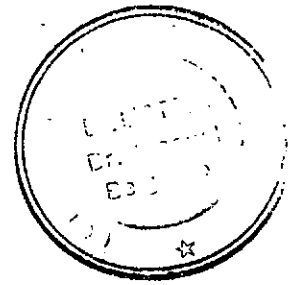


BS 721



CONSEJO FEDERAL DE INVERSIONES

PROVINCIA DE RIO NEGRO

MINISTERIO DE OBRAS Y SERVICIOS PUBLICOS

DEPARTAMENTO PROVINCIAL DE AGUAS

ESTABLECIMIENTO DEPURADOR CLOACAL EN

SAN CARLOS DE BARILOCHE

PRIMER INFORME PARCIAL: "ANTECEDENTES"

EXPERTO: ING. MARCELO JOSE PUJOL

1989

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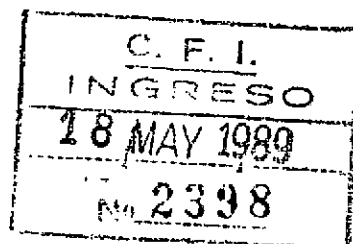
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MARCELO JOSE PUJOL

La Plata, 15 de mayo de 1989

Señor
Secretario General del
Consejo Federal de Inversiones
Ing. Juan José Ciacera
S/DESPACHO

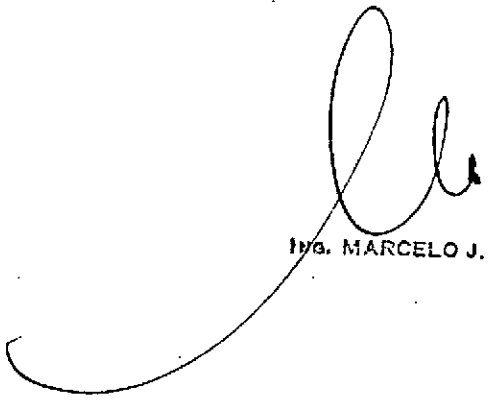


REF: Establecimiento Depurador Cloacal
San Carlos de Bariloche. Exp. 442.

De mi mayor consideración:

Tengo el agrado de dirigirme a Ud. con el objeto de
var a su consideración el Primer Informe Parcial: "Antecedentes", en
ejemplares de 93 fojas cada uno.

Saludo a Ud. muy atentamente.



Ing. MARCELO J. PUJOL

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1 - ANALISIS DE ANTECEDENTES

1.1.- ANTECEDENTES APORTADOS POR EL CFI Y DPA

En general, los antecedentes obrantes en el CFI, en su mayor parte ya introducidos y utilizados en el Anteproyecto Preliminar, pueden considerarse actuales y exhaustivos aunque no suficientes para los requerimientos del Anteproyecto Preliminar.-

Fueron oportunos y de mucha utilidad los datos relativos a la explotación del servicio así como los planos de proyectos anteriores y de instalaciones existentes. En cambio el estado catastral-dominial actual del predio destinado a la planta depuradora, no surge de los antecedentes aportados.-

Son copiosos los datos hidrométricos del lago, pero lamentablemente no se cuenta con observaciones sistemáticas útiles para evaluar la evolución del lago en cuanto a eutrofización.-

Buena parte de los antecedentes originales de OSN fueron entregados por el CFI y entre ellos los referentes a estudios geotécnicos y planialtimétricos de detalle.-

Asimismo las copias del Libro de Cordini "El Lago Nahuel Huapi" y las del trabajo de Mogensen y Ortiz, "Estudio de la Contaminación de las Aguas del Lago Nahuel Huapi, 1981" facilitaron la introducción al tema.-

1.2.- ANTECEDENTES OBTENIDOS DEL ANTEPROYECTO PRELIMINAR

Particularmente en la etapa de "Estudios Preliminares" se pone de manifiesto la capacidad operativa del equipo de consultores para realizar una completa caracterización del medio básico en que el proyecto se desarrolla. Muy útiles fueron, asimismo, -- las mediciones realizadas por los consultores sobre los parámetros de explotación, así como la recopilación de análisis de OSN y propios. No se hicieron aportes significativos sobre eutrofización, ni tampoco en cuanto a la planialtimetría del predio. Hay un perfil batimétrico apropiado al nivel de esa etapa pero insuficiente para la presente etapa.-

1.3.- ANALISIS DE LOS INFORMES DE LA 1ª PARTE DEL ESTUDIO

Hemos podido comprobar la buena correlación con los hechos que presenta el diagnóstico formulado en el Anteproyecto Preliminar, una de sus etapas más fructíferas y completas. En cuanto al planteo que se hace del establecimiento depurador, el nivel en que han sido desarrolladas las cuatro alternativas (percoladores, barro activados, aireación extendida sistema "D Per" o "Carrousel") es muy adecuado a los efectos de la selección, -- aunque atados a patentes los dos últimos.-

El tratamiento de eliminación de fósforo ha sido en todos los casos prácticamente soslayado, quizá por la convicción de que no se justificaría implementarlo en la primer etapa.-

En cuanto a las hipótesis de trabajo y parámetros de diseño -- utilizados en la alternativa favorecida, los mismos son discutidos y comentados en el apartado 3.2.

2 - RECONOCIMIENTO DE CAMPO

El día 30/III/89 se comenzó a realizar los reconocimientos de campo en forma conjunta con las autoridades y técnicos del servicio San Carlos de Bariloche, entre cuyos fines figuraba la - programación de tareas a realizar por parte del DPA. El día 2/IV/89 se dieron por concluídos los reconocimientos de campo, - procediéndose a listar las tareas requeridas, las que apunta--ban, en lo esencial, a completar al nivel necesario los datos topográficos, catastrales y batimétricos y a actualizar datos de explotación y costos de energía.-

Particularmente llamativo resultó el advertir que el predio --destinado al Establecimiento estaba subdividido, con tres (o - más) construcciones en probablemente seis de los lotes de la - manzana 214, lo cual nos movió a solicitar un estudio dominial. También se determinaron las tareas geotopográficas necesarias y dos perfiles para relevar batimétricamente.-

Se sobrevoló la zona, tratando de completar una imagen ajusta--da de la interacción del lago con sus costas a los efectos de preveer la descarga.-

El entusiasmo evidenciado por los técnicos no ha alcanzado pa--ra superar reales dificultades que, lo sabíamos, iban a presen--tarse, razón por la cual gran parte de las respuestas corren - con atraso.-

El estudio dominial confirmó nuestra inquietud, revelando la - para nosotros novedad de que hay 28 lotes privados, 10 de los cuales han sido declarados de utilidad pública, siendo los de--más pertenencia del Fisco Provincial. Esto motivó lógica incer--tidumbre, despejada recién el día 3/V/89 con la recepción de - un telegrama manifestando la decisión por parte de las autori--dades provinciales de disponer de los terrenos que el proyecyo requiriese, aún por vía de expropiación. De cualquier manera, esa situación se tuvo en cuenta al momento de generar subalter--nativas de diseño, tratando de no interferir con las obras de primera ejecución.-

MEMORANDUM del Experto Contratado Ing. Marcelo Fujol
al Jefe de Servicio S.C.deBariloche Ing. Viotti

En relación con el Proyecto ESTABLECIMIENTO DEPURADOR CLOACAL
se convienen las siguientes tareas a cargo del D.P.A.:

1)Tareas Topográficas.

- Referenciación del polígono del predio al tegido urbano. Verificación de cesión de calles laterales. Ancho de ruta y de calles. Estudio de título y verificación de eventuales restricciones al dominio.
- Relevamientos de hechos existentes, referenciados al polígono. Casca de Bombas, desagüe a la costa, cañería de vuelco, manantiales, eventuales intrusos, etc. Relevamiento de las líneas de alta y baja tensión y de teléfonos.
- Vinculación altimétrica de la nivelación realizada por OSN con el sistema del I.G.M.Eventualmente, densificación de pts.
- Tres (3) perfiles batimétricos hasta superar la profundidad de 50 metros en los siguientes lugares:
 - a) sobre el caño de vuelco existente.
 - b) 200 metros aguas abajo (al EQ
 - c) En el extremo saliente de la punta, frente al vivero Steiner, reconocido por el Ing. Milano.
- Polígono de vinculación entre los límites del predio y la punta del vivero Steiner.

2)Datos del Servicio

- Implantación de una escala y determinación de la función H/Q en la Estación de Bombeo.
- Determinación del máximo caudal en tiempo seco y en tiempo lluvioso. Volúmen diario vertido.
- Temperaturas seriadas del lago, del líquido cloacal y del aire.
- Muestreo para análisis físico químico de los líquidos, con la frecuencia que se pueda (por. ej. diario). Incorporar determinación de DQO y de Fósforo.
- Facilitar los Planos s/obra de la Est. de Bom. y de la Cañería máxima.
- Disponibilidad y detalles de tramitación y condiciones para lograrxx los servicios de Teléfono, gas y energía eléctrica.
- Posibilidad y maneras de lograr agua corriente.
- Estudio de tarifas eléctricas en las siguientes condiciones de consumo:
 - a)400 Kw uniformes, es decir 288000 kwhora/mes.
 - b)Idem para 399kw y 401kw.
 - c)para 300 kw de consumo promedio, es decir a razón de 216000 kwhora/mes pero con picos de hasta 800 kw.

Protegido o / 11/4/89

La Plata, 27 de abril de 1989.

Señor Secretario General del
Consejo Federal de Inversiones
Ing. Juan José Ciacara
S/DESPAÑO

REF: Establecimiento Depurador Cloacal
para San C. de Bariloche.

De mi mayor consideración:

Tengo el agrado de dirigirme a Ud. en relación a la marcha de los trabajos de referencia que se llevan a cabo mediante el concurso del suscripto en calidad de experto, el Departamento Provincial de Aguas de la Provincia de Río Negro y personal profesional y técnico de ese Consejo.

De acuerdo con el Cronograma previsto, en ocasión de nuestra visita al lugar llevada a cabo el 30 de marzo ppdo., hemos acordado con funcionarios del D.P.A., las tareas de campo necesarias para el desarrollo de la primera etapa del trabajo. Una reciente comunicación recibida el 25 del corriente, da cuenta de la marcha de esas tareas, las que por diferentes razones están retrasadas (se adjunta copia).

Por otra parte, al confrontar los resultados parciales de nuestro análisis de antecedentes sobre los que veníamos trabajando con la información recibida, surge un imprevisto que requiere una definitiva toma de decisión por parte de la Provincia de Río Negro.

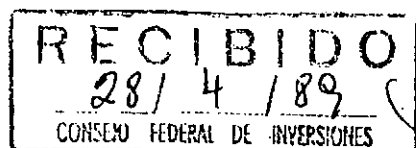
Concretamente, los terrenos destinados al emplazamiento de las obras están en parte subdivididos y enajenados; aún más, en algunas parcelas se advierten construcciones de carácter permanente.

Además, conforme progresan nuestros estudios sobre implantación de las unidades de tratamiento, surge como alternativa aconsejable la afectación de una parte del predio vecino perteneciente al I.N.T.A. (unas dos hectáreas).

Sin perjuicio de avanzar en temas desvinculados, es obvio que los antecedentes más importantes están asociados con el predio del propio Establecimiento, tanto en cuanto a su superficie, forma, altimetría, costas, etc., razón por la cual, la etapa en curso, "Análisis de Antecedentes", no puede ser concluida.

En base a lo expuesto, correspondería prorrogar el plazo previsto para esta etapa, y consecuentemente para la siguiente, hasta diez días contados a partir de la fecha en que las autoridades provinciales comuniquen la decisión al respecto.

Saludo a Ud. muy atentamente.



Ing. MARCELO J. PUJOL

C.F.I.
INGRESO
28 / ABR 1988
Nº 2100

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ATTACHED

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DESTINATARIO: SECRET. GRAL. DEL C.F.I.
ING. J. JOSE. CIACHA

TEXTO:

ATENCION: ING. NICOLAS RATTO

MANIFESTAMOS DECISION PROVINCIA EXPROPIACION TERRENOS NECESARIOS
PLANTA DEPURADORA LIQUIDOS CLONCALES GARILACHE EN FUNCION
ATRIBUCIONES SUPERINTENDENTE GENERAL Y DE ACUERDO A NECESIDADES A
NIVEL ANTEPROYECTO DEFINITIVO.

ATENCIÓN: ING. FERNANDO ERICA
INTENDENTE GENERAL DE OBRAS Y SERV. SANITARIOS.
DEPARTAMENTO PROVINCIAL DE AGUAS.

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817290PARU ARHMM

3 - ESTUDIOS BASICOS COMPLEMENTARIOS

La primera parte de este apartado será destinada al diagnóstico y pronóstico del Lago Nahuel Huapi en cuanto a su estado --trófico, un tema que conceptuamos de suma importancia, y pendiente de estudio, a los efectos de dar consistencia a la política a adoptar en materia de tratamiento de eliminación de fósforo.-

En la segunda parte se anticipa el desarrollo del diseño y dimensionado de la planta depuradora, movidos por la necesidad - de chequear la aptitud del predio disponible y la eventual necesidad de promover nuevas expropiaciones, comenzando por una revisión minuciosa de las hipótesis de trabajo en un nivel en - que, por los alcances propios del Anteproyecto Preliminar pudieron no haber sido desarrolladas en esa etapa. En esta segunda parte se desarrolla el proceso de eliminación de la carga - orgánica carbonácea.-

En la tercer etapa se volverá al tema del fósforo, proponiendo la estrategia que se juzgue más conveniente.-

3.1.- PRONOSTICO DE LA EVOLUCION DEL ESTADO TROFICO DEL LAGO NAHUEL HUAPI

3.1.1 - INTRODUCCION

La materia orgánica que llega a un lago es descompuesta por organismos heterotrofos que la convierten en dióxido de carbono a costa de insumir oxígeno. En cambio, los organismos fotosintéticos utilizan dióxido de carbono y producen oxígeno y materia orgánica vegetal. La eutrofización es la pérdida de estabilidad del ecosistema hacia un crecimiento desmedido de los fotosintéticos (algas). Cuando estos mueren se convierten en sustrato orgánico que ejerce nueva demanda de oxígeno; aún sin morir, muchas algas utilizan oxígeno para oxidar compuestos orgánicos fotosintetizados cuando disponían de luz, para así conseguir energía sustitutiva.-

Los lagos de montaña suelen ser oligotróficos, es decir que contienen pocos nutrientes en solución. Permanecen límpidos, con buenos niveles de oxígeno disuelto, el crecimiento vegetal no prospera y formas superiores de peces que como la trucha se alimentan de insectos, se ven favorecidos.-

Hay una evolución natural de los lagos hacia la eutrofización, pero es muy lenta. El principal factor antrópico o cultural que acelera extraordinariamente esa evolución es el vertido de aguas cloacales.-

3.1.2 - PARAMETROS DE EUTROFIZACION

Los principales indicadores del estado de un lago comunmente utilizados son: transparencia o profundidad Secchi, fósforo total, clorofila algal, nitrógeno particulado, oxígeno disuelto y tasa de utilización de oxígeno hipolimnético.-

Vollenweider en 1968 fué el primero en fijar concentraciones límites. Recientemente, H. Dobson ha presentado el siguiente cuadro:

Estado	Fósforo Total (en primavera) µg/l	Nitrógeno Particulado µg/l	Clorofila Algal (en verano) µg/l
Oligotrófico	< 10	< 50	< 2
Mesotrófico	10-30	5-150	2-6
Eutrófico	30-90	> 150	6-18
Hipertrófico	> 90	> 150	> 18

También se ha utilizado el "Volumen de Algas" en cm³/m³, como los consignados por Rosech, Cullen y Bek para lagos de Australia (5):

	Volumen de Algas	Fósforo Total	Nitrógeno Total	Clorofila "A"
Oligotróficos	< 1	< 5	< 250	0,3-3
Mesotróficos	1-3	5-30	250-1000	2-15
Eutróficos	3-5	30-1000	1000-10000	10-500

3.1.3 - CONTROL DE LIMITANTES

Para controlar la eutrofización puede actuarse sobre algunos elementos nutrientes, tales como C, N ó P, los que intervienen en la materia algal en relación de peso 40-7-1. Actualmente hay consenso en que el más apropiado de los tres como elemento de control o limitante es el P, por varias razones:

- En la mayoría de los lagos no polucionados por el hombre, el fósforo es el limitante natural. El C y el N están siempre presentes en forma facilmente utilizables, como dióxido de carbono y carbonatos el primero y como nitratos y amonios el segundo.-
- Está comprobado por la práctica de eliminación del fósforo

de los líquidos cloacales, que controlando el fósforo humano se controla el lago. Más aún, reañadiendo fosfato, los efectos se invierten (7).-

