Dec.	9	11

Rate of Change ($\Delta T\uparrow$) Limits: WQS requires that "the rate of temperature change shall not exceed 1°C per hour". The same mixing zone and effluent flow used to calculate the monthly average limits and daily maximum limits will be used to calculate ΔT limits. The calculation is described by equation 3.23:

$$T_{e\text{-rate}}\ (°C/\ hr) = 1\ (°\ C/\ hr) * D \qquad (3.23)$$

Equation (3.23) can be used for large heat sources when there are continuous temperature monitoring data. For smaller heat sources, the following special conditions may be included in the wasteload allocations to implement the rate of change criteria.

Special conditions: cessation of thermal inputs to the receiving water by the discharge shall occur gradually so as to avoid fish mortality due to cold shock during the winter months (November through March). The basis for this requirement is to allow fish associated with the discharge-heated mixing zone for the discharge to acclimate to the decreasing temperature. The decrease in temperature at the end of the calculated mixing zone shall not exceed 1ºC per hour.

The water temperature in the Mississippi River in Zone II shall not exceed the maximum limits shown in Table 3.10 during more than 1 percent of the hours in the 12-month period ending

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with any month: The same mixing zone and AWW effluent flow will be used to calculate both $T_{e\text{-}max}$ and $T_{e\text{-}max1\%}$. The calculation is described by equation 3.24:

$$T_{e\text{-}max1\%} = Tr + (T_{II} \text{ or } T_{III} - Tr) * D \qquad (3.24)$$

The effluent temperature limit based on meeting the $\Delta T\uparrow$ criterion of 3ºC (5.4ºF) will be used as the monthly average limit and the T_{max} limit will be used as the daily maximum limit

IMPLEMENTATION OF 567 IAC 61.2(5)"A" AND IAC 61.2(5)"B"

According to 567 IAC61.2(5)"a" and IAC61.2(5)"b", during the period November 1 to March 31, for the purpose of applying the 3°C temperature increase criterion, the minimum protected receiving stream flow rate below such discharges may be increased to not more than three times the rate of flow of the discharge, where there is reasonable assurance that the discharge is of such constant temperature and flow rate and continuous duration as to not constitute a threat of heat cessation and not cause the receiving stream temperature to vary more than 3°C per day. This is implemented as follows.

1. If there is a reasonable assurance that the discharge is of such constant temperature and flow rate and continuous duration, when the receiving stream flow is less than 2x the discharge flow a stream flow of two times the discharge flow rate in lieu of the $Q_{7\text{-}10}$ will be used in the above formulas.
2. This procedure applies only when calculating temperature limits for discharges into interior warm water streams and does not apply to discharges to cold water streams or the Mississippi or Missouri Rivers.

316(a) DEMONSTRATIONS

Section 316(a) of the Federal Water Pollution Control Act provides that a discharger can be granted an alternate thermal effluent limitation if the discharger can satisfactorily demonstrate that the effluent limits calculated based on water quality standards are more stringent than necessary to protect a balanced and indigenous community of aquatic organisms in the receiving waterbody. A Section 316(a) demonstration generally requires comprehensive studies which include an evaluation of historical stream and effluent data, characterization of resident species of fish and shellfish populations and predictive impact modeling. A discharger with an interest in possible alternate effluent limits based on section 316(a) should consult Implementation Guidance Evaluation and Process Thermal Discharge, (316(a)) Federal Clean Water Act, USEPA, 1974 and must contact the department for approval prior to undertaking any studies.

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3.9 pH

The pH standard applies to both Class A and Class B designated waters and it is described in IAC 61.3(3)"a"(2) and 61.3(3)"b"(2).

In wasteload allocations, pH criteria are met at and beyond the mixing zone (MZ). The allowed default MZ Dilution for pH is the same as other toxics listed in Table 1 of Chapter 61 – Water Quality Standards, which is 25% of the $Q_{7\text{-}10}$ flow for interior streams and 10% of the $Q_{7\text{-}10}$ flow for the Mississippi and the Missouri River. Site-specific MZ data may be provided in lieu of the default MZ values through either modeling or a field mixing zone study noted in this document.

The equations used to calculate the pH water quality based limits are shown below:

$$pH\ (WQS) = -\log\ \{(Q_e * 10^{-pHe} + Q_r*MZ*10^{-pHr})/(Q_e + Q_r*MZ)\} \qquad (3.25)$$

where:

- Q_e: effluent flow, AWW is used as the Q_e (cfs)
- Q_r: stream flow, annual $Q_{7\text{-}10}$ is used as the Q_r (cfs)
- pH_e: effluent pH, standard unit
- pH_r: stream flow pH, standard unit,
- MZ: mixing zone Dilution, dimensionless, between 0-1
- pH (WQS): pH criteria (6.5 to 9.0)

Rearrange Equation (3.25):

$$pH_e = -\log\ \{((Q_e + Q_r*MZ)10^{-pH(WQS)} - Q_r*MZ*10^{-pHr})/Q_e\} \qquad (3.26)$$

Equation (3.26) provides the allowed effluent pH values in order to meet the pH criteria in the receiving water of 6.5 to 9 standard units.

3.10 General Use Classification

61.3(1)"a" defines General Use Segments as intermittent watercourses and those watercourses which typically flow only for short periods of time following precipitation and whose channels are normally above the water table. These waters do not support a viable aquatic community during low flow and do not maintain pooled conditions during periods of no flow. The general use segments are to be protected for livestock and wildlife watering, aquatic life, noncontact recreation, crop irrigation, and industrial, agricultural, domestic and other incidental water withdrawal uses.

For aquatic life protection, acute criteria are applied to general use segments when the constituents have numeric criteria.

For livestock watering, the following cation and anion guideline values are applicable. The guideline values would be met at the boundary of the mixing zone for both general use segments and the designated use segments.

Table 3.11. Recommended Water Quality Guidelines for Protecting Defined Uses

Ions	Recommended Guidelines Values* (mg/l)
Calcium	1000
Chloride	1500
Magnesium	800
Sodium	800
Sulfate	2000
Nitrate+Nitrite-N	100

* Based on the guidelines for livestock watering.

3.11 Mixing Zone Procedures

Objective

The objective of this procedure is to provide guidance on the methods to be used in considering a mixing zone (MZ) while determining applicable effluent limitations for a wastewater discharge.

Background

Chapter 61.2(4) defines the MZ of a wastewater discharge. The MZ is located downstream of the zone of initial dilution (ZID). The standards contain specific criteria and considerations, which are to be used in determining the extent and nature of a MZ. The most restrictive of the provisions establishes the MZ dimensions and flow. The following is a summary of the key provisions of the standards, additional policies, and the sequence used in defining the regulatory MZ and ZID.

1. The maximum flow in the MZ for toxic parameters will be set as 25% of the 7QlO flow for interior streams, the Big Sioux River, the Des Moines River and 10% of the Mississippi and Missouri Rivers. The maximum flow in the MZ for ammonia is defined as a function of the dilution ratios.

2. The flow in the MZ will be restricted by the natural functions and influences of mixing, which limit how much water can mix with the discharge effectively. These influences

can be islands, semi-permanent sandbars, and manmade obstructions. For larger rivers the WLA calculations will use 25% of the portion of the flow in the main or side channel into which the facility discharges or the MZ travels, where that flow is separated from flow in the other channels of the river by sandbars or islands which have remained in place for more than three years. However, the maximum flow in the regulatory mixing zone should not be higher than the maximum flow defined in the above condition, number one.