- La eliminación del fósforo está facilitada por el hecho de que los líquidos que lo vehiculizan, son concentrados y manipulados en las plantas depuradoras.-

3.1.4 - FORMAS DEL FOSFORO AGREGADO

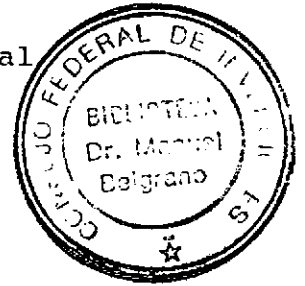
El fósforo puede presentarse en forma de fósforo orgánico - en la materia orgánica y en el protoplasma celular, como fosfatos inorgánicos complejos (polifosfatos) tales como los usados en detergentes y como ortofosfatos inorgánicos solubles. Estos últimos son los que corresponden al estado final del ciclo de degradación (PO_4^{3-} , HPC_4^{2-} , H_2PO_4^- , H_3PO_4) y pueden ser fácilmente asimilados sin necesidad de posteriores rupturas. Durante el tratamiento biológico, mucho del fósforo orgánico y de polifosfatos se vuelven ortofosfatos solubles, y se van con el efluente; una pequeña cantidad de fósforo orgánico también fuga con las células no retenidas y otra, con los barros en exceso, son devueltas al suelo. Si el tratamiento incluye remoción de fósforo mediante precipitación química, el efluente contendrá formas insolubles, como fosfato de calcio, aluminio o hierro. La práctica ha demostrado que estos compuestos no liberan el fósforo ni en las unidades de tratamiento, ni en los cuerpos receptores (6). En cambio los polifosfatos, si bien son duros o refractarios, terminan en ortofosfatos por hidrólisis.-

3.1.5 - FUENTES DE FOSFORO

Las fuentes pueden ser difusas o puntuales. Las primeras comprenden el escurrimiento superficial sobre áreas rurales urbanas que llegan al lago sin canalizar y las segundas, los lanzamientos y vuelcos de aguas servidas, domésticas o industriales. También hay fósforo meteórico aportado por las lluvias.-

El siguiente cuadro ha sido propuesto por Rast y Lee:

Uso del Suelo	Fósforo Total (g/m ² /año)
Urbano.....	0,1
Rural y Agrícola.....	0,05
Forestal.....	0,01
Lluvias y Partículas del Aire.....	0,02



Para evaluar el fósforo total aportado por la población humana, está el cuadro siguiente, según Vollenweider:

Cargas de fósforo sobre superficies de aguas superficiales debidas a excrementos humanos y otras fuentes, basadas en un promedio de - 2,25 g P/hab.día.

Densidad Poblacional Hab./Km ²	Fósforo g/m ² /año
50	0,04
100	0,08
150	0,12
200	0,16
300	0,24
500	0,40
1.000	0,80
2.500	2,00
5.000	4,00

Las concentraciones normalmente encontradas en aguas cloacales van de 8 a 10 mg P/l o bien $\frac{1}{20}$ de la concentración de DBO_5 (6). En el Anteproyecto Preliminar, se menciona que en el curso del estudio, el líquido cloacal de SC de B. presentó valores relativamente bajos de fosfato: 1,6 a 5,5 mg/l. - No se mencionan valores de P total. Sin embargo, estos valores pueden cambiar con los hábitos de la población y con el consumo de detergentes y los lavaderos.-

En (5) se cita como concentración común 12-15 mg/l de P total. En la planta de Gral. Sarmiento, P.B.A., hay valores entre -- 7,1 y 12,5 mg/l de $\text{PO}_4^{=}$ lo que equivale a proporciones de entre $\frac{1}{15}$ y $\frac{1}{20}$ con respecto a la DBO_5 . Metcalf-Eddy proporciona el siguiente cuadro:

	Fuerte	Medio	Débil
Fósforo Total	15 mg/l	8 mg/l	4 mg/l
Orgánico	5 "	3 "	1 "
Inorgánico	10 "	5 "	3 "

El fósforo meteórico depende fuertemente del desarrollo industrial de la zona y de la práctica de fertilización unida al grado de erosión del suelo. Se citan valores entre 0,01 y 0,1 mg/m²/año pero dadas las características del lago N.H. aún el límite inferior parece alto.-

Finalmente, las aguas subterráneas que descargan a los lagos, no transportan cantidades apreciable de fósforo debido a la insolubilidad de los fosfatos contenidos en los minerales y en el suelo (1).-

3.1.6 - CICLO DEL FOSFORO EN EL LAGO

Ya en el lago, el fosfato tiende a pasar por ciclos estacionales. El fosfato disuelto se acumula en invierno cuando la actividad fotosintética es baja y las aguas no están estratificadas. En primavera y verano, con el gradiente térmico y la actividad fotosintética, los fosfatos disueltos en el epilimnion se agotan debido a su utilización y almacenamiento por parte de los vegetales. Por eso que el tenor de fosfato disuelto debe medirse en invierno y el de clorofila en verano. Al abatimiento inicial suelen suceder florecimientos algales (lo cual puede parecer contradictorio) debido a la utilización del fósforo almacenado por los propios organismos. Esa modalidad de acaparar fosfato por encima de las necesidades actuales también se da en las bacterias y es la base (captación lujuriosa) de los métodos biológicos de eliminación de -

fósforo.-

Más tarde, la muerte y sedimentación de las algas da lugar a un lento transporte de fósforo del agua a los sedimentos, en contraste con la rápida captación por parte de las algas y plantas. Más lenta aún es la complicada cadena de reacciones que devienen en los depósitos bentónicos, como resultado de las cuales una parte del fósforo contenido es liberado nuevamente y reciclado al agua. Mucho del fósforo queda sepultado en forma de compuestos muy estables. Más aún si el fondo del lago recibe descargas periódicas de suelos. Este efecto de entrampamiento es más importante en lagos muy profundos y con altos períodos de retención hidráulica. Ambas características son distintivas del N.H. así como la ausencia de termoclinas. Posiblemente se deba a ellas el hecho de que hasta el día de hoy los signos de eutrofización están circunscritos a algunas costas, y no en forma dramática.-

Pero a nuestro entender, este comportamiento inercial del N. H. es un arma de doble filo, que obliga a actuar anticipadamente, porque también será muy lenta y trabajosa cualquier acción correctiva que pretenda encararse en el futuro.-

3.1.7 - MODELOS DE PRONOSTICO

Tanto para pronosticar una evolución hacia la eutrofización como para predecir los resultados de una acción tendiente a limitar la carga, se han desarrollado varios métodos.-

El primer estudio sistemático comenzó a partir de la década de los 60, como una decisión de la Organización para Cooperación y Desarrollo Económico (OECD). Vinculado a ese programa Vollenweider presentó en 1968 el siguiente cuadro:

NIVELES PROVISIONALES DE CARGAS PERMISIBLES PARA
NITROGENO TOTAL Y FOSFORO TOTAL BIOQUIMICAMENTE
ACTIVO, EN $q\ m^{-2}\ año^{-1}$ (1)

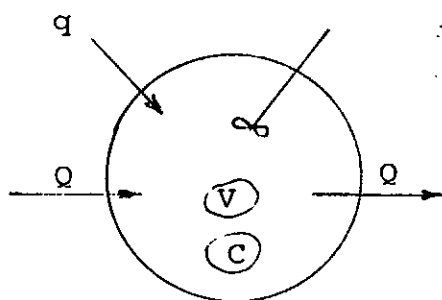
Profundidad Media (m)	Carga Permisible		Carga Peligrosa	
	N	P	N	P
5	1	0,07	2	0,13
10	1,5	0,10	3	0,20
50	4	0,25	8	0,50
100	6	0,40	12	0,80
150	7,5	0,50	15	1,00
200	9	0,60	18	1,20

Más adelante (1975), V. desarrolló un diagrama de carga superficial anual de P vs. el cociente entre la profundidad media y el tiempo de retención hidráulica. Este criterio ha sido respaldado por la EPA como base para establecer criterios de aportes para lagos y embalses de EE.UU. (criterios de calidad de Agua, EPA, USA, 1976) así como también por el Comité Conjunto Internacional Canadá - USA. Luego V. modificó nuevamente su planteo, haciendo intervenir más activamente la retención hidráulica (1976).-

El éxito de este nuevo modelo, llevó también a aplicarlo, mediante correlación, a otros parámetros indicadores de eutrofización tales como clorofila en verano, profundidad Secchi en verano y tasa de agotamiento de oxígeno hipolimnético (2). En general se lo cita como "Modelo Volenweider-OECD". Dada su importancia como herramienta práctica, nos hemos detenido en analizar brevemente sus fundamentos, apareciendo algunas relaciones muy interesantes y no siempre explicitadas, que expon-dremos a continuación.-

3.1.7.1. Modelo Conservativo de Mezcla Completa

En un reactor de mezcla completa que trabaja en régimen hidráulicamente estacionario Q , se viene adicionando un caudal másico q_0 de un cierto componente que no se destruye. A partir del tiempo $t = 0$ ese caudal másico se incrementa en q . La concentración C en el reactor comienza a prosperar desde $C_0 = -\frac{q_0}{Q}$ hasta llegar en un tiempo infinito a $C_\infty = -\frac{q}{Q}$. Veremos de hallar $C = C(t)$.



$$t = 0, q = q_0, C = C_0$$

$$t = \infty, q = q, C = C_\infty$$

Un balance másico diferencial nos da:

MASA ENTRANTE = INCREMENTO DE MASA ALMACENADA + MASA SALIENTE

$$q \, dt = V \, dc + c \, Q \, dt$$

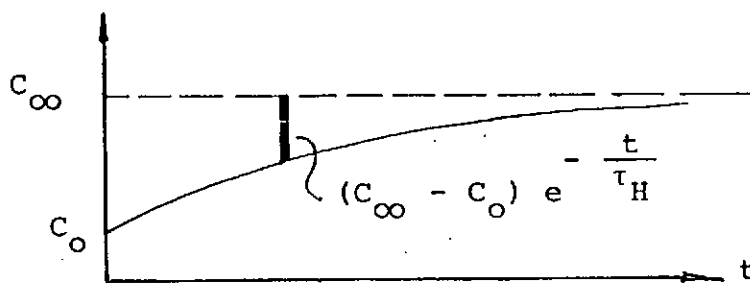
$$V \frac{dc}{dt} + Qc = q, \quad \text{si } t = 0, \quad c = \frac{q_0}{Q}$$

Ecuación lineal cuya solución es:

$$C = \frac{q}{Q} - \left(\frac{q}{Q} - \frac{q_0}{Q} \right) e^{-t \frac{Q}{V}}$$

o bien, utilizando $C_0 = \frac{q_0}{Q}$, $C_\infty = \frac{q}{Q}$ y $\tau_H = \frac{V}{Q}$ ó tiempo de retención hidráulica, la anterior puede escribirse:

$$C = C_\infty - (C_\infty - C_0) e^{-\frac{t}{\tau_H}}$$



en la que el segundo término, representa la carrera de C desde el origen de la perturbación hasta su estabilización. Si quisieramos plantear en que tiempo será recorrido el 64% de esa carrera, bastaría plantear:

$$e^{-\frac{t}{\tau_H}} = 1 - 0,64$$

$$-\frac{t}{\tau_H} = \ln 0,36 \Rightarrow t = \tau_H$$

El tiempo de retención hidráulica del N.H. es de 12,4 años, y en un lapso como ese, la mayor parte de la carrera de C hacia su valor de estabilización habrá tenido lugar. Una primer con clusión importante es que el lago tiene una respuesta relativamente rápida y que no convendría especular demasiado con la capacidad de almacenar sustancias en solución de sus aguas. - Por el contrario, las concentraciones observadas en los últimos tiempos, pueden considerarse bastante próximas a los valores estacionarios correspondientes a los actuales estados de carga, sin cometer errores importantes. Esta aproximación, -- evita la complejidad de los modelos dinámicos y lleva a con si derar solo los estados estacionarios.-

Aplicando el criterio de la concentración crítica de fósforo, podría entonces lograrse un diagrama como éste:

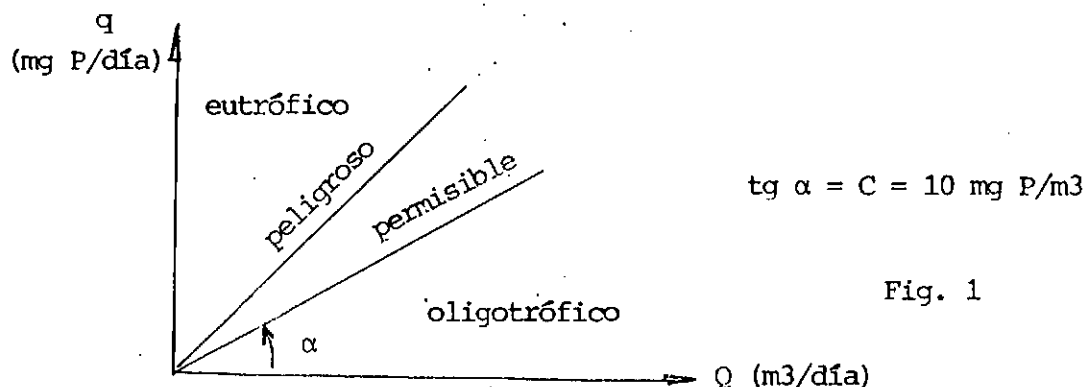


Fig. 1

3.1.7.2 Modelo no Conservativo de Mezcla Completa

Este modelo se basa en la misma ecuación de balance másico -- del modelo anterior, pero corregida con el agregado de dos -- términos que tienen en cuenta dos hechos importantes (3):

- a) Los sedimentos retienen hasta su total mineralización parte del fósforo, sin regenerarlo o liberarlo a la masa acuosa. Luego, se comportan como un sumidero de fósforo.-
- b) La mezcla completa debe interpretarse como hipótesis de di fusión instantánea en sentido horizontal. Verticalmente, -- empero, la estratificación de las aguas hace que la con cen tración del efluente (tomado como es lógico de las capas superiores del epilimnion) no represente la concentración media del lago.-

Para matematizar la parte que se "pierde" en el fondo se formula una ecuación de primer grado, es decir que se aprecia como constante la tasa anual de decaimiento:

$$K = \frac{\text{MASA DE FOSFORO PERDIDO}}{\text{MASA DE P CONTENIDO} \cdot \text{INTERVALO DE TIEMPO}}$$

$$\text{MASA DE P PERDIDA} = K (V C) dt$$

siendo como antes, V el volúmen del lago y C la concentración media.-

La estratificación se tiene en cuenta apreciando la parte --- $\alpha < 1$ de la concentración media que presenta el efluente. Es decir:

$$\text{MASA DE P QUE FUGA} = \alpha C Q$$

La ecuación de balance resulta:

$$q dt = V dc + K V C dt + \alpha C Q dt$$

o bien:

$$V \frac{dc}{dt} + (K V + \alpha Q) C = q$$

ecuación lineal cuya solución es:

$$C = \frac{q}{K V + \alpha Q} - \left(\frac{q - q_0}{K V + \alpha Q} \right) e^{-t \frac{K V + \alpha Q}{V}}$$

en la que, como antes, q_0 representa el flujo másico estacionario antes del cambio de régimen. Llamando:

$$C_0 = \frac{q_0}{K V + \alpha Q} = \text{concentración media a } t = 0$$

$$C_{\infty} = \frac{q}{K V + \alpha Q} = \text{concentración media de estabilización}$$

$$\frac{V}{Q} = \tau_H = \text{tiempo de retención hidráulica}$$

$$\frac{C V}{C (\alpha Q + K V)} = \tau_P = \frac{\text{MASA DE P CONTENIDO}}{\text{FLUJO MASICO PURGADO Y PERDIDO}} =$$

= tiempo de retención química del fósforo en régimen estacionario

la ecuación de estado puede escribirse:

$$C = C_{\infty} - (C_{\infty} - C_0) e^{-t/\tau_P}$$

que nos dice, otra vez, que cuando $t = \tau_P$ la concentración C ha recorrido el 64% de su carrera. Como τ_P es menor que τ_H de este modelo surge que la respuesta del lago es todavía más --- ágil que la que esperábamos del modelo anterior. Poniendo énfasis en los estados estacionarios:

$$C_0 = \frac{q_0}{K V + \alpha Q} \frac{V}{V} = \frac{q_0}{V} \tau_P \frac{Q}{Q}$$

$$C_0 = \frac{q_0}{Q} \frac{\tau_P}{\tau_H}$$

$$C_{\infty} = \frac{q}{K V + \alpha Q} \frac{V}{V} = \frac{q}{V} \tau_P \frac{Q}{Q} = \frac{q}{Q} \frac{\tau_P}{\tau_H}$$

$$C_{\infty} = \frac{q}{Q} \frac{\tau_P}{\tau_H}$$

En la práctica interesa entonces el cociente $\frac{\tau_P}{\tau_H}$ el que puede ser ajustado evaluando simultáneamente q_0 y C_0 . En estos casos es necesario medir C_0 en invierno o al comienzo de la primavera cuando, por enfriamiento de las capas superiores, las aguas están mezcladas. Luego, con la estratificación térmica, aparece el estancamiento de primavera.-

De observaciones realizadas en 17 lagos surgen valores de $\frac{\tau_P}{\tau_H}$ de entre 0,13 y 0,8, es decir, siempre $\tau_P < \tau_H$. El efecto "sumidero" tiende a disminuir el tiempo de retención química mientras el efecto "estratificación" a aumentarla: domina la primera. El valor α es adimensional (por ejemplo $\alpha = 0,8$) y el de K es t^{-1} (por ejemplo $K = 0,3 \text{ año}^{-1}$ para N.H.). En función de α y K , el cociente resulta:

$$\frac{\tau_P}{\tau_H} = \frac{\frac{V}{\alpha Q + K V}}{\frac{V}{Q}} \quad \frac{\tau_P}{\tau_H} = \frac{1}{\alpha + K \tau_H}$$

3.1.7.3 El Modelo Vollenweider - OECD

Aún cuando muchos esfuerzos se han destinado a fórmular modelos dinámicos en los que se trata de matematizar los procesos de crecimiento algal y utilización de sustratos, relacionándolos con las cargas de diferentes nutrientes y las condiciones ambientales (8) hasta el momento (1982) no se ha logrado ninguno con buena capacidad predictiva (2).-

Una alternativa de estos modelos dinámicos la constituye el modelo estadístico conocido como de V - OECD (1975). Rast y Lee (1978) ampliaron la base experimental de ese trabajo, lo cual contribuyó a consolidar esta importante herramienta. En 1976 V. halló una correlación estadística entre el cociente τ_P/τ_H y la retención hidráulica τ_H , comenzando a utilizar el parámetro "carga superficial normalizada" la que relacionó con la clorofila (4). En esa misma línea, Rast y Lee hallaron también otras relaciones entre ese parámetro perfeccionado y otros indicadores de eutrofización como la profundidad Secchi y la tasa de abatimiento de oxígeno hipolimnético. La calibración de esas correlaciones fué un tema muy de moda. En el país disponemos de un valioso trabajo del Dr. R. Quiros (9).-

Todos estos trabajos tienen una base empírico-estadística. El principal aporte es la elección de las coordenadas del plano en que las observaciones son mostradas. La primer idea fué la de dividir ambas coordenadas de la Figura 1 por el área del lago, y transportar las rectas "permisible" y "peligroso" al plano doble logarítmico. Entonces:

$$\log. \frac{q}{A} = \log (C_K \frac{Q}{A})$$

C_K = concentración crítica

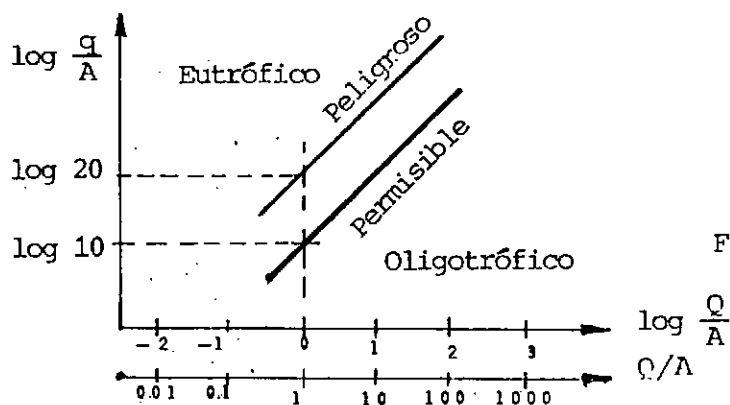


Fig. 2

Resultan dos rectas paralelas a 45° .-

Es decir, el "estado" queda determinado por la carga másica superficial aplicada de fósforo y por la carga hidráulica superficial del lago. -

En lugar de hablar de carga hidráulica superficial $\frac{Q}{A}$, V. prefirió una definición enteramente equivalente:

$$\frac{Q}{A} = \frac{Q \bar{Z}}{A \bar{Z}} = \frac{Q \bar{Z}}{V} = \frac{\bar{Z}}{\tau_H}$$

Siendo \bar{Z} = profundidad promedio.-

La novedad resultó en que al mostrar los resultados de la experiencia, los límites resultaron distorcionados hacia $\frac{q}{A} = \text{cte}$ en la zona de baja carga hidráulica superficial, como si en esa zona el lago se comportase como un eficiente sedimentador de fósforo: sólo con cargas másicas superficiales aplicadas -- por encima de un cierto umbral, el proceso de eutrofización se pondría en marcha. A este modelo se lo conoce como Vollenweider 1975.-

Posteriormente, V. adoptó los conceptos del modelo no conservativo antes expuesto, o sea el modelo de la permanencia química. Al contrastarlo con la experiencia halló una dependencia del cociente τ_P/τ_H con la retención hidráulica τ_H

$$\frac{\tau_P}{\tau_H} = \frac{1}{1 + \sqrt{\tau_H} \text{ (años)}}$$

Entonces, como

$$C_K = \frac{q/A}{Q/A} \frac{\tau_P}{\tau_H}$$

resulta:

$$q/Q = C_K (1 + \sqrt{\tau_H} \text{ (años)})$$

que representó en gráfico doble logarítmico con coordenadas $\frac{q}{Q}$ vs. τ_H :

$$\log \frac{q}{Q} = \log C_K + \log (1 + \sqrt{\tau_H})$$

V. adoptó $C_K = 10$ mg/m³ como concentración permisible y $C_K = 20$ mg/m³ como peligrosa. A este modelo lo llamaremos V.1976 (4).-

En la Figura. 3 se muestran superpuestos en el plano $\log \frac{q}{A}$ / $\log \frac{Q}{A}$, los tres modelos: a) conservativo, b) V.1975 y c) --- V.1976 para $C_K = 10$ mg/m³. En el modelo V.1976 en el plano -- $\log q/A - \log Q/A$ la curva de nivel $C_K = 10$ mg/m³ debe ser elegida en función de τ_H (años). Un punto de esta curva de abscisa - $\frac{Q}{A}$ se corresponde con un \bar{z} (m) = $\tau_H \frac{Q}{A}$ que puede ser hallado interpolando entre las curvas (rectas) de nivel $\bar{z} = \text{cte}$.

En la Figura 4 se muestra el modelo V.76 en el plano $\log \frac{q}{Q}$ vs. $\log \tau_H$ para $C_K = 10$ mg/m³ (permisible) y para $C_K = 20$ mg/m³ (peligroso o excesivo).-

La elección del C_K está llena de connotaciones. La eutrofización es un fenómeno gradual. En muchos casos se considera deseable cierto grado de eutrofización porque aumenta la producción de peces. En el caso del N.H., por las características -- que lo distinguen, así como las de la zona que lo rodean y, en fin, por sus tradiciones turísticas y culturales, nos inclinamos por límites muy estrictos.-

Una consecuencia directa del modelo V.76 fué que a partir de -- él se comenzó a plotear la clorofila, la profundidad Secchi y el abatimiento de oxígeno hipolimnético en función de :

$$\frac{q}{Q} = \frac{1}{1 + \sqrt{\tau_H \text{ (años)}}$$

Ejemplo de estas correlaciones se muestran en Figuras 5, 6 y - 7 (2).-

3.1.8 - APLICACION AL LAGO NAHUEL HUAPI

3.1.8.1 Datos del Lago

La poca densidad de información reunida sobre el estado trófico del N.H. y los propios límites de este estudio, dificulta -- la tarea tanto de diagnóstico como de pronóstico, relativizando sus resultados. Dentro de ese marco, el trabajo de R. Quiros (9) es de gran valor y mucha utilidad. Otras de las fuentes utilizadas se citan en (10) y (11).-

Datos Morfológicos

Volúmen.....	87.449 Hm3
Superficie.....	557 Km2
Altura s/n.m.....	764 m.
Profundidad media.....	157 m.
Profundidad máxima.....	438 m.
Relación profundidad/ \sqrt{A}	1/53,8
Longitud línea media.....	74,4 Km
Ancho: al SE.....	7,4 Km
Ancho: en Nariz del Diablo incluso I.Victoria...	10,2 Km
Longitud de costa.....	357,35 Km
Desarrollo de costa: $357,35/2/\sqrt{\pi 557}$	4,27

Profundidad	% de Superficie
0 a 25 m	8,13
25 a 100 m	21,71
100 a 200 m	30,38
200 a 300 m	33,90
> 300 m	---5,83---
	100,00

Datos Hidrológicos

Superficie de cuenca	2.758 Km2
Caudal módulo	224 m3/seg.
Caudal medio anual máximo	326 m3/seg.
Caudal medio anual mínimo	134 m3/seg.
Crecida máxima posible	5.195 m3/seg.

Datos Limnológicos

Temperatura en °C (ver 10)

M e s	Tem. de Superficie			Tem. de Fondo		
	1927	1928	1930	1927	1928	1930
Enero	----	12,5	----	----	7,10	----
Febrero	----	----	----	----	----	----
Marzo	----	----	----	----	----	----
Abril	----	----	14,6	----	----	7,3
Mayo	----	9,2	----	----	7,55	----
Junio	----	----	----	----	----	----
Julio	----	7,42	----	----	7,45	----
Agosto	----	7,25	----	----	7,2	----
Setiembre	----	9,3	9,3	----	7,05	7,35
Octubre	9,1	10,05	10,3	7,2	7,1	7,35
Noviembre	----	----	13,3	----	----	7,4
Diciembre	15,8	----	----	7,3	----	----

Ciclo Anual:

Julio/Agosto: Período de circulación invernal - Homoterma.

Setiembre: Se inicia la estratificación térmica en las capas - superiores.

Octubre/Noviembre: Estancamiento de primavera. La masa está es tratificada. La termoclina es pobre y peque ña. No hay epilimnion.

Diciembre: Preparación del estancamiento de verano.

Enero/Febrero: Estancamiento de verano.

Abril/Mayo: Preparación del período de circulación invernal.

Estado de nutrición en el verano tardío de 1984 (9):

Profundidad Secchi.....	12,5 m.
Clorofila total.....	0,41 mg/m3
Fósforo total.....	3,8 mg/m3
Nitrógeno Kjeldahl.....	15 mg/m3
Color escala cloro platinado de cobalto.....	5
$\tau_H = 87.449 \times 10^6 / 224 / 86.400 / 365$	12,4 años
$1 + \sqrt{\tau_H}$	4,52
Carga hidráulica superficial.....	12,7 m/año

Todos estos datos son bastante consistentes, salvo los de nutrición, que se requerirían sistemáticos.-

3.1.8.2 Carga de Fósforo

Se desarrolla a continuación el cálculo de los aportes anuales de P.

a) Debidos al Terreno

. Aporte específico: 0,01 g P/m2/año

. Superficie de cuenca: 2.758 Km2

$$q_1 = 0,01 \text{ g P/m}^2/\text{año} \times 2.758 \times 10^6 \times 10^{-3} \text{ Kg/q} = \\ = 27.580 \text{ Kg P/año}$$

b) Debido a la Precipitación Directa

. Aporte específico: 0,01 g P/m2/año

. Superficie del lago: 557 Km2

$$q_2 = 0,01 \times 557 \times 10^6 \times 10^{-3} = 5.570 \text{ Kg P/año}$$

c) Debido a Efluentes Domésticos de S.C. de Bariloche

. Aporte específico: 2,00 g/hab/día

Población estable (Censo 1980) 41.302 hab.
 Turistas entrados 350.000 tur/año
 Tiempo medio de permanencia 4 días
 Población media anual (1980)

$$41.302 + \frac{350.000 \times 4}{365} = \dots\dots\dots 45.137 \text{ hab.}$$

$$q_3^{1980} = 45.137 \times 2 \times 10^{-3} \times 365 = \dots\dots\dots 32.950 \text{ Kg/año}$$

$$* q_3^{2000} = 128.000 \times 2 \times 10^{-3} \times 365 = \dots\dots\dots 93.440 \text{ Kg/año}$$

$$* q_3^{2010} = 184.000 \times 2 \times 10^{-3} \times 365 = \dots\dots\dots 134.320 \text{ Kg/año}$$

$$* q_3^{2020} = 263.000 \times 2 \times 10^{-3} \times 365 = \dots\dots\dots 191.990 \text{ Kg/año}$$

d) Debido a efluentes domésticos de la población dispersa

Tanto en las márgenes rionegrinas como en las neuquinas, la población radicada en sus márgenes no es despreciable y su crecimiento está vinculado al de S.C. de B.. El término en concepto de aporte neto de P al lago se considerará igual - al 10% del calculado en c).-

* Se aceptan las hipótesis de crecimiento del Anteproyecto Preliminar.

e) Carga anual de fósforo sin tratamiento, en Kg P/año

Año	Terreno	Precipitación	S.C.de B.	Pob.Disp.	Total
1980	27.580	5.570	32.950	3.295	69.395
2000	27.580	5.570	93.440	9.344	135.934
2010	27.580	5.570	134.320	13.432	180.902
2020	27.580	5.570	191.990	19.199	244.339

- f) Carga anual de fósforo con tratamiento de desfosfatación del efluente cloacal, en Kg P/año.-

Se considerará que el 75% del fósforo producido por la población conectada al sistema es eliminado con el tratamiento de sus desagües cloacales

Año	Población Conectada	Fósforo * Eliminado	Carga sin Tratamiento	Carga con Tratamiento
2000	80.000	43.800	135.934	92.134
2010	160.000	87.600	180.902	93.302
2020	236.700	129.593	244.339	114.746

* Ejemplo: $0,75 \times 2 \times 80.000 \times 10^{-3} \times 365 = 43.800 \text{ Kg/año}$

3.1.8.3 Representación en el Gráfico del Modelo V.1975

Considerando la totalidad de la superficie del lago y el módulo del Río Limay, la abcisa resulta:

$$\frac{Q}{A} = \frac{224 \text{ m}^3/\text{seg} \times 86.4000 \times 365}{557 \text{ Km}^2 \times 10^6} = 12,7 \text{ m/año}$$

y las ordenadas, en gr P/m²/año:

Año	Carga Másica Superficial de P	
	Sin Tratamiento	Con Tratamiento
1980	0,125	-----
2000	0,244	0,165
2010	0,325	0,168
2020	0,439	0,206

Ejemplo: $\frac{69.395 \times 10^3}{557 \times 10^6} = 0,125$

Se representa en la Figura 8 conjuntamente con las curvas de nivel C = 10 y 20 mg/m³.



3.1.8.4 Interpretación según el Modelo V.76

Aplicando la ecuación $C = \frac{q}{Q} \frac{1}{(1 + \sqrt{t_H})}$ podemos hallar la concentración de P que es dable esperar en el lago en las distintas condiciones de carga:

A ñ o	Concentración de P en mg/m3	
	Sin Tratamiento	Con Tratamiento
1980	2,17	----
2000	4,25	2,88
2010	5,66	2,92
2020	7,65	3,59

Ejemplo: $\frac{69.395 \text{ Kg/año} \times 10^6}{224 \times 86.400 \times 365} \frac{1}{1 + \sqrt{12,4}} = 2,17 \text{ Kg/m}^3$

En la Figura 9 se representan los valores tabulados y las curvas de nivel para C = 5, 10 y 20 mg/m3.