3. The length of the MZ may not exceed the most restrictive of the following seven conditions:
   a. The distance to the juncture of two perennial streams.
   b. The distance to a public water supply intake.
   c. The distance to the upstream limits of a heavily used recreational area.
   d. The distance to the middle of a crossover point in a stream where the main current flows from one bank across to the opposite bank.
   e. The distance to another MZ.
   f. A distance of 2000 feet.
   g. The location where the MZ contained the percentages of stream flow noted in one and two above.

4. The length of and flow in the ZID for toxics may not exceed 10% of the MZ values.

The chronic criteria for toxics and ammonia nitrogen will be met at the boundary of the MZ. The acute criteria for toxics and ammonia nitrogen will be met at the boundary of the ZID. Although not specifically discussed in the standards, the effects of the Biological Oxygen Demand (BOD) are not expected to be observed until after the end of the regulatory MZ. This is because the movement of water through the MZ normally will occur faster than the biological uptake of oxygen due to the BOD.

These two zones will be determined in one of two manners, by actual field measurements at design low stream flow conditions or by use of a dispersion model such as the CORMIX model. It is the goal of the department to obtain all necessary information of these zones from the information submitted in a wastewater treatment facility's NPDES permit application. A field procedure protocol has been developed for a NPDES applicant to obtain actual field data. Unless site specific mixing zone data are submitted for consideration, default regulatory mixing zones defined in IAC 61 are used.

Special Limitations On Mixing Zone:

The following defines the conditions where a mixing zone might not be appropriate:

(1) Where drinking water contaminants are a concern, mixing zones shall not encroach on drinking water intakes;
(2) Mixing zones would be restricted such that they do not encroach on areas often used for harvesting of stationary species such as shellfish;

(3) Mixing zones and zones of initial dilution would not be appropriate for bioaccumulating pollutants, such as Mercury, Chlordane, PCB and Dieldrin;
(4) Mixing zones should not be located in critical habitat for threatened or endangered species;
(5) Mixing zones would not be appropriate where established mussel beds exist, for example, on the Mississippi River, mixing zones and initial dilutions should not be allowed to encroach known mussel beds;
(6) No mixing zones are allowed for State owned Lakes and Wetlands (which is consistent with IAC 61)
(7) No zone of initial dilution will be allowed in waters designated as Class B(CW), Cold Water (which is consistent with IAC61).
(8) For reservoirs (Coralville Lake, Lake Red Rock and Saylorville Lake) on streams designated as Class B waters and big pool conditions such as on the Mississippi River, the default mixing zone and zone of initial dilution will use 1% and 0.1% of the Q7-10 and Q1-10 of the stream flow for toxics; 1% and 0.1% of the Q30-10 and Q1-10 of the stream flow for ammonia nitrogen unless site-specific mixing zone data are available. Site specific mixing zone data through modeling or field study may be used in lieu of default mixing zone sizes.

Mixing Zone Study Guidance:

The following are the basic field data requirements for mixing zone (MZ) studies. This field data is to be provided by a National Pollutant Discharge Elimination System (NPDES) applicant for recalculation of the local MZ. The purpose of the recalculation is to more closely approximate the local MZ using site specific data instead of statewide data. Contact should be made with the Department's Water Resources Section staff prior to beginning any field study.

1. Stream Characteristics
It should be noted that the terms low flow and low stream flow are used in the following discussion. These terms are not synonymous with the design low flow. The facility can provide information on the actual mixing zone characteristics during low stream flow conditions to demonstrate that a different percentage of the low stream flow is mixing with the effluent than projected by the Department.

Stream surveys to gather mixing zone data should be collected as near to the design low flow as is normally feasible during the year. A mixing zone study should be performed at stream flows not exceeding 3 to 5 times the design low flow. Stream flow conditions closer to the design low flow are desirable for those locations where normal flows during the year approach the design low flow or where the flows are controlled by impoundments. Several different field efforts are being considered in obtaining the mixing zone information, Visible Assessment, Dye Injection – Visible Boundary, and Dye Injection – Fluorometric Measurements.

a. Visible Assessment: This procedure is a simple field documentation of the effluent's mixing with the stream under low stream flow conditions. Pictures, video, drawings, or a more detailed map along with some physical stream data should be provided to illustrate how the two waters (effluent and receiving stream) are combining. Typically, the effluent can be seen (foam, turbidity, or color differences) to mix with the stream. Some facilities have added dye to the effluent to facilitate the visible assessment. This approach should be adequate on a smaller, shallow stream. A letter authorizing the discharge of dye will be required from the Department before dye can be introduced into the stream.

The objective is to demonstrate whether or not the effluent flow is completely mixing with the stream within the allowed mixing zone length. With no additional documentation on the mixing characteristics in the zone of initial dilution, default of 5% design low flow will be used for the ZID in the WLA calculation.

- The visible assessment description should include the following items for a distance of 2000 feet downstream (unless other distance limitation is known to apply) and 200 feet upstream of the outfall:
  (1) Describe the stream bed materials: sand, fine or coarse gravel, mud, or rock.
  (2) Note pools and riffles and areas of uniform depths. Estimate length and number thereof and the rapidity of the variations (i.e. gradual, alternating occasionally, or alternating frequently).
  (3) Describe the amount of weed growth and snags in the stream in terms of negligible effects on the stream flow to severe effects on the stream flow.
  (4) Describe the amount of meandering within the 2000 feet distance.
  (5) Describe other features which might effect the MZ such as delta formation at the stream mouth, other discharges, perennial springs, etc.

- A description is needed of the outfall during a low stream flow period. This should include an indication of the discharge flow during the period being described, preferably with pictures. Describe such things as the size and configuration of splash pools, outfall height or depth, outfall diameter (if normally filled during discharging), and/or average velocity of flow exiting outfall when submerged.

- The Department encourages the submission of additional field data. This would include at least two cross sections of the stream at low flow, one at an upstream location and one at the anticipated MZ. Each cross section should include a minimum of 10 depth measurements (depths taken at least every two feet if stream width is less than 40 feet and at least every 5 feet if less than 100 feet, otherwise every 10 feet). Stream velocities should be provided if the dilution ratio is less than 3:1, one upstream, one at the anticipated mixing zone, and one spaced evenly downstream of the outfall and the MZ. If there are several pools and riffles, additional cross sections are needed to provide a more accurate indication of average depths.

b. Dye Injection – Visible Boundary Measurements: The objective of this procedure is to provide greater accuracy in characterizing the mixing of an effluent with the receiving stream by using a visible dye injected into the effluent. The following is a brief summary of the procedures that should be followed:

1. Lay out downstream station locations along shoreline at interval of 50', 100', 200', 500', 1000', 1500', and 2000' below the outfall.
2. Assemble boundary marking floats or stakes. Test stream depth for float line length and ability to wade.
3. Run short test of dye introduction into the effluent. The dye introduction is normally poured as a slug of dye into the effluent at the last manhole or at the outfall.
4. Run actual dye study and set out markers. Time of travel between stations may also be obtained, if desired.
5. Measure stream flow, (depth, velocity, cross section) at selected downstream sites and upstream of outfall. It is important to determine the amount of flow in the dye plume at both the MZ and ZID locations. Obtain effluent flow measurement at time of dye injection.
6. Prepare a report of the findings.

This will take a field crew of three people approximately two days to complete. The data assembly and preparation of the report will take several days. This is not a widely used type of study, but it is able to provide quantifiable data, particularly on larger waterbodies. The procedures may be modified if needed for specific stream conditions. Data results need not show 100% mixing. The key is to perform the study at or near design low stream flow conditions. Models are available to project the percentages of mixing obtained during field flow conditions to design low flow regime.

c. Dye Injection - Fluorometric Boundary Measurements: The objective of this procedure is to provide even greater accuracy in characterizing the mixing of an effluent with the receiving stream by using a florescence dye injected into the effluent. This is a rarely used approach as it is more staff intensive, but it has provided very quantifiable results.