3.1.8.5 Niveles de Clorofila, Profundidad Secchi y Tasa de Abatimiento de Oxígeno Hipolimnético - Resumen

Entrando con las concentraciones de P calculadas en el apartado anterior, en los gráficos de correlación de Jones y Lee se obtiene el siguiente cuadro de valores de los principales parámetros de eutrofización:

PARAMETROS INDICADORES DE EUTROFIZACION

P A R A M E T R O	Sin Tratamiento				Con Tratamiento			
	1980	2000	2010	2020	2000	2010	2020	2020
Fósforo (mg/m3)								
	2,17 (1)	4,25	5,66 *	7,65 *	2,88	2,92	3,59	
Clorofila "A" (mg/m3)								
Lím. Superior 95%	3,3	5	6	9	4,1	4,2	5,1	
Valor Probable	0,81 (2)	1,3	1,9	2,1	1,00	1,05	1,10	
Lím. Inferior 95%	0,205	0,40	0,48	0,61	0,28	0,285	0,32	
Profundidad Secchi (m)								
Lím. Superior 95%	21	17	16	12	20	19	18	
Valor Probable	8,0 (3)	5,8	5,1	4,3	7,0	6,8	6,05	
Lím. Inferior 95%	2,9	2,0	1,95	1,8	2,4	2,35	2,15	
Abatimiento de Oxígeno Hipolimnético								
Lím. Superior 95%	0,41	0,58	0,61	0,71	0,5	0,5	0,51	
Valor Probable	0,11	0,18	0,20	0,22	0,115	0,116	0,140	
Lím. Inferior 95%	0,032	0,05	0,06	0,07	0,0402	0,0404	0,042	

(1) Medido por Quiros en Febrero/marzo de 1984: 3,8 mg/m3

(2) Medido por Quiros en Febrero/marzo de 1984: 0,41 mg/m3

(3) Medido por Quiros en Febrero/marzo de 1984: 12,5 mg/m3

(*) Valor Excesivo s/criterio de Rosech-Cullen-Bek

(**) Valor Excesivo s/criterio de Dobson

3.1.9 - CONCLUSIONES

En su estado actual, el Lago N.H. se ubica bastante cómodamente en la categoría de oligotrófico según los criterios habitualmente utilizados para la clasificación de los lagos. No obstante, evoluciona desfavorablemente. Aún de mantenerse en el nivel actual la carga de nutrientes, la concentración de clorofila llegaría a 1 mg/m³ y la profundidad Secchi se estacionaría en un valor inferior a 8 m..-

Pero aceptando hipótesis razonables de crecimiento demográfico se concluye que, de no adoptarse precauciones, la salud del lago empeoraría con ritmo sostenido aunque no dramático, llegándose a convertir, en la década del 2020, en un lago moderadamente mesotrófico.-

Este pronóstico considera a todo el cuerpo, en su conjunto, como una unidad. Pero así como la profundidad y el tiempo de retención son factores a favor, así también en las costas, de desembocaduras y fiordos de poca profundidad -particularmente - aquellos sometidos a los efectos antrópicos o culturales- no están alcanzados por las anteriores conclusiones y es así como en esos lugares se aprecia ya proliferación de fitoplankton y de macrofitas, es decir, signos de eutrofización parcial o zonal.-

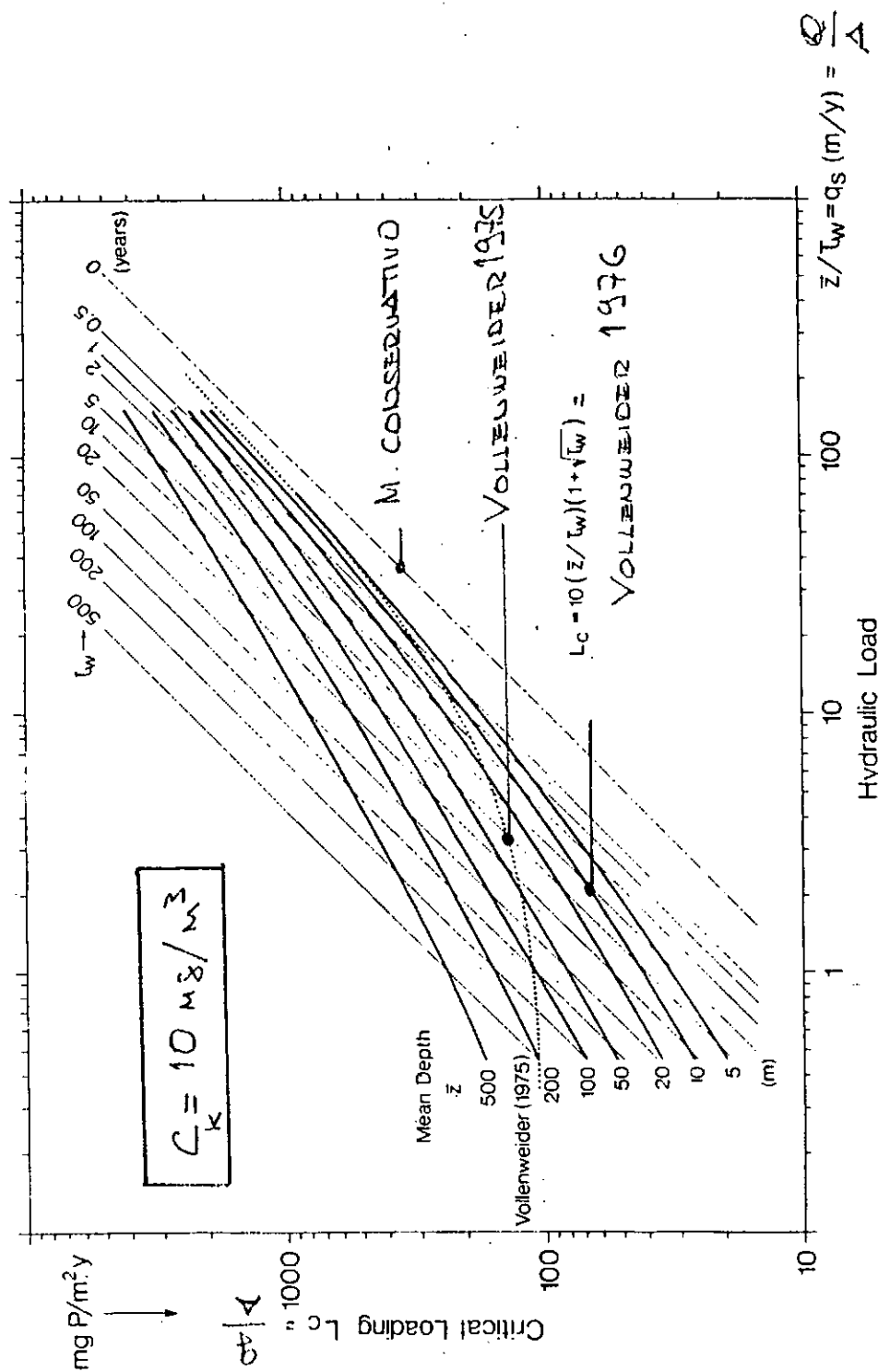


Fig. 3 Representación conjunta de los modelos Vollenweider - OECD (4).

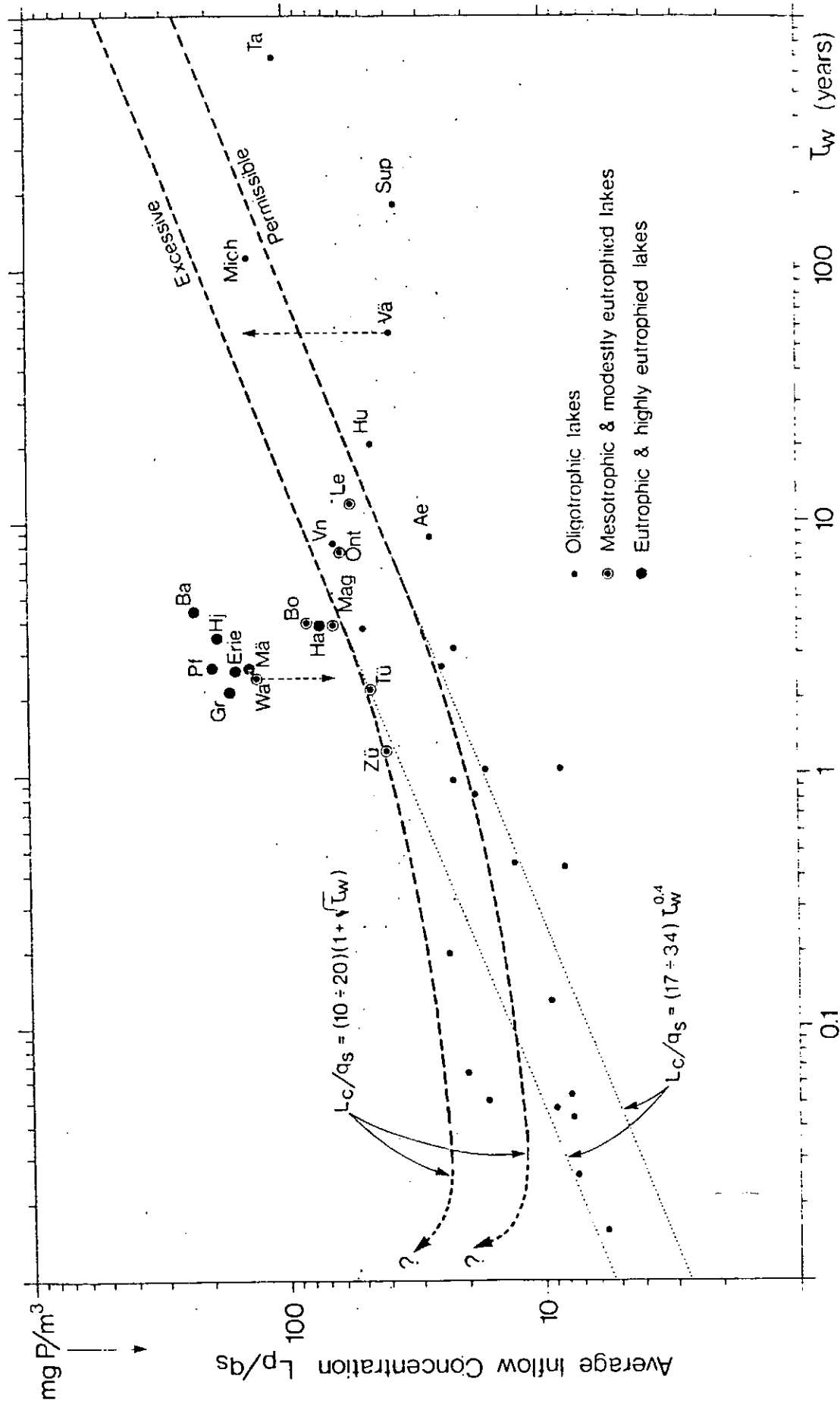


Fig. 4 Representación gráfica del modelo Vollenweider 1976 y de los estados de algunos ejemplos conocidos (4).

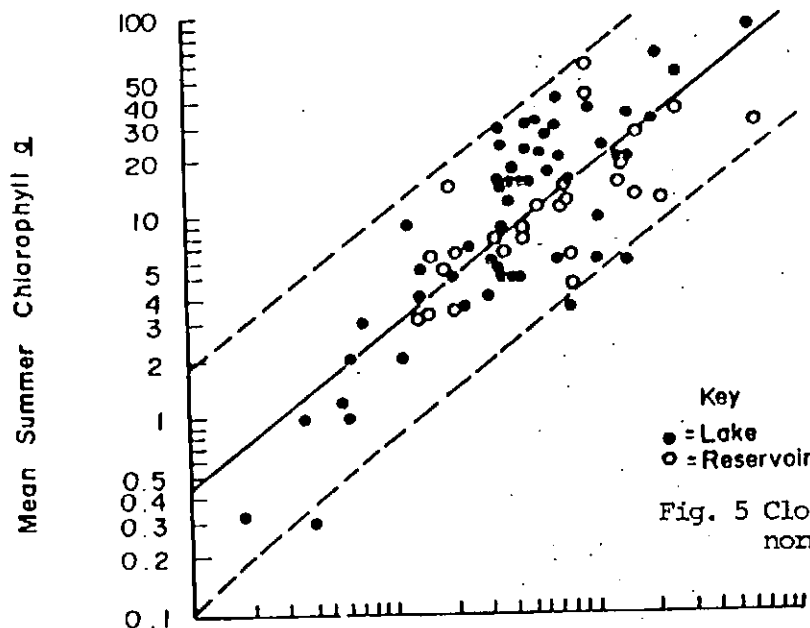


Fig. 5 Clorofila "a" vs. carga anual normalizada de fósforo

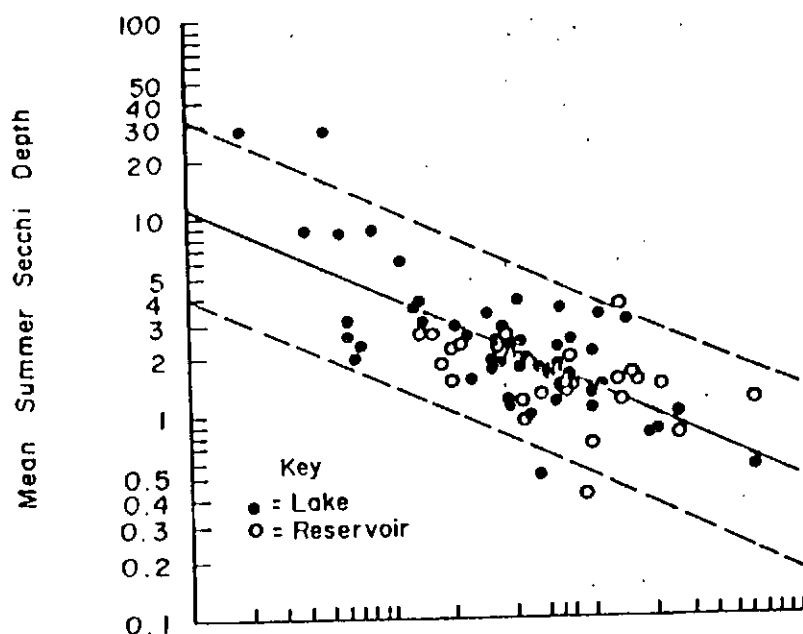
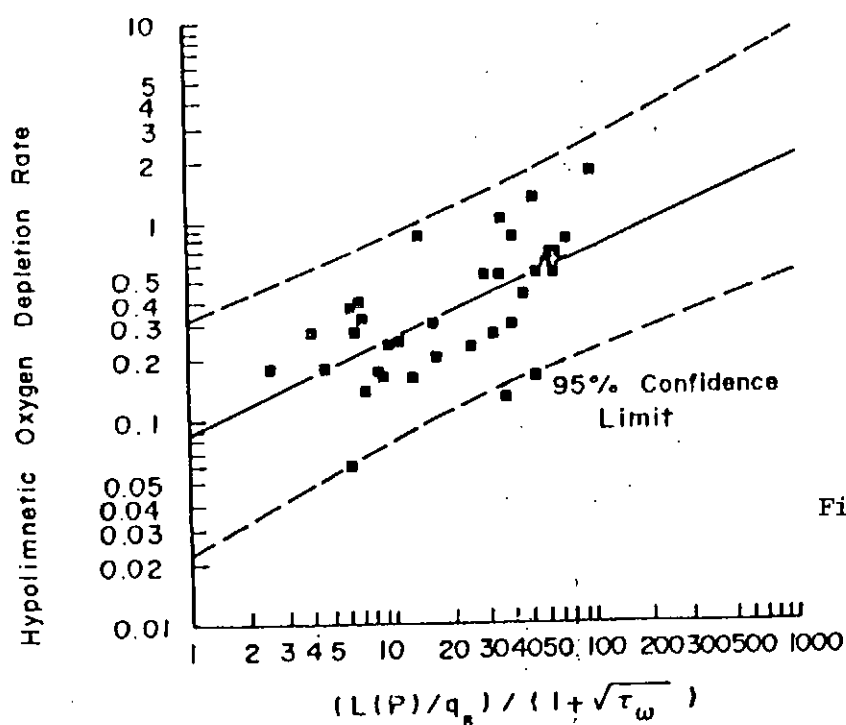


Fig. 6 Profundidad Secchi vs. carga anual normalizada de fósforo



KEY

$L(P)$ = areal annual phosphorus load
 ($\text{mg P m}^{-2} \text{ yr}^{-1}$)

q_s = mean depth \div
 hydraulic residence
 time = Z/τ_w
 (m yr^{-1})

τ_w = hydraulic residence
 time (yr)

Fig. 7 Abatimiento de oxígeno hipolimnético vs. carga anual de fósforo

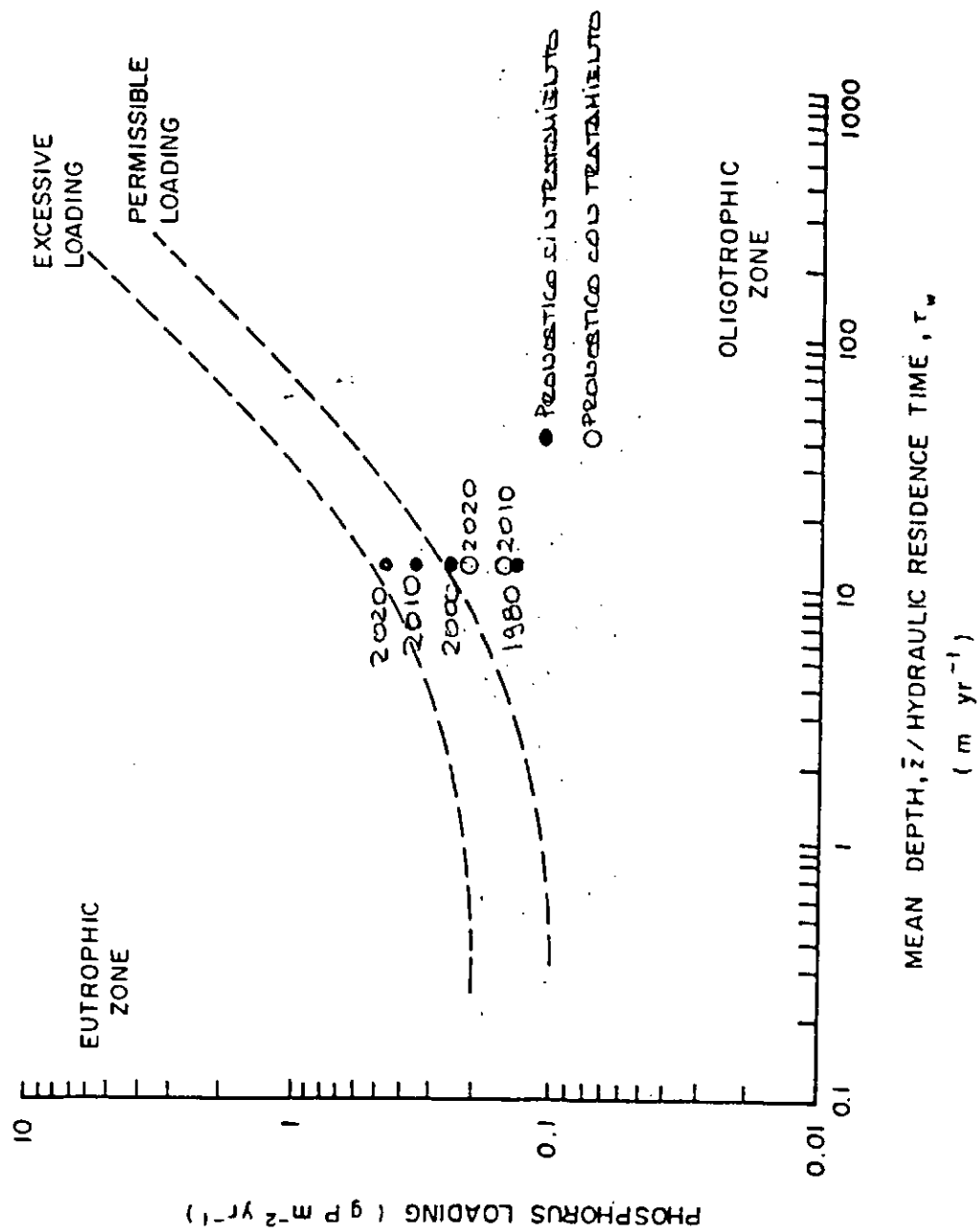


Fig. 8 Pronóstico del Lago Nahuel Huapi según modelo Vollenweider, 1975.

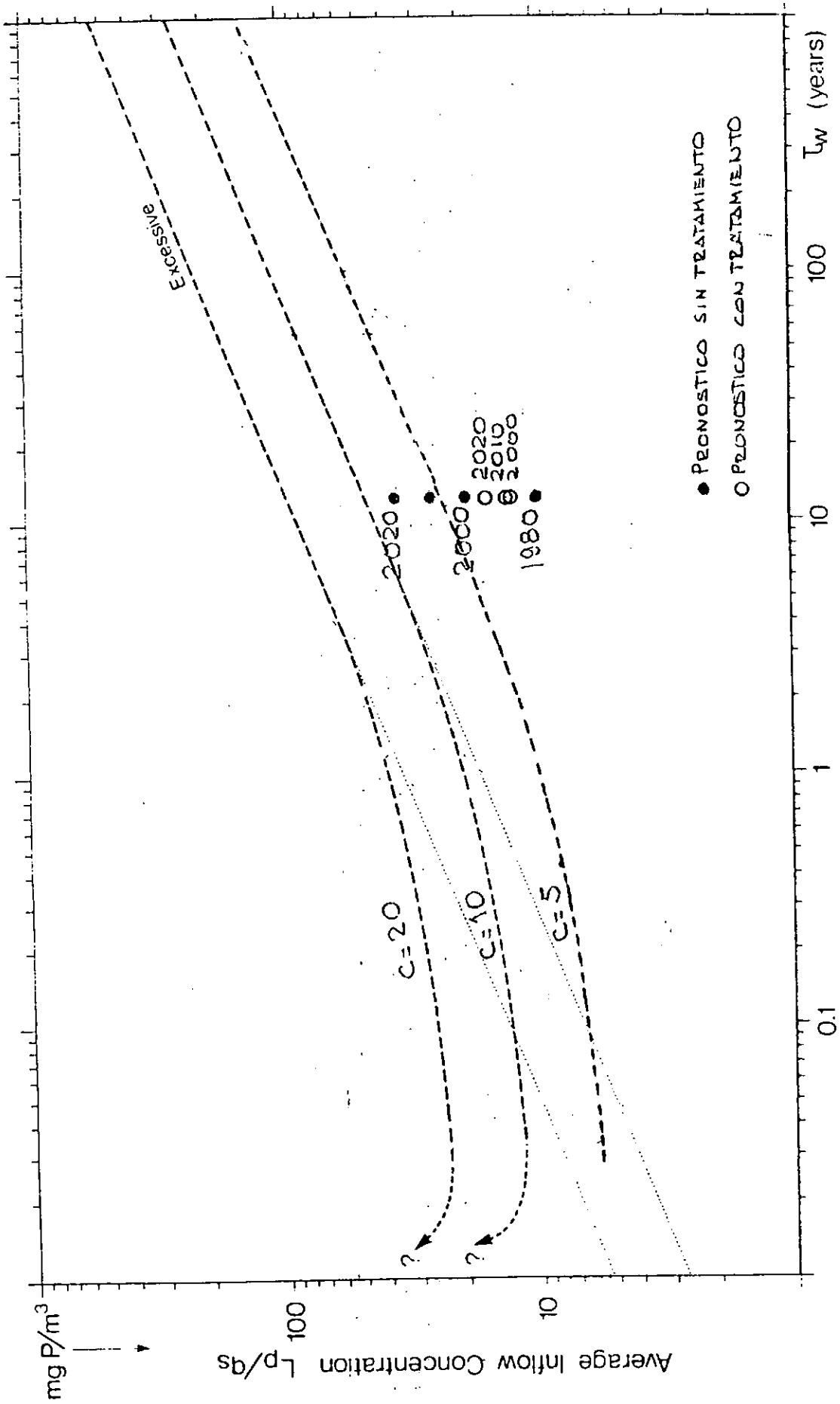


Fig. 9 Pronóstico del Lago Nahuel Huapi, según modelo Vollenweider 1976 (4).

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A PHOSPHORUS RESIDENCE TIME MODEL: THEORY AND APPLICATION

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Chemical residence time has been used as a basis for modeling the chemical content of the ocean for some time (Barth, 1952). Recently, Vollenweider (1969) and other workers have used what is the chemical residence time approach to model the rate of recovery of several lakes following pollution abatement. The purpose of this paper is to relate the theoretical basis for the chemical residence time model, as it applies to phosphorus, discuss the limits, capabilities and applications of the model.

DEVELOPMENT OF THE MODEL

In some early attempts to model chemical concentrations in lakes it was assumed that from the standpoint of long term trends, a lake may be likened to a completely mixed reactor subjected to continual and constant chemical influx. Losses from the reactor occur only through the outlet. Continuity considerations then give

$$V \frac{dc}{dt} = Qc_i - Qc \quad (1)$$

where V is the volume of the lake, L^3 ; Q is the volumetric flow rate, L^3T^{-1} ; c_i is the constant influx concentration, ML^{-3} and c is the concentration in the lake, ML^{-3} . Integrating and applying the boundary condition that at time = 0, $c = c_0$ yields

$$c = c_i + (c_0 - c_i)e^{-t/R_w} \quad (2)$$

where: $R_w = VQ^{-1}$, the hydraulic residence time. Equation (2) shows that following a decrease (or increase) in the chemical influx, steady-state conditions are approached exponentially in accordance with the hydraulic residence time of the basin and that three hydraulic residence times would be required for a lake to reach 95% of its new steady-state concentration. Thus, based on considerations of this type, it has been estimated that 90 yr would be required for Lake Michigan to achieve a 95% response to a decrease in phosphorus input (Ramey, 1967).

Although the hydraulic residence time model may be applicable for conservative elements, it has serious deficiencies when nonconservative elements, such as phosphorus, are considered. The model has two characteristics which should be noted: the steady-

state concentration in the lake is identical to the input concentration, and the losses from the lake occur only through the outlet. Neither of these conditions apply to many lakes. Mean annual concentrations in lakes are often much lower than mean annual input concentrations, and the major loss of phosphorus frequently results from deposition to the sediments, not from discharge through the outlet.

For non-conservative elements, the validity of the model is improved if internal losses as well as outwash losses are taken into account in the mass balance. Thus, equation (1) must be modified to account for the reactivity of the element in the lake. Studies have shown that the sediments of lakes act as sinks or traps for phosphorus. Although there is some release of phosphorus from sediments, the net flux of phosphorus over an annual cycle is to the sediments. This follows from the fact that a significant portion of the organisms that settle onto the bottom are refractory so that regeneration of the phosphorus upon mineralization is less than 100 percent (Jewell, 1971).

To account for internal losses, equation (1) is modified to

$$V \frac{dc}{dt} = Qc_i - Qc - kcV \quad (3)$$

when k is the internal loss rate constant, T^{-1} .

The rate of internal loss is written as a first order reaction, in which the loss is directly proportional to the mean content of the lake. This assumption is reasonable, at least as a first approximation (see Vollenweider, 1969). In the case of phosphorus, k accounts for the net loss of P to the sediments and kCV is the net phosphorus sedimentation rate. The above modification could also be adapted to nutrients such as nitrogen and carbon, but the relationship would necessarily be more complex since a gas phase must be considered in the aqueous chemistry of nitrogen and carbon. Equation (3) may be potentially useful as a model for the silica cycle in natural waters, however.

In a stratified lake the possibility that the surface layer may contain different amounts of a non-conservative chemical than the bottom waters due to biological, chemical or physical processes must also be considered. For example, phosphorus concentrations in the surface water of a stratified lake during certain

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times of the year may be only a fraction of the average concentration over the whole lake on account of its removal from the surface layer by biological and chemical processes. Thus, the outwash concentration during those periods of the year when the lake is not well mixed may be significantly different than the average concentration over the whole lake. In many situations, this effect will be of minor significance when an annual cycle is considered, particularly when the hydraulic residence time is long in relation to the period of stratification. However, in those situations when stratification must be taken into account, Equation (3) may be modified in the following manner:

$$V \frac{dc}{dt} = Qc_i - Q\alpha c - kVc \quad (4)$$

where α = dimensionless proportionality factor that relates the mean annual outwash or surface water concentration to the mean annual concentration over the whole lake. Consequently, it is assumed that the average annual outwash concentration is directly related to the annual concentration over the whole lake by a constant factor. Since the total phosphorus concentration in the epilimnion of most lakes is less than the hypolimnion concentration, α will generally be less than one. For example, over the last two years the average concentration of total phosphorus in the surface waters of Lake Mendota, Madison, Wisconsin, was about 70% of the average concentration for the entire lake. Thus, α would have a value of about 0.7 for lake Mendota. For lakes which do not stratify, α would be equal to unity. Note that in the above models, it has been assumed that the water balance is such that yearly inflow equals yearly outflow. Dingsman and Johnson (1971) have more properly equated the water balance equation with the mass balance equation, but for the purposes of this paper the above simplification will be utilized.

RESPONSE TO CHANGES IN NUTRIENT INFLUX

Of special interest to those concerned with lake management is the rate of improvement which might be expected as a result of reduced nutrient loadings (or vice versa) to a lake. For example, how fast and to what extent will a lake respond to a step-change in the phosphorus influx, such as would result from improved wastewater treatment? Equation 4 may be applied directly to gain insight regarding the anticipated response. Rearranging Equation (4) gives

$$dc + \frac{(Q\alpha + kV)}{V} c dt = \frac{Q}{V} c_i dt \quad (5)$$

By substitution, this can be simplified to:

$$dc + \frac{1}{R_p} c dt = \frac{1}{R_w} c_i dt \quad (6)$$

where

$$R_p = \frac{V}{Q\alpha + kV},$$

which is the phosphorus residence time for the lake. Equation (6) may be integrated directly, and if at $t = 0$, $c = c_0$, then

$$c = c_i \frac{R_p}{R_w} - \left(c_i \frac{R_p}{R_w} - c_0 \right) e^{-t/R_p} \quad (7)$$

The steady-state concentration in the lake, c_∞ , for this model is not identical to the input concentration, but differs by a factor which is the ratio of the phosphorus and hydraulic residence times.

$$c_\infty = c_i \frac{R_p}{R_w} \quad (8)$$

Equation (7) may then be expressed in terms of the ultimate steady-state concentration by

$$c = c_\infty - (c_\infty - c_0) e^{-t/R_p} \quad (9)$$

or

$$\frac{(c - c_\infty)}{(c_0 - c_\infty)} = e^{-t/R_p} \quad (9a)$$

Equations (8) and (9) can be used to estimate the gross response of a lake to a step reduction in the phosphorus influx. If the lake was at steady-state prior to the input reduction, it can be seen from Equation (8) that, ultimately, the mean content of the lake will be reduced in direct proportion to the change in the influx, i.e. a 50% reduction in the input will result in a 50% reduction in the mean content of the lake. Equation (9) shows that the new steady-state concentration will be approached exponentially as a function of the phosphorus residence time (Fig. 1). The time required to reach 50% of the expected change is $0.69 R_p$; 95% of the expected change will be reached in a period equal to three phosphorus residence times.

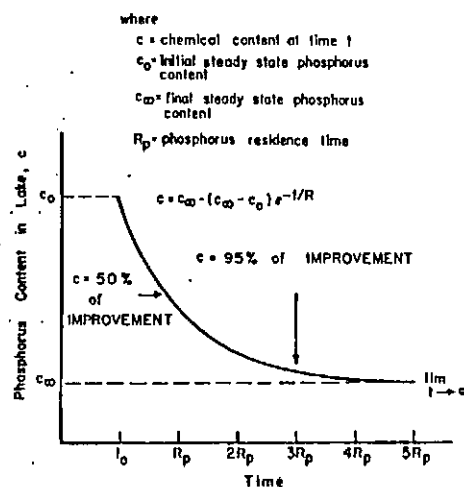


Fig. 1. Rate of recovery of a lake following a step-change reduction of the phosphorus influx.

DETERMINATION OF R_p

For steady-state conditions, R_p can be determined most conveniently from equation (8).

$$R_p = \frac{R_w c_{ss}}{c_i} = \frac{V c_{ss}}{Q c_i} \quad (10)$$

Equation (10) shows that the phosphorus residence time for a lake which is in equilibrium with its input can be determined if the mean annual content, $V c_{ss}$, is divided by the annual input to the lake, $Q c_i$. If it is assumed that the lake is at steady-state prior to the input reduction, then equation (10) may be used to compute R_p using data obtained before the input reduction. Since R_p is a function only of V , Q , k and α , R_p is not influenced by changes in the input concentration.

Once the phosphorus residence time is known, the rate of response to phosphorus abatement (or pollution) can be predicted from the model. The phosphorus residence time accounts for the overall sedimentation process as well as effects of stratification.

The use of chemical residence time circumvents determining the values of k and α . Alternatively, if k and α are known or estimated (as for example from phosphorus sedimentation rates) and V and Q are known, R_p can be calculated directly. If R_p is known and the steady-state phosphorus content is known, then the input of phosphorus could be estimated by solving equation (10). This would be a possible method of roughly estimating the loading of phosphorus to a lake.

It is interesting to note the relationship between the phosphorus residence time and the hydraulic residence time. From their basic definitions, it can be shown that

$$R_p = \frac{1}{\frac{\alpha}{R_w} + k} \quad (11)$$

If the lake under consideration is well mixed ($\alpha = 1$), equation (11) shows that, for a given k , the larger the hydraulic residence time becomes the less effect it has on the phosphorus residence time. Thus, as R_w becomes large, R_p approaches $1/k$. In contrast, as the water residence time becomes smaller, the chemical reactivity becomes less important and R_p approaches the value of R_w . Also, when k is small R_p approaches R_w , which indicates that for this condition phosphorus acts like a conservative substance. Plots of equation (11) for various values of k are given in Fig. 2.