This study is very similar to the Visible Dye effort noted above, however, the actual measurement of dye concentrations (or collection of water samples for later analysis) will be made at various locations in the mixing zone. The dye will be fed into the effluent at a constant rate/concentration over the duration of time required to collect all dye samples. The collection of dye samples (or measurement of concentrations) will be made across the stream from the shoreline until a point in the stream where no additional dye is expected. The same station locations will be used starting at the lower location and proceeding upstream. Stream flow measurements as noted above also will be required. This will take a field crew of three to four people approximately two to three days to complete. The data assembly, analysis, and preparation of the report will take several days.

2. Use of Mixing Zone Study Results

The Department will use the mixing zone study results to recalculate WQ-based permit limits. It is important to note that the level of accuracy is greatly improved by providing site specific data of the Mixing Zone (and ZID if applicable) while still ensuring that the WQS are met at any point along the mixing zone boundary. It is recommended that the Mixing Zone study be performed prior to NPDES Permit re-issuance. When it is not feasible to complete a Mixing Zone study prior to the permit re-issuance, the Mixing Zone may be an item of the compliance schedule. The Water Resources Section can provide the facility with preliminary WQ-Based permit limits to aid in evaluating the need for Mixing Zone study. It is recommended that contact be made with the Water Resources Section staff to discuss the scope of a mixing zone study and receive necessary variances if dye is to be injected into the stream.

3. Installation of a Diffuser

Several facilities have constructed an instream diffuser to disperse their effluent across a more significant portion of the stream. This is an artificial means to increase the mixing zone. Typically 75 - 80% of the low stream flow is passed across a diffuser. Several facilities have designed diffusers to force 100% of the low flow across the diffuser. Partially buried pipe with risers or rock encased perforated pipe are being used. Several permits may be required for this type of structure. No mixing zone study is needed for the use of a diffuser. However, a follow-up stream study will be required to demonstrate that the diffuser is working properly.

3.12 Determination of Stream Flow Velocity

Stream flow velocities are often needed for performing bacteria or total residual chlorine decay calculations. Site-specific velocity is always preferred. Sometimes the stream velocity can be estimated based on stream morphology. When site-specific velocity data are not available, the following default flow velocities are used in the WLA calculation:

a. 0.1 – 0.3 fps in general use streams
b. 0.5 fps in gravity discharge pipe or storm sewer
c. 1 – 2 fps for pressured pipe flows, such as pressured sewer outfall pipe.

3.13 Site Specific Data Collection

The permittee may choose to collect site-specific ambient and/or effluent water chemistry data in lieu of the statewide default values and submit the data for Department consideration. Wastewater treatment facilities are encouraged to plan ahead when considering any site-specific data gathering effort. Some of these efforts require seasonal data particularly collected during low stream flow conditions. Contact should be made with the Department's Water Resources Section staff prior to beginning any site-specific data collection. Both local and regional data may be collected and submitted to the Department.

a. Local Values: If the applicant desires that local values be used, they must supply a minimum of 2 years of water chemistry readings (hardness, sulfate and chloride) and sample at least once per month.

1) Background Water Chemistry: The applicant must submit a minimum of 2 years of grab samples for ambient background water chemistry and the samples must be collected at least once per month. More frequent monitoring in a shorter time period than 2 years may be allowed if the applicant can demonstrate that the monitoring data is representative of a typical year. Monitoring values should be obtained from upstream of the outfall at a representative location of the true upstream background condition of the discharge.

In cases for certain pollutants, seasonal water chemistry data may be required to catch the most critical conditions such as low stream flow and high temperature conditions. The sample plan must be able to catch the required critical.

2) Effluent Water Chemistry: For effluent water chemistry determinations, a 24-hour composite sample of the final effluent is required. For intermittent discharges, a 24-hour composite sampling may not be feasible; a representative grab sample is also acceptable.

b. Regional Background Values: Regional water chemistry data could be available that represents the upstream background conditions. For example:

(1) Another facility, at a reasonable distance upstream of the facility of interest, has collected background readings of water chemistry data (such as hardness, sulfate and chloride) that is representative of the background chemistry for the facility of interest;

(2) Ambient monitoring data are available within the same watershed (such as STORET data) that is representative of the upstream background conditions of the facility of interest;

The ambient data will be used instead of the statewide default water chemistry values.

3.14 BOD and DO WLAs

The calculation of a wasteload allocation for conventional pollutants will consider the instream dissolved oxygen impacts of carbonaceous biochemical oxygen demand (CBOD), ammonia nitrogen (NH_3-N), and any other oxygen demanding materials. The wasteload allocation for NH_3-N and other oxygen demanding materials is also addressed in separate sections, as these pollutants can also cause acute and chronic toxicity.

The wasteload allocation of the oxygen demanding pollutants are determined directly from the results of water quality models which account for the fates of the pollutants as they move down the receiving stream.

A first screening procedure may be applied before water quality modeling is performed. The use of a screening procedure is intended to provide a quick method to determine if a CBOD discharge of standard secondary or BPT/BAT[1] from the treatment facility is causing a water quality violation. This step may not be necessary if the treatment facility is known to be causing a water quality violation, or if the facility requires advanced treatment. This calculation, as with the use of the water quality models, will be performed using the design low stream flow (7Q10), the treatment facilities design dry and wet weather flow (if applicable), the appropriate standard secondary CBOD5, and assumed ammonia levels. With the various alternative treatment limits allowed in the definition of standard secondary, the specific permitted CBOD5 levels for the selected (or expected) type of treatment must be used in the screening calculations. This approach uses a conservative assimilation rate of CBOD5 (20 lbs/d/cfs) which has been derived from past modeling results.

Screening Calculation:

a. Available Stream Capacity

Staff will calculate the available stream capacity for CBOD5 below the discharger in question by the following relationships. CBODL is the stream capacity (or loading) carbonaceous BOD5 in pounds per day.

For CBOD5

$$(Qu + Qd)\ 20\ \text{lbs/d/cfs} = CBOD_L \qquad (3.27)$$

where:

Qu = Critical stream flow, cfs

Qd = Dry weather design discharge flow, cfs

$CBOD_L$ = Stream capacity carbonaceous BOD5, lbs/day

Treatment Facility Loading

The loading from the treatment facility at its specific standard secondary level is given by the following equation.

For BOD5

$$(CBOD5)\ (8.34)(Qd) = CBODe \qquad (3.28)$$

where:

CBOD5 = Technology or standard secondary CBOD5, mg/L

[1] BPT = Best Practical Treatment are EPA derived minimum treatment levels that are required for both municipal and industrial wastewater treatment facilities. BAT = Best Available Treatment are EPA derived levels for industry.

Qd = Dry weather discharge flow, mgd
CBODe = Carbonaceous BOD5 in the effluent, lbs/day
8.34 = Conversion factor

Stream Capacity vs. Effluent Loading

If the stream CBOD5 capacity ($CBOD_L$) above is larger than the technology or standard secondary CBOD5 (CBODe), the stream is termed effluent limited for CBOD and no additional modeling is required. The effluent limitation for CBOD5 will be the level set for standard secondary or the technology level.

If according to the above comparison the stream is not effluent limited, the stream should be modeled using the Streeter-Phelps Model or QUALIIK model. However, unusual factors or stream conditions might warrant undertaking the next calculation step even if the stream is effluent limited. These unusual conditions might include: several dischargers within close proximity, discharge of large algal concentrations, discharge of elevated ammonia nitrogen levels, and loadings to the stream at or near stream capacity.