STEADY-STATE ASSUMPTIONS

A basic assumption of the model is that the influx of phosphorus to a lake is constant. Realistically, the annual influx of phosphorus to a lake may differ somewhat from year to year due to natural random variations in, for example, the annual amount of

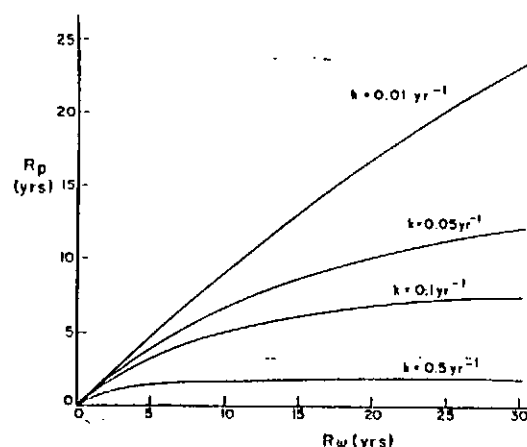


Fig. 2. Relationship between R_p and R_w , $1/[(\alpha/R_w) + k]$ for hypothetical values of k ($\alpha = 1$).

runoff or precipitation. Also, the influx may be changing continuously for some lakes, reflecting, for example, the steadily increasing use of phosphates by man. If the input of a chemical is continuously changing, the phosphorus concentration within the lake will not reach a steady-state concentration with the influx, but will lag continuously from the steady-state condition. If large, this lag could lead to significant errors in estimating R_p and the extent to which in-lake concentration will change as a result of reduced influx.

To estimate the potential significance of continually increasing input concentrations, equation (6) was solved for the condition that the input concentration was a linear function of time rather than a constant. The effect of lag was then evaluated by comparing at a given time the concentration which would occur in the lake from a continuously increasing input to the steady-state concentration which would result if the input concentration was constant. For the conditions described, it can be shown that mean annual concentrations would lag the predicted steady-state value by an amount given by

$$\text{lag} = \Delta c_i \frac{R_p^2}{R_w} \quad (12)$$

where Δc_i = the annual rate of increase in the input concentration, MT^{-1} . Since R_p is normally smaller than R_w , often by a considerable amount, the error in concentration which results from steady-state assumptions is always smaller than the product of R_p and the yearly incremental increase in the input concentration. Since this annual increase is often small, long term temporal trends can be ignored in many practical situations.

CALCULATIONS OF R_p

Although the model was derived based on the mean annual phosphorus content (obtained from the average of systematic measurements over a whole year), the mean content at spring turnover or during the winter period might be used as the basis for the

annual total phosphorus content. During these periods, lakes are generally well mixed (except under ice cover) and biological productivity is often at an annual minimum. The use of concentrations obtained at the time of spring turnover to forecast whether a lake may be expected to produce excessive growths of algae or other aquatic plants is well known (Sawyer, 1947; Vollenweider, 1968). It is, therefore, reasonable that winter or spring-turnover phosphorus levels are proportional to the annual loading to the lake (see Vollenweider, 1968 for a discussion of this relationship). One obvious advantage of using winter or spring turnover measurements would be that one sample could be used to characterize the entire lake.

However, caution should be exercised when using spring turnover concentrations. It was observed at ice out (late April) of 1971 on Lake Mendota, that the total phosphorus content decreased by over 50% compared to the content measured just prior to ice out. The cause of this decrease was deduced to have been caused by a bloom of diatoms (the lake actually appeared brown to observers along the shore). The diatoms took up the available phosphorus (most of the total) and then settled to the bottom; the process removed a large portion of phosphorus from the water column. Phosphorus was subsequently released from the diatoms as they decomposed on the lake bottom. By June, when summer stratification first developed, the phosphorus content had increased to near the level measured before ice out and was again available for summer algae blooms. Although this may have been an unusual occurrence, such a possibility must be considered when measuring the spring turnover phosphorus level.

The phosphorus residence time has been calculated for a number of lakes where sufficient data is available. The results are given in Table 1 along with the hydraulic residence times of the respective lakes. Table 1 shows that in all cases the phosphorus residence time is less than the hydraulic residence time, indicating the lakes are acting as a sink for phosphorus. It should be noted that although the chemical content of lakes, either the mean annual content or the mean vernal content, may be determined with reasonable reliability, the rate of phosphorus influx is often much less precisely known, especially when the influx is estimated from land use patterns and generalizations from the literature (see Sonzogni and Lee, 1974b). Nevertheless, the fact that all phosphorus residence times listed in Table 1 are less than the hydraulic residence times, even considering that they are crude estimates, gives strong evidence that the response of a lake to phosphorus abatement will be more rapid than that predicted from the hydraulic residence time.

TESTS OF THE MODEL

One of the best examples of a lake which has responded to a decreased nutrient flux is Lake Washing-

Table 1. Estimated mean residence times for water and phosphorus for various lakes

Lake	Water Residence Time (yr)	Phosphorus Residence Time (yr)	Source
Washington	3.2	0.8	Megard (1971)
Hinnetonka	25	0.8	Megard (1971)
Sebasticook	3.5	1.4	Megard (1971)
Norrviken	0.6	0.3	Megard (1971)
Clear	6	2	Megard (1971)
Mendota	4.5	0.8	Sonzogni and Lee (1974)
Hichigan	30	6	Lee (1972)
Erie	2.6	0.34	Dabron (1973)
Ontario	7.6	3.2	Dabron (1973)
Agerisee	0.7	2.3	Vollenweider (1969) ^a
Turlersee	3.2	0.7	Vollenweider (1969)
Hellwilersee	3.8	2.7	Vollenweider (1969)
Rodensee - Obersee	4.9	3.8	Vollenweider (1969)
Fraßlitzersee	2.6	1.7	Vollenweider (1969)
Zürichsee - Untersee	1.5	1.2	Vollenweider (1969)
Greifensee	2.0	1.4	Vollenweider (1969)
Baldiggersee	4.5	3.1	Vollenweider (1969)

^a as calculated from Vollenweider (1969) based on mean vernal content instead of mean annual content.

ton (Edmondson, 1969; 1971). Between 1963 and 1968 sewage effluents were diverted from entering Lake Washington so that the annual input of phosphorus was reduced by about 50%. The response of the lake to the decreased nutrient influx was both prompt and sensitive. During the winter of 1969 the concentration of soluble orthophosphorus was only 28% of the winter concentration before diversion. Megard (1971) has compared the actual rate at which the phosphorus content decreased following diversion with that predicted from the phosphorus residence time model. The observed rate of decrease was parallel to the predicted rate and the 1969 measured concentration was similar to the predicted concentration for 1969. Megard found that the observed rate at which the phosphorus content decreased was parallel to the predicted rate, despite the fact that his model assumed that the complete diversion was done in one step whereas the diversion was actually completed in stages over a few years. Furthermore, he found that the measured mean annual concentration in 1969 was very close to the new steady-state concentration predicted by the model. In general, the Lake Washington case provides a reasonably successful test of the model.

The 1958 diversion of sewage effluent from Lake Waubesa and Lake Kegonsa, the third and fourth lakes, respectively, in the Yahara chain of lakes located near Madison, Wisconsin, provide other examples of the response of lakes to a decreased phosphorus loading. It is of interest to compare, using the limited data available, the observed rate at which the lakes responded to the diversion with that predicted based on the phosphorus residence time model (see Sonzogni and Lee, 1974a).

The hydraulic residence times of these shallow, well mixed lakes were of the order of a few months so

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that, as discussed previously, the phosphorus residence time may be assumed to be about the same as the hydraulic residence time.

For Lake Waubesa, the theoretical hydraulic residence time (and, consequently, R_p) was about 0.22 yr after diversion (Sonzogni and Lee, 1974a). If sewage effluent is taken to have contributed 90% of soluble inorganic phosphorus (Sawyer *et al.*, 1944), then, according to the exponential recovery model, in less than a year after the December, 1958, sewage diversion the equilibrium phosphorus content should have been reduced by nearly 90%. Unfortunately, there is no information on what the soluble inorganic phosphorus was the winter immediately prior to the diversion. However, compared to 1950 and earlier the mean winter concentration 1 yr after diversion was reduced by over 75%.

Lake Kegonsa, which had a post diversion hydraulic residence time of about 0.31 yr, was thought to receive most of its soluble inorganic phosphorus input from Lake Waubesa (Sawyer *et al.*, 1943; 1944), so that a 90% reduction in the phosphorus flux to Lake Waubesa should also have resulted in nearly a 90% reduction of the input to Lake Kegonsa. However, Kegonsa's rate of recovery depended not only on its own flushing period but also on that of Lake Waubesa and the reach of the Yahara River between the two lakes (this reach includes a small, shallow lake-like widening of the river). Since the combined theoretical hydraulic residence times is slightly greater than 0.51 yr, Lake Kegonsa should have reached 95% of its new equilibrium phosphorus concentration within two years. This represents an upper figure, since it has been assumed that Lake Kegonsa did not begin to recover until Lake Waubesa had equilibrated with the new input.

The first winter after diversion the data shows a small decrease in the soluble inorganic phosphorus concentration in the outlet of Lake Kegonsa compared to 1950 values, while the second winter after diversion the soluble inorganic phosphorus concentration was reduced by nearly 70%. Again, the lack of data immediately prior to diversion precludes a more accurate assessment of the situation, but it appears that the observed rate at which Lake Kegonsa equilibrated to the decreased chemical flux was close to the rate predicted by the exponential decay model. Thus, both Lake Waubesa and Lake Kegonsa appeared to have recovered at a rate corresponding to that predicted by the model.

Of considerable importance is the fact that the sediments of Lake Washington, Lake Kegonsa and Lake Waubesa did not act as major sources of phosphorus to the overlying waters following diversion. The Zellersee, a European lake, as cited by Vollenweider (1969), is another example of a lake whose sediments were not a source of phosphorus following a reduction of the phosphorus influx. As discussed by Lee (1970), shallow lakes such as Lake Waubesa and Lake Kegonsa which have received very large amounts of

phosphorus should show the greatest overall release of phosphorus from the sediments due to the much higher wind and organism induced mixing between the sediments and the overlying waters. Consequently, the above examples present strong evidence against significant buffering effects of the sediments subsequent to a reduction of the phosphorus income of a lake.

PHOSPHORUS SEDIMENTATION RATE

It is of interest to compare the phosphorus deposition rate calculated from a mass balance [Equation (3)] with the rate estimated from sediment analyses. Taking Lake Mendota as an example, the rate of phosphorus deposition during recent years was estimated from sediment core studies to be 1.19×10^3 mg P m⁻² yr⁻¹ for the deep hole area (23 m) of the lake (Bortleson, 1971). This rate was found to increase toward shallower waters, the maximum rate (1.6 times the rate found for the deep hole area) observed at a twelve meter station. The average phosphorus sedimentation rate over the whole lake was not estimated by Bortleson (1971), but the rate might be expected to fall somewhere between the deep hole and 12 m rate.

At steady-state, equation (4) shows that the phosphorus sedimentation rate (kVc) is equal to the annual input rate (Qci) minus the annual outflow rate (Qzc). Using an average flow rate of 80 mgd for Lake Mendota (McCaskey, 1955), an α value of 0.7 (see previous discussion), and a mean annual phosphorus concentration of 0.12 mg l⁻¹ P (Sonzogni and Lee, 1974a), the phosphorus sedimentation rate is calculated to be 1.35×10^3 mg P m⁻² yr⁻¹. This value is remarkably close to the rate estimated from Bortleson's (1971) sediment core studies.

Megard (1971) made similar calculations for Lake Washington and found excellent agreement between the phosphorus deposition rate as calculated from sediment analysis (1.5×10^3 mg P m⁻² yr⁻¹) and the rate estimated from a materials balance (1.7×10^3 mg P m⁻² yr⁻¹). While the close agreement of phosphorus sedimentation rates as calculated by the two independent methods may be fortuitous in view of the roughness of the data, the results support the basic mass balance relation and indicate that the model is not grossly in error.

UTILIZATION OF THE MODEL FOR LAKE MANAGEMENT

When considering the costly diversion or elimination of nutrient sources to a lake, it is often asked what effect such action will have on the water quality, especially with regard to the phosphorus level in the lake, and how fast this effect will take place. Lake Mendota, the first lake in the Yahara chain of lakes, provides an example of a eutrophic, dimictic lake whose phosphorus income was recently reduced by an estimated 20% (Sonzogni and Lee, 1974a). The effect of this reduction on the phosphorus content

of the lake has not yet been determined, although it is under investigation (Sonzogni, 1973). Because it is established that the sediments of Lake Mendota serve as a sink for phosphorus (Bortleson and Lee, 1972), the time needed to equilibrate to a new phosphorus flux should be more rapid than predicted from the hydraulic residence time (4.5 yr).

The phosphorus residence time for Lake Mendota may be estimated by dividing the mean annual steady-state phosphorus content by the annual phosphorus loading. The mean annual phosphorus content was determined for 1970-1971 (assuming the content at this time was an equilibrium content) from detailed sampling at nearly weekly intervals. The annual phosphorus loading for Lake Mendota was recently estimated by Sonzogni and Lee (1974b). It should be noted that the estimated annual phosphorus loading is only a rough approximation and may vary widely from year to year. Nonetheless, using this data, a mean phosphorus residence time of about 0.9 yr is obtained for Lake Mendota.

Since the change in Q due to the diversion was negligible, the mean phosphorus residence time for Lake Mendota should not have changed after diversion, so that a new steady-state phosphorus content should be reached about 3 years following the start of diversion. Thus, the total phosphorus content is predicted to decrease by about 20% about three years after diversion. The mean annual concentration of total phosphorus would decrease from about 0.12 to 0.10 mg l^{-1} P, or by about 0.02 mg l^{-1} P, as a result of the diversion.

Unfortunately, because of the relatively small decrease in the estimated phosphorus loading as a result of the 1971 diversion, it is doubtful that the predicted change in the mean annual phosphorus content will be seen analytically. Normal year to year variability in the loading from other sources, as well as other factors, may overshadow the effect of the diversion. Thus, the diversion will probably not serve as a significant test case for the model. This is not to say that the 1971 diversion was not of value. On the contrary, the diversion served essentially as a preventive measure to avoid future degradation of the water quality of the lake. This degradation could have occurred if waste water, steadily increasing in amount as a result of a rapidly expanding population on the north and west side of the lake, were to continue to be discharged into the lake.

Considerable attention has recently been focused on the eutrophication of Lake Michigan and the effect of curbing excessive phosphorus enrichment of the lake. Water pollution control agencies of the states bordering on Lake Michigan and the federal government (EPA) established in 1968 an 80% phosphorus removal standard for waste waters discharged into Lake Michigan. The criteria was essentially met as of December, 1972. This action has, according to an estimate recently made by Lee (1972), reduced the annual phosphorus influx to the lake by over 50%.

A question of great importance is the rate at which Lake Michigan will respond to this limitation of the phosphorus influx.

Because the hydraulic residence time of Lake Michigan is on the order of 30 yr, some persons have indicated that it may take nearly 100 yr or longer for Lake Michigan to equilibrate to the reduced phosphorus flux (Baumgartner, as cited by Risley, 1968). Such predictions are based for the most part on the hydraulic residence time model which necessarily assumes phosphorus behaves conservatively. However, in view of the long hydraulic residence time of Lake Michigan, it is felt that it is much more appropriate to use a residence time model based on the chemical reactivity of the element.

Recently, Lee (1972) has made an estimate of the phosphorus residence time for Lake Michigan, and thereby predicted the response of the lake to the reduced loading using the phosphorus residence time model. Using data from the Phosphorus Technical Committee to the Lake Michigan Enforcement Conference (Zar, 1972), a total loading of 18.1 million pounds yr^{-1} of phosphorus was assumed to enter the lake prior to the reduced loading. The steady-state content was calculated by assuming an average phosphorus concentration of 0.01 mg l^{-1} P (Schelske, 1972) over the whole volume of the lake (5×10^{15} l; Hutchinson, 1957). Thus, it is computed that the phosphorus residence time in Lake Michigan is about 6 yr. Therefore, based on the phosphorus residence time model, it would be expected that about three residence times, or 15-20 yr, will be needed to achieve 95% of the expected change in the phosphorus content.

It should be noted that the above rate of recovery assumes that the phosphorus content of Lake Michigan in 1971 (before the decreased loading was accomplished) was in equilibrium with the phosphorus loading to the lake (Lee, 1972). The Phosphorus Technical Committee Report (Zar, 1972) presents data which show that, on the average, the total phosphorus concentration in the water supply intake of the Chicago Water Filtration Plant has been increasing at a significant rate during the past 15 yr, indicative of a small but steady rate of increase of the phosphorus loading. If this data is considered representative of the entire lake, it means that if the annual phosphorus load were to be maintained constant at the 1971 level, the phosphorus concentration would increase for several years before equilibrium is reached. Nevertheless, as was discussed earlier, even if the phosphorus concentrations were tending to increase as a result of a small, steady increase in loading prior to 1972, the assumption of a steady-state condition in 1971 is reasonable within the limits of the data. Moreover, the dramatic drop in the total phosphorus input (approximately 60%) during 1972 would completely overshadow the loading effect prior to 1972. This follows from the fact that the residence time model utilizes an exponential recovery with the greatest

recovery immediately following the input reduction. To be sure, a new steady-state concentration will be reached in much less time than predicted from a hydraulic residence time model.

It should be noted that the phosphorus concentration used in the above calculations may be high for Lake Michigan as a whole. Some of the data on the total phosphorus present in the open waters of the lake have a total phosphorus concentration of less than $0.01 \text{ mg l}^{-1} \text{P}$ (Lee, 1972). Thus, the phosphorus residence time may be less than the 6-yr value computed on the basis of the $0.01 \text{ mg l}^{-1} \text{P}$ value for the mean annual concentration of the lake.

CONCLUSION

The phosphorus residence time model is potentially useful for assessing a variety of lake rehabilitation procedures. Effects of improved waste water treatment may be simulated by reducing the influx concentration, c_i ; the impact of diversion projects can be estimated by altering both c_i and Q ; while in-lake schemes such as alum treatment, artificial destratification or hypolimnetic aeration may be assessed by modifying k . As attempts at lake renewal increase and more is learned about the phosphorus balance in lakes, the model will become more refined. Nonetheless, the phosphorus residence time model described above, though admittedly crude, does provide a simple, easily used and generally realistic basis for predicting the result and rate of many lake renewal endeavors.

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ADVANCES IN DEFINING CRITICAL
LOADING LEVELS FOR PHOSPHORUS
IN LAKE EUTROPHICATION ⁽¹⁾

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Abstract

Models for defining critical loading values for phosphorus, proposed over the last years, are reviewed. Using the concept of relative residence time of phosphorus (i. e. the phosphorus residence time relative to that of water τ_p/τ_w) new loading criteria are developed. Accordingly, for phosphorus controlled lakes, the transition range between oligo- and eutrophy is derived from

$$I_c (\text{mg}/\text{m}^2 \cdot \text{y}) = (10 \text{ to } 20) \cdot q \cdot (1 + \sqrt{\bar{z}/q})$$

applicable over a large variety of lakes of different mean depth \bar{z} and hydraulic load q . These criteria are not in contradiction to previous ones but represent a further refinement in direction of more universal criteria. Further, an attempt has been made to relate phosphorus loading to predicting the average chlorophyll *a* concentrations during the summer growth phase.

1. - INTRODUCTION.

It is now well accepted that eutrophication of lakes depends on excessive discharges of phosphorus and nitrogen to inland waters. This led to the development of what is now called the nutrient loading concept (cf. Vollenweider and Dillon, 1974; Vollenweider, 1975). This concept implies that a quantifiable relationship exists between the amount of nutrients reaching a lake and its trophic degree measurable with some kind of trophic scale index. The call for a trophic scale index evolved from a need generally felt to give better meaning and significance to the classical limnological categories of oligo-, meso- and eutrophy. This question is not entirely solved as yet, but advancement toward the development of a universal scale of this sort is underway by a number of researches.

Meanwhile, further progress has been made toward improved criteria for estimating critical levels for phosphorus loading. This notion applies to lakes in which the production level is controlled by phosphorus, and implies that the trophic nature of a lake may change if discharge of phosphorus exceeds a certain but quantifiable level, which in turn, depends on the limnological characteristics of the body of water in question.

The establishment of such critical levels has undergone several stages in the past. Because of the scanty information available at the time, the first possible estimation of the transition range for critical loading

of phosphorus accounted for the effect of mean depth as the sole reference parameter, giving the following approximation

$$L_c \text{ (mg/m}^2 \cdot \text{y)} \approx (25 \text{ to } 50) \bar{z}^{0.5} \quad (1)$$

(Vollenweider, 1968). Further improvement on this criterion was possible using a simplified mixed reactor model which — in addition to mean depth — included terms for the hydraulic residence time and a sedimentation function (Vollenweider, 1969, 1975; cf. also Dillon and Rigler, 1974; Lorenzen, 1974). At this basis, then, the next level of approximation may be written

$$L_c \text{ (mg/m}^2 \cdot \text{y)} = [P]_c^{sp} \bar{z} (\rho_w \pm \sigma_p) \quad (2)$$

$$\approx [P]_c^{sp} (\bar{z}/\tau_w \pm \bar{z}\sigma_p) \quad (2a)$$

where $[P]_c^{sp}$ is a critical concentration of total phosphorus (mg PT/m³), for simplicity taken at spring overturn, \bar{z} is the mean depth (m), $\rho_w = 1/\tau_w = Q_s/V$ defining the flushing rate per year, and σ_p relates to the sedimentation rate of phosphorus. Common limnological experience suggests that the lower limit of $[P]_c^{sp}$ may safely be assumed to be 10 mg P/m³ with an upper limit not in excess of 20 mg/m³. σ_p , the only unknown in (2), however, could not be estimated independently, but had to be derived from available data indirectly (see below).

Eq. (2a) offers two interpretations:

- a) The critical loading of comparable lakes is directly proportional to their mean depth, and to some extent indirectly proportional to the hydraulic residence time of water (1); in addition, the loading tolerance depends also on the apparent velocity of sedimentation of phosphorus (second term in parenthesis).
- b) Considering the meaning of \bar{z}/τ_w which equals the hydraulic load q_s in m/y per unit surface, however, it appears that mean depth, as an independent parameter, in part is lost. The extent to which this may be true depends on the relationship between \bar{z} and τ_w .

2. — NUMERICAL SOLUTIONS FOR EQ. (2).

According to what has been said above, the development of (2) into an equation useful for estimating critical loadings depends on how σ_p is dealt with. In principle, several ways are open. From the steady state equation underlying (2), one can derive that

(1) The proportionality to \bar{z} relates to the dilution of the incoming phosphorus load and the reciprocal proportionality to τ_w to its likely time of residence in the lake.

$$\sigma_p = \frac{F_p(\bar{z})}{\bar{z} \cdot [\bar{P}]_{\lambda}}$$

where $F(\bar{z})$ represents the flux of P through the horizon at \bar{z} in $\text{mg P/m}^2 \cdot \text{y}$, and $[\bar{P}]_{\lambda}$ represents an average total phosphorus concentration in mg P/m^3 over the column from 0 to \bar{z} . Accordingly (2 a) becomes

$$L_c' (\text{mg/m}^2 \cdot \text{y}) \approx [\text{P}]_c^{\text{sp}} \left(\bar{z}/\tau_w + \frac{F_p(\bar{z})}{[\bar{P}]_{\lambda}} \right) \quad (3)$$

The shortcoming of this equation lies in the scarcity of data available for evaluating $F(\bar{z})/[\bar{P}]_{\lambda}$. Table 1 lists some values calculated for a few Swiss lakes using estimated deep point sedimentation rates and spring overturn concentrations. The few values are in the same order of magnitude; however, it would be premature to generalize these data over a large spectrum of lakes.

Table 1. - Evaluation of $F_p(\bar{z})/[\bar{P}]_{\lambda}^{\text{sp}}$.

	$F_p(\bar{z})$ Sedimentation*) $\text{mg/m}^2 \cdot \text{y}$	$[\bar{P}]_{\lambda}^{\text{sp}}$ Spring Overturn Concentration*) mg/m^3	$F_p(\bar{z})/[\bar{P}]_{\lambda}^{\text{sp}}$ Apparent Sedimenta- tion Velocity m/y
Aegerisee	130	7.6	17
Zürichsee	310	32	9.7
Hallwilersee	460	40	11.5
Greifensee	950	118	8.1

Average 11.6 ± 3.9

*) For original data cf. Vollenweider, 1969.

For a second attempt, σ_p may also be estimated from

$$\sigma_p = \frac{L_p}{\bar{z} \cdot [\bar{P}]_{\lambda}} - \rho_w$$

which follows directly from the steady state equation pertaining to the mixed reactor lake model (cf. Vollenweider, l.c.). In this procedure, it

was possible to use a much larger data bank including lakes of various physiographic and trophic characteristics. Results for some 25 lakes are plotted in Fig. 1 against mean depth.

The trend over this large spectrum indicates that σ_p depends inversely on \bar{z} although the scattering of the individual lakes is but little

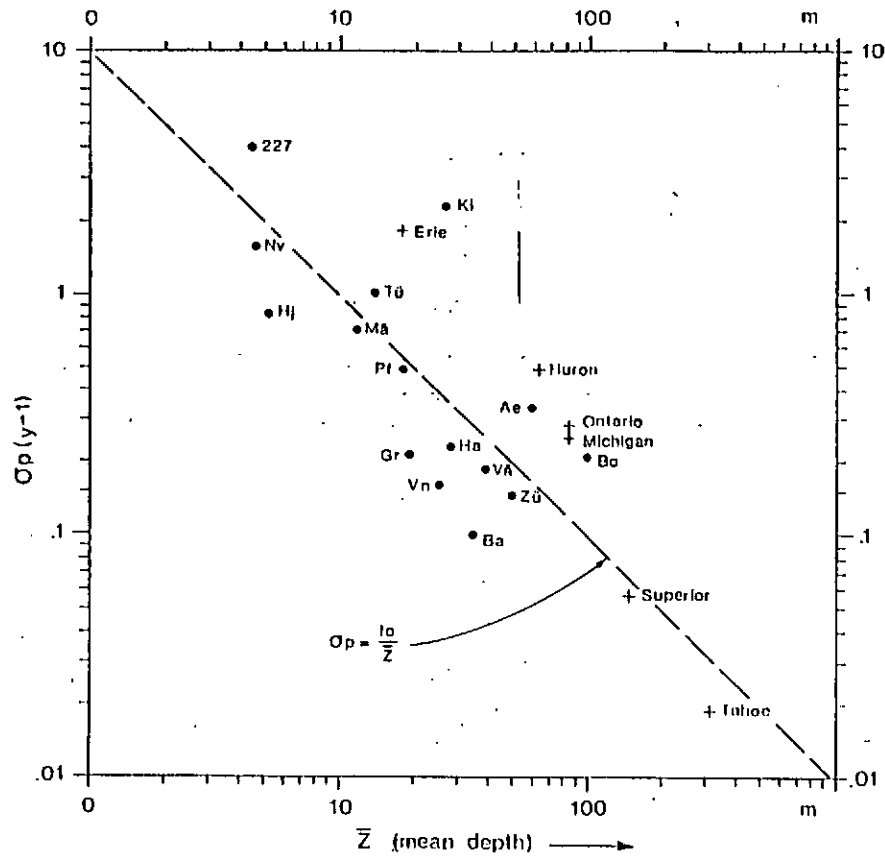


Fig. 1. - $\sigma_p = \frac{I_p}{\bar{z} / [P]_{\lambda}} - p_w$.

related to either trophic conditions or other limnological characteristics except that lakes of considerable surface extension (e.g. the Laurentian Great Lakes) consistently show higher σ_p values than smaller lakes of comparable mean depth. This peculiarity cannot be explained as yet.

As a general rule, however, σ_p may be approximated by

$$\sigma_p = 10/\bar{z}$$

Introducing this relation into (2a), then L_c would be given as

$$L_c \text{ (mg/m}^2 \cdot \text{y)} = [\overline{P}]_c^{\text{cr}} (\bar{z}/\tau_w + 10), \quad (4)$$

or, assuming for $[\overline{P}]_c^{\text{cr}} = 10 \text{ mg/m}^3$

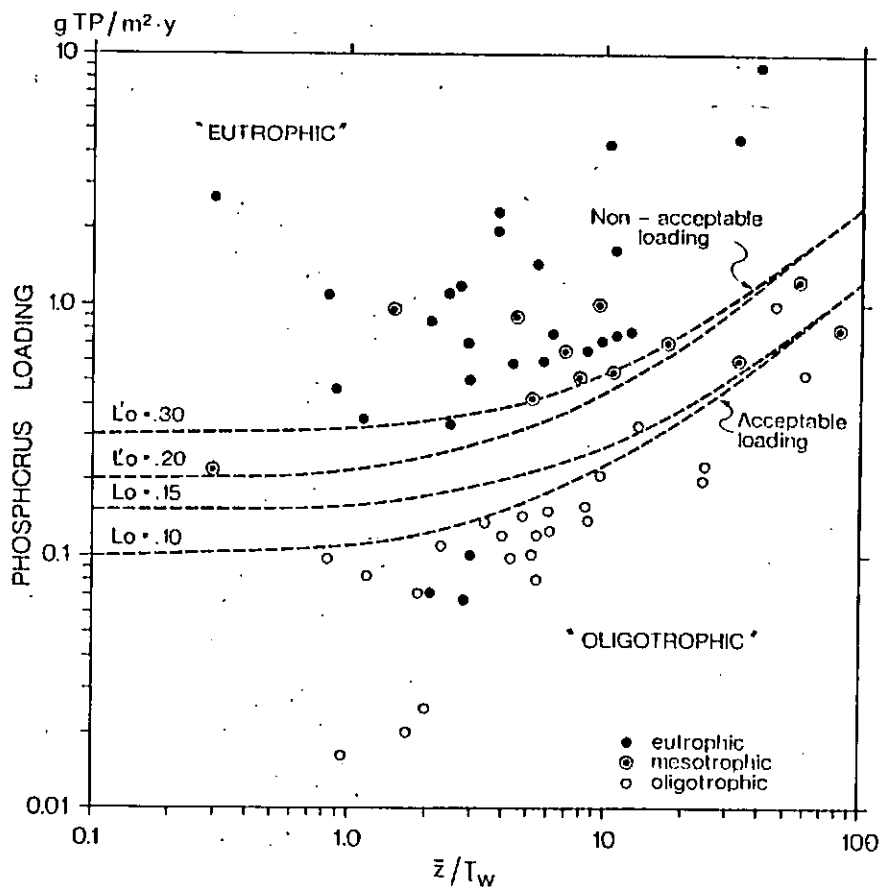


Fig. 2. - Test of loading tolerance for phosphorus according to Eq. (4). Result from an OECD Workshop.

$$L_c \text{ (mg/m}^2 \cdot \text{y)} = 100 + 10 \bar{z}/\tau_w. \quad (4a)$$

Lakes receiving a phosphorus load essentially below the corresponding estimates from (4a) should be oligotrophic. Conversely, it is reasonable to predict that lakes receiving at least twice the load as calculated from (4a) would be on the eutrophic side.

Eq. (4) has been tested in a workshop exercise on a large variety of lakes recently studied in connection with the North American project of the OECD Cooperative Programme on Eutrophication (1). In order to account for the statistical uncertainties regarding σ_p in (3), allowance has been made for some marginal variability of the constants in (4).

The result of this workshop exercise is shown in Fig. 2. From the more than 60 lakes plotted, only a small fraction (three or four lakes defined as «eutrophic» and «mesotrophic», respectively) do not fit the expected pattern. Conversely, a corresponding exercise plotting L_p against \bar{z} only resulted in a much larger fraction of misplaced lakes.

In view of this experience, it can be concluded that (4) represents a considerable improvement over (1). It is further remarkable that the two procedures discussed thus far give practically the same order of magnitude for the apparent settling velocity, i.e. about 10 m/y (2). However, this value lies considerably below experimentally determined sinking velocities (cf. e.g. Burns and Pashley, 1974); accordingly the apparent settling velocity, as used in this paper, is not a physical entity *per se*, but a balance value integrated in time over all positive and negative settling velocities which in addition includes the effects due to remineralization over that period of time. Therefore, it would be misleading to introduce real settling velocities into Eq. (4). If done, this would result in an increased apparent loading tolerance, particularly for lakes of low hydraulic load, and hence, lead to erroneous predictions.

3. - CRITICAL ASSESSMENT.

Although Eq. (4) appears to be by far more satisfactory for predicting loading tolerances than the original criteria based on mean depth only, there is legitimate ground to argue that, using Eq. (4) as reference criterion, one ignores, or at least underestimates, the effect of mean depth, i.e. the dilution function. Indeed, $\bar{z}/\tau_w = q_s$ represents the hydraulic load which is independent of mean depth.

At the basis of his two-layer model — which is more elaborate than our mixed reactor model — Imboden (1974) has demonstrated that the

(1) OECD Workshop 1974 at CCIW, Burlington. Participants have been requested to plot their P loading values against \bar{z}/τ_w of a blank diagram indicating at the same time the trophic character (oligo-, meso-, eutrophic) of the lake in question. Eq. (4) has been drawn after completion of the exercise. Names of the lakes in Fig. 2 are omitted to preserve the anonymity of participants.