Water Quality Modeling:

The ability of a stream to maintain an acceptable dissolved oxygen (DO) concentration is an important consideration in determining its capacity to assimilate wastewater discharges. DO is used in the microbial oxidation of organic and certain inorganic matter present in wastewater. Oxygen supplied principally by reaeration from the atmosphere will replace any DO lost through oxidation processes. If, however, the rate of oxygen use exceeds the rate of reaeration, the DO concentration may decrease below minimum allowable standards.

Water quality models are useful tools that can be used to predict the effects of point and nonpoint sources on dissolved oxygen levels in a waterbody. Water quality modeling is an attempt to relate specific water quality conditions to natural processes using mathematical relationships. A water quality model usually consists of a set of mathematical expressions relating one or more water quality parameters to one or more natural processes. Water quality models are most often used to predict how changes in a specific process or processes will change a specific water quality parameter or parameters.

Water quality models vary in complexity from simple relationships which attempt modeling a few processes under specific conditions to very complex relationships which attempt to model many processes under a wide range of conditions. The simpler models are usually much easier to use and require only limited information about the system being modeled but are also limited in their applicability. Steady-state models in which certain relationships are assumed to be independent of time fall into this category. More complex models may relate many natural processes to several water quality parameters on a time-dependent basis. These models are usually harder to apply and require extensive information about the system being modeled, but also have a broader range of applicability. Dynamic models fall into this category.

To predict the variation in DO, as well as ammonia concentration in streams, a simplified Excel spreadsheet implementing the modified Streeter-Phelps DO Sag equation and a more complex mathematical model such as QUALIIK have been used in Iowa. Input data for the models is developed from existing technical information and site specific field investigations of selected streams. When sufficient data is not available, conservative assumptions are applied until site specific information becomes available.

THEORY AND METHODOLOGY

Modeling Theory

Dissolved oxygen (DO) concentrations in streams are controlled by many factors including atmospheric reaeration, biochemical oxygen demands (carbonaceous and nitrogenous), algal photosynthesis and respiration, benthal oxygen demands, temperature, and the physical characteristics of the stream. Many of these factors are difficult, if not impossible, to accurately assess. As a result of this difficulty, limitations on the use of these controlling factors are discussed below.

Photosynthesis can produce large quantities of oxygen during the day if algae are present in the stream. Conversely, at night, algal respiration creates an oxygen demand. Both photosynthesis and respiration are included in the QUALIIK model. Phytoplankton photosynthesis is a function of temperature, nutrients, and light. Phytoplankton respiration is represented as a first-order rate that is attenuated at low oxygen concentration. Benthal oxygen demands result from anaerobic decomposition of settled organic material at the bottom of the stream. These reactions release carbonaceous and nitrogenous organic materials that create biochemical oxygen demands. The inclusion of benthal oxygen demands in the QUALIIK model requires extensive field surveys to determine the real extent of sludge deposits within a stream and coefficients that describe the release into the water. In most instances no data is available to accurately describe sludge deposition areas. Benthal oxygen demands are usually not included in the Excel spreadsheet model. QUALIIK has the sediment oxygen demand (SOD) component. The sediment-water fluxes of dissolved oxygen and nutrients are simulated internally rather than prescribed. That is, SOD and nutrient fluxes are simulated as a function of settling particulate organic matter, reactions within the sediments and the concentrations of soluble forms in the overlying water. The SOD simulation is bested used when sufficient field data are available to calibrate and verify the rate constants. If field data are not available, default rate constant values can be used.

Nitrogenous BOD is due to the oxidation of ammonia to nitrates by certain species of bacteria. This oxidation process is called nitrification. Nitrification is a two-step process whereby a specific bacterial species oxidizes ammonia to nitrite and a different bacterial species oxidizes the nitrite to nitrate. Theoretically, approximately 4.5 mg/L of oxygen are required to oxidize 1.0 mg/L of ammonia (expressed as nitrogen) to nitrate. This theoretical value may conservatively over estimate the oxygen demand of nitrification as the nitrifiers obtain oxygen from inorganic carbon sources during combined energy and synthesis reactions. Actual values obtained have varied between 3.8 and 4.5 mg/L of oxygen per mg/L of ammonia nitrogen (NH_3-N). The spreadsheet implementing the Streeter-Phelps equation uses 4.33 as the ratio of nitrogenous BOD to NH_3-N. Assuming secondary wastewater treatment plant effluents contain NH_3-N levels of 10 mg/L during summer operations and 15 mg/L during winter periods, the equivalent nitrogenous BOD (should all the ammonia be converted to nitrates) is approximately 40-46 mg/L (summer) and 62-68 mg/L (winter).

Modified Streeter-Phelps DO Sag Model

Dissolved Oxygen Deficit Equation

The spreadsheet uses the modified Streeter-Phelps equation to predict DO deficit within the stream. This approach recognizes carbonaceous and nitrogenous BOD, atmospheric reaeration, and initial DO deficit. The effects of photosynthesis and benthal oxygen demands are usually not specifically considered unless site specific data are available. The Streeter-Phelps equation that is implemented in the spreadsheet is as follows:

$$D(t) = \frac{K_1 L_o}{K_2 - K_1}(e^{-K_1(t)} - e^{-K_2(t)}) + \frac{K_N N_o}{K_2 - K_n}(e^{-K_n(t-t_0))} - e^{-K_2(t-t_0))}) + D_o e^{-K_2(t)} + \frac{(R-P)}{K_2}(1 - e^{-K_2(t)}) + \frac{SOD}{K_2 H}(1 - e^{-k_2 t})$$

where:

D(t) = DO deficit at time t, mg/L
Do = Initial DO deficit, mg/l
Lo = Initial ultimate carbonaceous BOD concentration, mg/L
No = Initial ultimate nitrogenous BOD concentration, mg/L
K1 = Carbonaceous deoxygenation rate constant, base e, day -1
KN = Nitrogenous deoxygenation rate constant, base e, day –1
K2 = Reaeration rate constant, base e, day -1
t = Time of travel through reach, day
SOD=sediment oxygen demand, g O_2/ft^2/day
H=average stream depth, ft
R = Algal respiration oxygen utilization, mg/L/day
P = Photosynthetic oxygen production, mg/L/day

t_0 = nitrogenous lag time, days; when a wastewater contains both carbonaceous and nitrogenous oxygen demand, there is usually a time lag before the onset of nitrogenous

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oxygen demand. The value of t_0 may be experimentally determined where effluent or stream field measurements are practicable. In the case of well nitrified effluents, the value of t_0 may generally be considered to be less than 1 day. Note that for t less than t_0 the nitrogenous term does not enter into the calculation of D(t).

Since the initial ultimate nitrogenous BOD is normally not readily available, it is estimated based on the equation as follows:

$$N_0 = 4.33 N_{n0}$$

Where:

N_{no}, initial ammonia nitrogen concentration, mg/L

In this equation, the rates of oxygen utilization due to carbonaceous and nitrogenous BOD are expressed as first order reaction rates. This is an accepted procedure for the carbonaceous demand, but represents a simplification for the nitrogenous demand. The other traditional Streeter-Phelps components (Streeter, 1925) remain unchanged.

The ultimate carbonaceous and nitrogenous BOD concentrations as a function of time (t) are calculated as follows:

$$L(t) = L_o e^{-K_1(t)}$$

$$N(t) = N_o e^{-K_N(t)}$$

where:
L(t) = Ultimate carbonaceous BOD at time, t, mg/L
N(t) = Ultimate nitrogenous BOD at time, t, mg/L

Since nitrification is a two-step process, many researchers have proposed that it is a second order reaction. However, most water quality models assume that it is a first order reaction for the ease of programming and usage.

Nitrifying bacteria are generally present in relatively small numbers in untreated wastewaters. The growth rate at 20°C is such that the organisms do not exert an appreciable oxygen demand until about eight to ten days have elapsed in laboratory situations. This lag period, however, may be reduced or eliminated in a stream due to a number of reasons including the following: the discharge of large amounts of secondary effluent containing seed organisms, and nitrifier population buildup on the stream's wetted perimeter. In biological treatment systems, substantial nitrification can take place with a resultant build-up of nitrifying organisms. These nitrifying bacteria can immediately begin to oxidize the ammonia present and exert a significant oxygen demand in a stream below the outfall.

It is known that the biological nitrification process is generally more sensitive to environmental conditions than carbonaceous decomposition. The optimal temperature range for growth and reproduction of nitrifying bacteria is 26° to 30° C. It is generally concluded that the nitrogenous BOD will assume greatest importance in small streams which receive relatively large volumes of secondary wastewater effluents during the low flow, warm weather periods of the year (August and September). These conditions were used for the low flow determination of allowable effluent characteristics during summer periods. During winter low flow periods (January and February), nitrification will have limited influence upon the oxygen demand due to the intolerance of nitrifying bacteria to low temperatures. During analysis of winter low flow conditions, limited nitrification was observed.

Rate Constant Determination

Deoxygenation Rate Constants

The carbonaceous deoxygenation rate constant (K1) for most streams will vary from 0.1 to 0.5 per day (base e, 20 °C). Early work by Streeter and Phelps (Streeter, 1925) determined an average value for the Ohio River of 0.23/day at 20°C (0.1/day, base 10). This value has been accepted and commonly used with reasonable results.

Specific deoxygenation rates for selected Iowa stream segments have been determined from stream surveys performed since 1977. These specific rates showed wide variations within each stream segment and among various streams. Thus, the carbonaceous deoxygenation rate of 0.2/day at 20°C is still used as an initial starting point in calibration/verification efforts.

Information on nitrogenous deoxygenation rates is extremely limited; however, available information indicates that nitrification rates (when active nitrification does occur) are somewhat greater than carbonaceous oxidation rates. Therefore, the nitrogenous deoxygenation rate (KN) (0.3/day at 20°C was selected) is used as input data unless calibration/verification efforts provide a more reliable value. Reaeration Rate Constant

Five reaeration rate constant estimation methods were provided in the spreadsheet. Each reaeration model is more accurate than others in certain circumstances. The spreadsheet gives the users the options to choose the most suitable reaeration model for a specific case.

1). The Tsivoglou & Neal (1976) model

This formulation is based on the premise that the reaeration capacity of nontidal fresh water streams is directly related to the energy expended by the flowing water, which in turn is directly related to the change in water surface elevation.

$$K_2 = C \times S \times V$$

Where :

K2 = Reaeration rate constant, base e, day -1

S = Stream bed slop, m/m

V = Stream velocity, m/s

C= Constant, 31,183 for stream flow between 1 cfs to 15 cfs (0.0283 to 0.4247 cms), and 15,308 for stream flow between 15 to 3,000 cfs (0.4247 to 84.95 cms)

2). Owens et al. (1964) model

This formulation is also called Owen-Gibbs and applies for stream velocity in the range of 0.1 to 5.0 fps and stream water depth in the range of 0.4 foot to 11 feet.

$$K_2 = 5.32 \times \frac{V^{0.67}}{H^{1.85}}$$

Where :

H = Stream water depth, m

3). O'Connor & Dobbins (1958) model

This formulation is more accurate when applied to moderately deep to deep channels. The suitable water channel depth should be in the range of 1 foot to 30 feet with a velocity range from 0.5 fps to 1.6 fps.

$$K_2 = 3.93 \times \frac{V^{0.5}}{H^{1.5}}$$

4). USGS (Pool-riffle) Melching and Flores (1999) model

Two formulations are included in this model, each is suitable for a certain stream flow range. When stream flow is less than 0.556 cms (or 19.64 cfs), the formulation is,

$$K_2 = 517 \times \frac{(VS)^{0.524}}{Q^{0.242}}$$

Where :

Q = Stream flow, m^3/s

When stream flow is greater than 0.556 cms (or 19.64 cfs), the formulation is,

$$K_2 = 596 \times \frac{(VS)^{0.528}}{Q^{0.136}}$$

5). USGS (Channel-control) Melching and Flores (1999) model

Similarly two formulations are included in this model, each is suitable for a certain stream flow range. When stream flow is less than 0.556 cms (or 19.64 cfs), the formulation is,

$$K_2 = 88 \times \frac{(VS)^{0.313}}{H^{0.353}}$$

When stream flow is greater than 0.556 cms (or 19.64 cfs), the formulation is,

$$K_2 = 142 \times \frac{(VS)^{0.333}}{H^{0.66} \times B_t^{0.243}}$$

Where :

Bt = the top width of the channel, m

Temperature Corrections

Temperature corrections for the carbonaceous and nitrogenous deoxygenation rate constants and the reaeration rate constants are performed within the computer model. The following equations define the specific temperature corrections used in the program:

K1(T) = K1(20) (1.047 (T-20))

K2(T) = K2(20) (1.024 (T-20))

KN(T) = KN(20) (1.083 (T-20))

where: T = Water temperature, °C

The temperature corrections for the three rate constants are widely accepted formulations.

The principal factor affecting the solubility of oxygen is the water temperature. DO saturation values at various temperatures are calculated based on Standard Methods for the Examination of Water and Wastewater, 21th Edition:

$$C_S = \exp(-139.34411 + \frac{1.575701 \times 10^5}{T + 273.15} - \frac{6.642308 \times 10^7}{(T + 273.15)^2} + \frac{1.243800 \times 10^{10}}{(T + 273.15)^3} - \frac{8.621949 \times 10^{11}}{(T + 273.15)^4})$$

where:

T = Water temperature, °C

Cs = Saturation value for oxygen at temperature, T, at standard Pressure of 1 atm, mg/L

Stream Velocity Calculations

Stream velocities are important in determining reaeration rates and the downstream dispersion of pollutants. The spreadsheet calculates velocity based on either a variation of the Manning's Formula for open channel flow or the Leopold-Maddox predictive equation.

The reality is that the field data of a lot of small streams are not available. Assumed velocity values between 0.1-0.3 fps in small streams can be used in the spreadsheet.

Manning's Formula

Each element in a particular reach can be idealized as a trapezoidal channel. Under conditions of steady flow, the Manning equation can be used to express the relationship between flow and depth as:

$$V = \frac{1.49 R^{2/3} S^{1/2}}{n}$$

where:

V = Velocity, fps

R = Hydraulic radius, ft

S = Channel Slope ft/ft

n = Roughness coefficient

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For a river or stream with a width much greater than its depth, the value of R is approximately equal to the mean depth. If both sides of the equation are multiplied by the cross-sectional area (width)(mean depth), the following equation results:

$$Q = \frac{1.49}{n} W H^{5/3} S^{1/2}$$

where:

H = Mean river depth, ft

Q = Discharge, cfs

W = Water surface width, ft

S = Slope ft/ft

n = Roughness coefficient

All variables except for "H" are input values. Internally, the program solves the above equation for H, then calculates the velocity V by:

$$V = Q/A = Q/WH$$

River slopes were obtained from existing stream profiles when available, but usually were taken from USGS topographic maps. Slopes obtained from USGS maps are rather generalized, and more accurate river profiles would greatly improve the accuracy of velocity determinations.

River widths were estimated from information obtained from field observations, flow, and cross-sectional data at each USGS gauging station.

The following table shows the roughness coefficient for various open channel surfaces. The value of 0.035 is being used on Iowa streams unless the physical characteristics of the stream are more accurately reproduced by another value.

Table 3.12. The Manning roughness coefficient for various open channel surfaces (from Chow et al. 1988).

MATERIAL	n
Man-made channels	

Concrete	0.012
Gravel bottom with sides:	
Concrete	0.020
mortared stone	0.023
Riprap	0.033
Natural stream channels	
Clean, straight	0.025-0.04
Clean, winding and some weeds	0.03-0.05
Weeds and pools, winding	0.05
Mountain streams with boulders	0.04-0.10
Heavy brush, timber	0.05-0.20

Manning's n typically varies with flow and depth. As the depth decreases at low flow, the relative roughness usually increases. Typical published values of Manning's n, which range from about 0.015 for smooth channels to about 0.15 for rough natural channels, are representative of conditions when the flow is at the bankfull capacity. Critical conditions of depth for evaluating water quality are generally much less than bankfull depth, and the relative roughness may be much higher.

In developing the particular model run for a stream segment, depth and velocity data from stream gauging stations or from field surveys are used to extrapolate depth and velocity at other points along the segment. The extrapolation is a rough approximation;

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however, it is reasonably close over the average length of a stream. When available, the uses of field investigations to determine actual stream velocities and depths at many selected stream sites in the modeled segment have improved the accuracy of the model.

The Manning's equation is used where little historical flow and velocity information exists in the stream segment. If flows and velocities are measured during a calibration sampling event, the roughness coefficient "n" can be calibrated. However, in most instances, more reliable flow velocity relationships can be modeled by using the power equations.

Power Equations

Power equations (sometimes called Leopold-Maddox relationships) can be used to relate mean velocity and depth to flow for the elements in a reach,

$$V = aQ^b$$

$$H = \alpha Q^\beta$$

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where a, b, α and β are empirical coefficients that are determined from velocity-discharge and stage-discharge rating curves, respectively. The values of velocity and depth can then be employed to determine the cross-sectional area and width by

$$A_c = \frac{Q}{V}$$

$$W = \frac{A_c}{H}$$

where:

V = Stream velocity, ft/sec

Q = Discharge, cfs

H = Mean river depth, ft

W = Water surface width, ft

Ac= Cross sectional area, ft2

It is significant to point out that the empirical constants a and b apply to a specific stream cross section. The value of “b” represents the slope of a logarithmic plot of velocity versus discharge. The value of “a” represents the velocity at a unit discharge. The exponents b and β typically take on values listed in Table 3.13. Note that the sum of b and β must be less than or equal to 1. If this is not the case, the width will decrease with increasing flow. If their sum equals 1, the channel is rectangular.

Table 3.13. Typical values for the exponents of rating curves used to determine velocity and depth from flow (Barnwell et al. 1989).

Equation	Exponent	Typical value	Range
$V = aQ^b$	b	0.43	0.4~0.6
$H = \alpha Q^\beta$	□	0.45	0.3~0.5

The power equations have been used in many studies and have been found to produce reliable results when the empirical constants are properly evaluated. However, its use is limited to streams for which historical data are not available for determining representative values for the empirical constants. A regression analysis can be performed on several sets of velocity-discharge data to determine the empirical constants. The data selected for use in the analysis corresponds to low stream flow conditions since the use of elevated stream flow data may bias the results.

Reaches of uniform cross section, slope, and roughness parameters rarely characterize stream systems, the empirical constants are determined for several representative cross

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sections of each stream system to be modeled. The same values of the empirical constants usually do not apply to all reaches along a stream segment unless field measured data indicates otherwise. Velocity and discharge values can also be obtained from the USGS gauging station data or from stream surveys.

QUALIIK Model

QUALIIK is a river and stream water quality model that is intended to represent a modernized version of the QUAL2E model (Brown and Barnwell 1987).

QUALIIK is similar to QUAL2E in the following respects:

QUALIIK is similar to QUAL2E in the following respects:

- One dimensional. The channel is well-mixed vertically and laterally.
- Steady state hydraulics. Non-uniform, steady flow is simulated.
- Diurnal heat budget. The heat budget and temperature are simulated as a fuction of meteorology on a diurnal time scale.
- Diurnal water-quality kinetics. All water quality variables are simulated on a diurnal time scale.
- Heat and mass inputs. Point and non-point loads and abstractions are simulated.

The QUAL2K framework includes the following new elements:

- Software Environment and Interface. Q2K is implemented within the Microsoft Windows environment. Numerical computations are programmed in Fortran 90. Excel is used as the graphical user interface. All interface operations are programmed in the Microsoft Office macro language: Visual Basic for Applications (VBA).
- Model segmentation. QUAL2E segments the system into river reaches comprised of equally spaced elements. QUAL2K also divides the system into reaches and elements. However, in contrast to QUAL2E, the element size for Q2K can vary from reach to reach. In addition, multiple loadings and withdrawals can be input to any element.
- Carbonaceous BOD speciation. QUAL2K uses two forms of carbonaceous BOD to represent organic carbon. These forms are a slowly oxidizing form (slow CBOD) and a rapidly oxidizing form (fast CBOD).
- Anoxia. QUAL2K accommodates anoxia by reducing oxidation reactions to zero at low oxygen levels. In addition, denitrification is modeled as a first-order reaction that becomes pronounced at low oxygen concentrations.
- Sediment-water interactions. Sediment-water fluxes of dissolved oxygen and nutrients can be simulated internally rather than being prescribed. That is, oxygen (SOD) and nutrient fluxes are simulated as a function of settling particulate organic matter, reactions within the sediments, and the concentrations of soluble forms in the overlying waters.

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- Bottom algae. The model explicitly simulates attached bottom algae. These algae have variable stoichiometry.
- Light extinction. Light extinction is calculated as a function of algae, detritus and inorganic solids.
- pH. Both alkalinity and total inorganic carbon are simulated. The river's pH is then computed based on these two quantities.
- Pathogens. A generic pathogen is simulated. Pathogen removal is determined as a function of temperature, light, and settling.
- Reach specific kinetic parameters. QUAL2K allows you to specify many of the kinetic parameters on a reach-specific basis.
- Weirs and waterfalls. The hydraulics of weirs as well as the effect of weirs and waterfalls on gas transfer are explicitly included.

A detailed documentation and User's Manual for QUALIIK water quality model can be obtained from the EPA website. The User's Manual provides a documentation of the theoretical aspects of the model as well as a description of the model input and data requirements, which are not reproduced in this document.

Specific input sequences and formats are presented in the User's Manual. Detailed procedures for calibrating the rate constants to specific stream conditions are also presented in the User's Manual. While running the program for a specific stream or for calibrating a segment, the suggested ranges for reaction coefficients are presented in Table 3.14. These values serve as a guide for a run of the QUALIIK program.

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TABLE 3.14

RECOMMENDED RANGES FOR REACTION COEFFICIENTS

FOR QUAL-IIK

DESCRIPTION	UNITS	RANGE OF VALUES
Ratio of chlorophyll-a to algae biomass	ug Chl-a/Mg A	10 - 100
Fraction of algae biomass that is nitrogen	Mg N/Mg A	0.07 – 0.09
Fraction of algae biomass that is phosphorus	Mg P/Mg A	0.01 – 0.02
O_2 Production per unit of algal growth	Mg O/Mg A	1.4 – 1.8
O_2 Uptake per unit of algae respired	Mg O/Mg A	1.6 – 2.3
O_2 Uptake per unit of NH3 oxidation	Mg O/Mg N	3.0 – 4.0
O_2 Uptake per unit of NO2 oxidation	Mg O/Mg N	1.0 – 1.14
Rate constant for the biological oxidation of NH_3 to NO_2	1/Day	0.10 – 1.00
Rate constant for the biological oxidation of NO_2 to NO_3	1/Day	0.20 – 2.00
Rate constant for the hydrolysis of organic-N to ammonia	1/Day	0.02 – 0.4
Dissolved phosphorus removal rate	1/Day	0.02 – 0.4
Organic phosphorus settling rate	1/Day	0.001 – 0.10
Algal settling rate	ft/Day	0.5 – 6.0
Benthos source rate for phosphorus	Mg P/day-ft	Highly Variable

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Benthos source rate for NH3	Mg N/day-ft	Highly Variable
Organic P decay rate	1/Day	0.1 – 0.7
Carbonaceous deoxygeneration rate constant	1/Day	0.02 – 3.4
Reaeration rate constant	1/Day	0.0 - 100

RECOMMENDED RANGES FOR REACTION COEFFICIENTS

FOR QUALIIK

- Continued -

DESCRIPTION	UNITS	RANGE OF VALUES
Rate of loss of CBOD due to settling	1/Day	-0.36 to 0.36
Benthic oxygen uptake	Mg O/day-ft	Highly Variable
Coliform die-off rate	1/Day	0.5 – 4.0
Maximum algal growth rate	1/Day	1.0 – 3.0
Algal death rate	1/Day	0.024 – 0.24
Preferential NH_3 uptake factor	--------	0.0 – 0.9
Algal N to organic N decay rate	1/Day	0.11
Algal respiration rate	1/Day	0.05 – 0.5
Michaelis-Menton half-saturation constant for light	Langleys/min	0.02 – 0.10
Michaelis-Menton half-saturation constant for nitrogen	mg/L	0.01 – 0.20

Michaelis-Menton half-saturation constant for phosphorus	mg/L	0.01 – 0.05
Non-algal light extinction coefficient	1/ft	Variable
Algal light extinction coefficient	(1/ft)/(ug Chl-a/L)	0.005 – 0.02

MODELING DATA SOURCES

The bulk of the work in stream water quality modeling is the collection and interpretation of all available data describing the stream system to be modeled. This section describes procedures and data sources that may be used in stream modeling for wasteload allocations.

Wastewater Discharges

The required data for each discharger consists of effluent flow rates and effluent characteristics such as Biochemical Oxygen Demand (BOD), ammonia nitrogen (NH_3-N), Dissolved Oxygen (DO) concentrations, and temperature. The specific location and characteristics of some smaller wastewater discharges are often unknown and are determined from field investigations or during special stream surveys. Most wastewater discharge information is available in the Departmental files.

River Miles

The first step in modeling a river system is determining the locations of all tributaries, wastewater dischargers, dams and other critical points along the river. The total length of the main channel of the river to be modeled must be established and river miles need to be located such that the location of tributaries, etc., can be determined. The best maps to start with are USGS topographic maps. These consist of section maps (scale: 1:250,000) and quadrangle maps (scale: 1:24,000). Other maps such as state and county road maps can also be used to supplement the USGS maps.

Field Reconnaissance

The following data can be collected during special stream surveys:

1. The precise location of wastewater discharges.
2. The location, condition, height, and type of dams and the nature and approximate length of the pool created by the dam.
3. Approximate river widths at bridge crossings.
4. Approximate shape of channel cross sections.

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5. Channel characteristics that will aid in determining the channel roughness coefficients.

The special stream survey should be performed, if possible, during flow conditions that represent the flows used in the modeling effort. Stream discharge information during stream surveys may be verified from data obtained from the USGS. The stream flow observed during stream surveys is often greater than the 7Q10. Data such as river widths need to be extrapolated downward to represent 7Q10 conditions. Shapes of channel cross sections are an aid in this determination.

River Channel Slopes

After river miles and locations are established, the next step is the determination of river channel slopes. During low flow conditions it can be assumed that river channel slopes are essentially the same as the slope of the water surface. Channel profiles can be used as representative of water surface slopes. In some cases, profiles of the river have already been determined. The U.S. Army Corps of Engineers usually does this as part of the work conducted prior to proposal or construction of flood control reservoirs. Without accurate profiles, river slopes can be determined from USGS contour maps by locating the points where contour lines cross the river. Stream slopes that are calculated from contour maps only represent an average value over the distance of the river between contour intervals. USGS quadrangle maps (if available) are a more reliable source of slope data. A GIS elevation coverage can also be used to obtain the stream slopes. Often, these are the only sources available and are the best method of slope determination without an extensive field survey.

River Widths and Roughness Coefficients

River widths and roughness coefficients can be estimated during the field reconnaissance. Roughness coefficients can also be estimated from Table 16.1 above.

The variation of river widths with discharges can often be determined from data at USGS gauging stations. The USGS periodically calibrates each gauge. The results from these calibrations are available on USGS website and include widths, cross-sectional area, mean velocities, and discharges. Reasonably accurate estimations of river widths at the desired discharge can usually be made with this gauging station information along the river widths measured during special stream surveys.

Stream Flow

In the determination of flow conditions throughout the river system to be modeled, all available data from USGS flow measuring stations as well as flow rates from all of the wastewater discharges must be obtained. River flows need to be allocated among tributary, groundwater, and wastewater inflow sources. The design low flow is used as the modeling basis, and is

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determined from a statistical analysis of the flow records at each of the gauging stations in the river system. Design low flows have already been determined for partial and continuous gauging stations (i.e. Statistical Summaries of Selected Iowa Stream flow Data Through September 1996" by USGS and Iowa Natural Resources Council, Annual and Seasonal Low-Flow Characteristics of Iowa Streams, Bulletin No. 13, 1979). The design low flows at gauging stations must then be allocated to tributaries based on drainage areas. Tributary drainage areas may be available from existing publications (i.e. Larimer, O.J., Drainage Areas of Iowa Streams, Iowa Highway Research Bulletin No. 7, 1957) or they can be determined from USGS contour maps. The Department staff uses a GIS tool that estimates the stream critical low flows using drainage area, critical low flows at a nearby USGS gaging station and USGS Plate 4 that includes the estimated stream flow per square miles for different regions.

A summation of tributary inflows and wastewater discharges often is less than the gauged flow. The difference is usually distributed along the main channel of the river as a uniform inflow in terms of cfs per mile of river reach length. If the gauged flow is less than the summation of tributary and wastewater inflows then it is possible to allot a uniform outflow from the main river channel.

Tributary and Groundwater Quality

Values for BOD, NH_3-N, and DO of tributary and groundwater inflow are required for stream modeling. Often, a main tributary to the stream being modeled has also been modeled. In this case, the water quality of the tributary just before discharge into the main stream (as determined by the model) is used. If the tributary is small and has several wastewater discharges, hand calculations can be done to determine its water quality just before entering the main stream.

If the tributary is free of continuous discharging wastewater facilities, water quality has been assumed to be good. The tributary water quality input values are: ultimate BOD – 6 mg/L; NH_3-N concentrations – statewide background concentrations and DO at 6 mg/L.

Groundwater is also noted to be of high quality. The model input values for groundwater are ultimate BOD of 6 mg/L and NH3-N at 0 mg/L. Groundwater DO's may be quite low depending on how it enters the stream. If it is subsurface flow, DO may be close to zero. A groundwater DO of 2 mg/L is used in wasteload allocation (WLA) work in Iowa.

Rate Constants

The reaeration rate constant (K2) is usually determined from one of many available predictive formulas shown in the previous section. The document titled "Rates, Constants, and Kinetics Formulations in Surface Water Quality Modeling – EPA/600/3-85/040, June 1985" can be a good source for obtaining the initial values for rate constants.

Carbonaceous and nitrogenous deoxygenation rate constants are best determined experimentally for a specific wastewater effluent and/or calibrated for a specific stream. However, when specific values are not available, "typical" values from similar streams may be used. In most cases the carbonaceous deoxygenation rate constant (K1) will not be less than 0.2 per day (20°C). Values as high as 3.4 per day (20°C) have been reported in the literature.

Less information is available on the nitrogenous deoxygenation rate constants or nitrification rates in streams. Experimental work in Illinois (State of Illinois, Environmental Protection Agency, Guidelines for Granting of Exemptions from Rule 404(C) and 404(F) Effluent Standards, Oct., 1974) determined that the nitrogenous deoxygenation rate constant (KN) ranged from 0.25 to 0.37 per day with an average value of 0.29 per day at 20°C. The current model uses a KN value based on stream calibration from the modeled stream or similar streams. Other rate constants for benthic and algal kinetics are based on calibration data or literature values. Specific explanations of these rate constants are in the User's Manual for the QUALIIK model.

Dams and Impoundments

The damming of a stream creates special conditions for water quality modeling. For modeling purposes, dams and the resulting impoundments can be put into one of two classifications.

1. Large dams that back up rather extensive impoundments. Flow through the impoundment is not "plug flow" and inflow may be dispersed in a variety of vertical and horizontal directions.
2. Low head dams which essentially make the river channel wider and deeper for a maximum distance of several miles. Flow through the impoundment is primarily "plug flow".

Class 1 dams and impoundments cannot easily be modeled to predict water quality. The modeling effort should be stopped at the beginning of the impoundment and started again below the dam. Water quality below the dam can be estimated from knowledge of the size of the impoundment, the method of water withdrawal, and water quality data from stream surveys. Water taken from the lower levels of an impoundment during periods of summer stratification may be low in DO. If water flows over a spillway or an overflow weir it may be close to the DO saturation point. One can expect the BOD and NH3-N concentrations in the discharge from large impoundments to be low unless the impoundment is highly eutrophic.

Class 2 dams and impoundments can be modeled by treating the impoundment as an enlarged or slower moving reach of the river. The length of the pool backed up by the dam may be divided into one or more reaches. Widths can be approximated from field observations. Slopes are taken as the water surface elevation and are quite small, generally elevation drops off no more than a foot over the length of the pool.

The dams may be treated as a reach 0.001 miles or 5.28 feet in length. The slope of this reach then becomes the dam height divided by 5.28 feet. The only water quality parameter that is significantly affected through the dam reach is the DO. Tsivoglou's reaeration rate constant prediction formula can be used to quite effectively predict reaeration over a dam. The equation for change in the DO deficit with time is:

$$D_t = D_o e^{-K_2 t}$$

where:

Dt = DO deficit at time, t

Do = DO deficit at time zero

K2 = Reaeration rate constant

Tsivoglou's reaeration rate constant predictive equation (neglecting ice conditions) is:

$$K_2 = \frac{c\Delta H}{t}$$

where:

c = Escape coefficient

ΔH = Change in elevation in time, t

Substituting into the DO deficit equation one obtains:

$$Dt = D_o e^{-c\Delta H}$$

Example:

With a dam 10 feet high and c = 0.115/ft. the ratio of Dt/Do is 0.32 or the deficit is 32 percent of the deficit at time zero. This is a DO deficit recovery of 68 percent.

QUALIIK includes a component that can estimate the effect of control structures on oxygen and it is described as follows:

Oxygen transfer in streams is influenced by the presence of control structures such as weirs, dams, locks, and waterfalls. Butts and Evans (1983) have reviewed efforts to characterize this transfer and have suggested the following formula,

$$r_d = 1 + 0.38 a_d b_d H_d (1 - 0.11 H_d)(1 + 0.046T)$$

where rd = the ratio of the deficit above and below the dam, Hd = the difference in water elevation [m], T = water temperature (°C) and ad and bd are coefficients that correct for water-quality and dam-type. Values of a_d and b_d are summarized in Table 7 – coefficient values used to predict the effect of dams on stream reaeration. QUALIIK manual. If no values are specified, QUALIIK uses the following default values for these coefficients: a_d = 1.25 and b_d = 0.9.

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IV. Permit Derivation Procedure

This section describes the method used to translate a wasteload allocation (WLA) into an NPDES permit limit. The procedures are applied to any discharger in the state (municipal, industrial, or semi-public) for whom a water quality-based permit limit is required. The purpose of these procedures are to provide an effluent limit which will statistically assure that the WLA will not be exceeded due to the variations in facility operation, monitoring and parameter analysis.

Statistical-Based Procedure:

Maximum Daily Limits (MDL) and Average Monthly Limits (AML) will be calculated using the statistical procedure noted in Appendix C, Iowa Permit Derivation Methods (pages C1-C3). The Iowa statistical-based procedure adopts the modified 1991 EPA Technical Support Document (TSD) methodology. For toxics, this procedure will consider the required sampling frequency for each water quality based parameter noted in Chapter 63 of the department rules and any known coefficient of variation (CV) for each parameter. This CV may be based on the individual treatment facility's operations. Where the CV data is lacking, a value of 0.6 will be used. If a wastewater treatment facility selects to increase the monitoring frequency, the corresponding permit limits will be calculated to reflect this increased frequency. For ammonia, the permit limits are derived directly from the acute WLAs and chronic WLAs.

In addition, technology-based requirements must also be met.

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Definition of Variables:

WLA_a = Acute Wasteload Allocation
WLA_c = Chronic Wasteload Allocation
CV = Coefficient of Variation
n = Sampling Frequency
MDL = Maximum Daily Limit
AML = Average Monthly Limit

Statistical-Based Procedure:

The modified 1991 EPA Technical Support Document (TSD) methodology is adapted for the Iowa statistical-based procedure to derive the permit limits from the wasteload allocations. The following section describes the different procedures used to derive the permit limits for ammonia and toxics.

1. Ammonia
   $MDL = WLA_a$
   If $WLA_c < WLA_a$, $AML = WLA_c$
   Otherwise, $AML = MDL = WLA_a$

2. Toxics

First, a treatment performance level (LTA and CV) needs to be determined to allow the effluent to meet the WLA requirement. Where two requirements are specified based on different duration periods (i.e., WLA_a and the WLA_c), two performance levels are calculated.

The LTA_a is determined by the following equation:

$$LTA_a = WLA_a e^{\left[0.5\sigma^2 - z\sigma\right]}$$

$$\text{where } \sigma^2 = \ln\left(CV^2 + 1\right)$$

The LTA_c is determined by the following equation:

For 4-day chronic averaging period (i.e., for toxics):

$$LTA_c = WLA_c e^{\left[0.5\sigma_4^2 - z\sigma_4\right]}$$

$$\text{where } \sigma_4^2 = \ln\left(CV^2 / 4 + 1\right)$$

The z value for the LTAs is based on a 0.01 probability basis, i.e. the 99th percentile level, with a value of 2.326. The default CV value is 0.6 unless applicable data is provided by the wastewater treatment facility.

Next, permit limits are derived directly from the corresponding LTA value; in other words, the MDL is calculated from LTA_a and the AML is calculated from the LTA_c.

The MDL is calculated by the following equation:

$$MDL = LTAe^{\left[z\sigma - 0.5\sigma^2\right]}$$

$$\text{where } \sigma^2 = \ln\left(CV^2 + 1\right)$$

The z value for MDL is based on a 0.01 probability basis, i.e. the 99th percentile level, with a value of 2.326.

The AML is calculated using the equation:

$$AML = LTAe^{\left[z\sigma_n - 0.5\sigma_n^2\right]}$$

$$\text{where } \sigma_n^2 = \ln\left(CV^2 / n + 1\right)$$

The z value for AML is based on a 0.01 probability basis, i.e. the 99th percentile level, with a value of 2.326. The monitoring frequency (n) will follow the requirements noted in the department's rule, Chapter 63. However, the n value used to calculate the AML should always be greater or equal to 4/month to guarantee meeting the criterion.

If the above calculated AML is greater than the MDL, set AML = MDL.

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