(2) In a comment of S. C. Chapra (1975), using data of Kirchner and Dillon (1975) and their empirical relationship of phosphorus retention against hydraulic load derived on 15 Ontario lakes, the author found by least square techniques 16 m/y for the apparent settling velocity.

effects of mean depth and hydraulic load on permissible loading levels can be separated in principle. At a first glance, it would appear that his criteria are more adequate than ours. However, a critical analysis of his diagram reveals an inconsistency not easily noted, i.e. the strong negative bending of derived loading tolerance curves for lakes having a mean depth of less than 20 to 30 m. From his theory, it would follow e.g. that for lakes having a mean depth of some 20 m, the loading tolerance would range from 100 mg/m² · y for lakes receiving a hydraulic load of about 9 m/y (0.025 m/day), to only 250 mg/m² · y for lakes receiving a hydraulic load of 73 m/y (0.2 m/day). This is in contradiction to experience on highly flushed lakes. Indeed, for the above example, the average inflow concentration in the first case would be 11 mg/m³ (100/9), in the second, however, only some 3.5 mg/m³ (250/73). Studies conducted by Dillon and Rigler (l.c.) and Kerekes (1975) on Canadian lakes, show that highly flushed lakes with average inflow concentration as high as 10 mg P/m³ can receive a total annual load of several thousands of mg/m² · y without noticeable alteration of their oligotrophic character. Accordingly, one would expect the loading tolerance curves in Imboden's diagram to bend in the opposite sense, i.e. flatten out with decreasing mean depth. The usefulness of the notion of «average inflow concentration» shall be further explored later on.

The two arguments have given rise to further studies on the critical loading concept.

4. - IMPLICATIONS OF PHOSPHORUS RESIDENCE TIME AS A REFERENCE PARAMETER.

In light of what has been stated above, Eq. (4) needs to be modified to account for mean depth in addition to hydraulic load. As has been discussed elsewhere (cf. Vollenweider, l.c.) the basic attempt in pursuing a mixed reactor model was to conserve, as far as possible, the dimensional consistency. However, this cannot be done without forcing the system by over-simplifying the connected relationships. In order to overcome the problem, one has at least two ways open, either to attempt to model lakes from the point of view of an interactive dynamic more-layer system in the sense as discussed by Imboden (l.c.), or to try to circumvent the nontrivial difficulties connected with this concept by giving up the dimensional consistencies, using instead statistically given parameter connections. This, indeed, can be defended not only because of our inability to sufficiently cope with the complex interactions between the different systems components in real situations, but also that at least some of the interactions, in addition to being non-linear, are to a considerable degree stochastic in nature. Accordingly, whatever the way of choice might be, we are forced into adopting certain more or less defensible shortcuts.

The possibility of a shortcut results from consideration of the residence time of phosphorus. An adequate definition of this notion encounters the same difficulty as the corresponding definition of the water residence time (cf. Piontelli and Tonolli, 1964; Boyce, 1975). However, for a variety of practical purposes, theoretical filling-time of a lake, $\tau_w = V/Q$, appears to be as appropriate as any other more elaborate definition. In principle, the same concept can be expanded to any substance entering a lake and requires the sole knowledge of the total load and the average, or an appropriately selected value for the total content of that substance in the lake, i. e.

$$\bar{\tau}_M = \frac{M_\lambda}{I_M} ;$$

or, for phosphorus

$$\bar{\tau}_p = \frac{P_\lambda}{I_p} = \frac{P_\lambda/V}{I_{p,v}} = \frac{[P]_\lambda}{[P]_i} \quad (5)$$

Eq. (5) defines the filling time, i. e. the hypothetical time necessary to bring the phosphorus concentration of a lake to its present level starting from concentration equal 0. This version has been used by Sonzogni *et al.*, 1973; Vollenweider, 1975, a. o.

However, this definition contains some unrealistic connotations. Because of the fact that the phosphorus loading — with the exception of direct inlets of high concentration which account only marginally for the total water discharge — is not independent from the water balance, $I_{p,v}$ (the volumetric loading) has primarily computational meaning. Therefore, it would probably be more meaningful for comparative purposes to consider the residence time of phosphorus relative to that of water, i. e.

$$\pi_r = \bar{\tau}_p/\tau_w = \frac{P_\lambda}{I_p} \cdot \frac{1}{V/Q} = \frac{[P]_\lambda}{[P]_i} \quad (6)$$

which defines the ratio between average lake concentration, $[P]_\lambda$, and the average inflow concentration $[P]_i$. The question then arises of what relation exists between π_r and the residence time τ_w , considering a large enough sample of lakes of different limnological characteristics.

In Fig. 3, numerical values of $\bar{\tau}_p/\tau_w$ from some 21 lakes are plotted against τ_w . Lakes having $\tau_w < 1$ have been excluded. As expected, the statistical relationship between $\bar{\tau}_p/\tau_w$ is neither independent nor inversely proportional to τ_w but somewhere in between, i. e. $\bar{\tau}_p/\tau_w$ tends to decrease with increasing τ_w . Regression calculations have been made in two steps, originally including only 14 lakes, supplemented in a later stage by another 7 lakes. Both steps are shown in order to demonstrate that inclusion of further data would hardly alter the picture. The two

regression lines are not significantly different. Therefore, it can be concluded that the relative residence time of phosphorus depends on the residence time of water by a statistical relationship of the form

$$\bar{\tau}_p/\tau_w = x \cdot (\tau_w)^{-\alpha} ; 0 < \alpha < 1 . \quad (7)$$

From the mixed reactor theory follows further that

$$\bar{\tau}_p/\tau_w = \frac{\rho_w}{\rho_w + \sigma_p} = \tau_r, \quad (8)$$

i. e. σ_p can be estimated from τ_r whatever the specific meaning of σ_p may be. Combining (7) and (8), and introducing this in (2), one therefore can postulate that

$$L_c = \frac{[P]_c}{x} \cdot (\bar{z}/\tau_w) \cdot \tau_w^\alpha = \frac{[P]_c}{x} \cdot (\bar{z}/\tau_w)^{(1-\alpha)} \cdot \bar{z}^\alpha . \quad (9)$$

The first expression of (9) gives the critical loading as a function of the hydraulic load and water residence time, the second as a function of the hydraulic load and mean depth; both expressions are equivalent. As has been discussed in the introduction to this section, Eq. (9) is no longer correct in the dimensions unless x is redefined.

5. - IMPROVED LOADING CRITERIA.

As a first step one may introduce in (9) values for α and x which result directly from the correlation analysis of Fig. 3. Accordingly,

$$L_c \text{ (mg/m}^2 \cdot \text{y)} = 15.4 (\bar{z}/\tau_w)^{0.583} \cdot (\bar{z})^{0.415} \quad (9 \text{ a})$$

which may be simplified

$$L_c \text{ (mg/m}^2 \cdot \text{y)} = 17 (\bar{z}/\tau_w)^{0.6} \cdot (\bar{z})^{0.4} . \quad (9 \text{ b})$$

With Eq. (9 a) and (9 b) it is therefore possible to define the relative contribution of the hydraulic load and mean depth separately, and accordingly can be developed into a simple nomogramme (cf. Fig. 4). Prior to discussing its shortcomings, we shall discuss its relation to (1). Essentially, Eq. (9 b) and (1) differ in the magnitude of the exponent in \bar{z} (0.4 versus 0.6); further, the factor which takes care of the hydraulic load in (9 b) can be thought of to be included in the proportionality factor of (1). Indeed, a review of all lakes which originally had been included in the $L_c - \bar{z}$ diagramme revealed that the average (\bar{z}/τ_w) for all lakes was about 8.5. From this the following loading tolerances for

lakes of 10 and 100 m respectively, using Eq. (1) (lower limit) and Eq. (9b), are computed.

	Eq. (1) $25 \bar{z}^{0.6}$	Eq. (9b) $17. (8.5)^{0.6} \bar{z}^{0.4}$
\bar{z}		
10 m	99	154
100 m	396	387

According, for lakes of an average hydraulic load of some 10 m/y, the estimated loading tolerance results in the same order of magnitude, yet, Eq. (1) would fail for lakes of very low and very high flushing rates

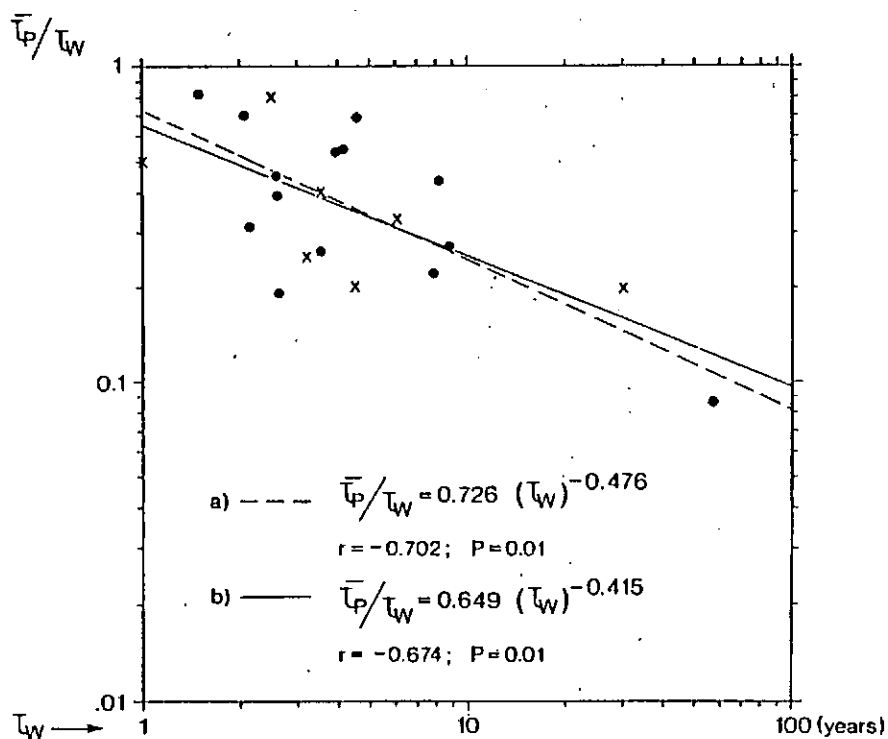


Fig. 3. - Relative phosphorus residence time as function of water residence time (filling time).

respectively. It is, however, comforting how close the estimates are for average conditions.

In regard to the shortcomings of (9), one can see from Fig. 4 that the loading tolerance curves — although accounting for the hydraulic load — do not flatten out for lakes of lower mean depth, as expected, yet, the pattern, generally speaking, is considerably different from that of Imboden's diagramme.

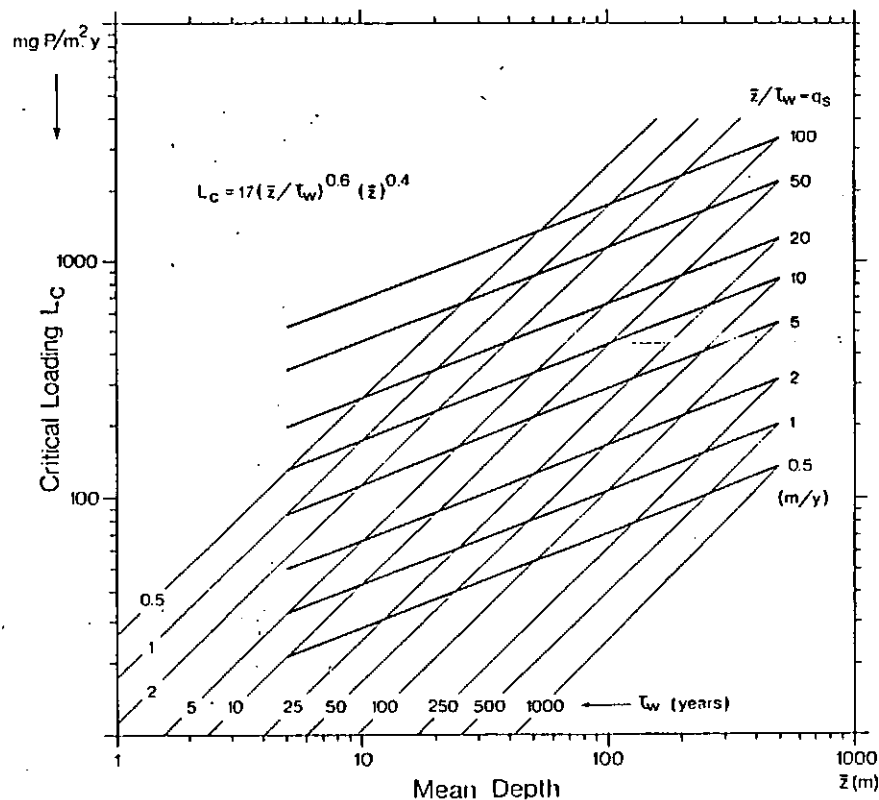


Fig. 4. - Critical Loading (lower limit) for phosphorus according to Eq. (9b).

The inadequacy of (9) for the mean depth range of less than 20 m and lakes of increased hydraulic load is due to the fact that the statistical relation given in Fig. 3 regarding the dependency of $\bar{\tau}_p/\tau_w$ on τ_w cannot be linearly extrapolated below $\tau_w < 1$. Indeed, one would have to expect that

$$\bar{\tau}_p/\tau_w \rightarrow 1, \text{ for } \tau_w \rightarrow 0.$$

An approximation which takes care of this would be

$$\bar{\tau}_p/\tau_w = \frac{1}{1 + \tau_w^x},$$

which, for our purposes, can be simplified to

$$\bar{\tau}_p/\tau_w = \frac{1}{1 + \sqrt{\tau_w}} = \frac{1}{1 + \sqrt{\bar{z}/q_s}}, \quad (10)$$

Numerically, (10) gives practically the same values for $\bar{\tau}_p/\tau_w$ at values of $\tau_w > 5$, with an intercept of 0.5 for $\tau_w = 1$, i.e. slightly less than the statistical intersection. Therefore, it might be that (10) underestimates the average trend slightly for τ_w values < 1 . Unless larger

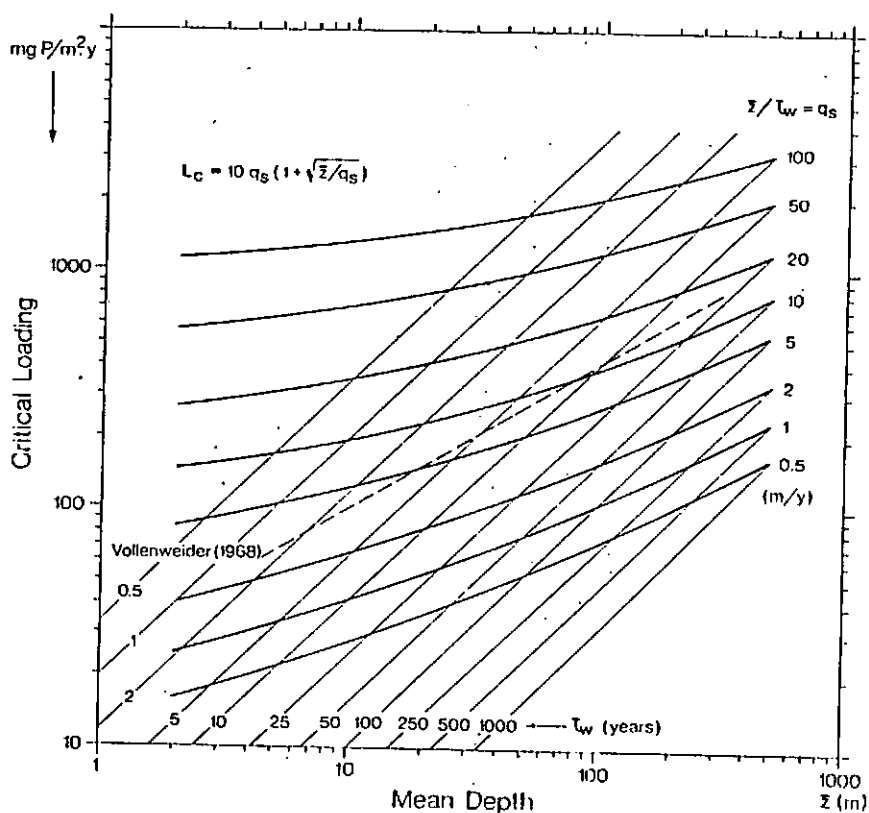


Fig. 5. - Critical Loading (lower limit) for phosphorus according to Eq. (11a) plotted as function of mean depth.

inconsistencies are encountered, however, there is no reason for essentially modifying Eq. (10).

Combining (10) with (8) and (2), then the more generalized relationship

$$L_c \text{ (mg P/m}^2 \cdot \text{y)} = [\overline{P}]_c^{\text{sp}} \cdot \bar{z} \left(\frac{1 + \sqrt{\tau_w}}{\tau_w} \right) \quad (11)$$

gives a criterion which holds over the total spectrum of combinations of mean depth and hydraulic load. Considering that $\bar{z}/\tau_w = q_s$, $\tau_w = \bar{z}/q_s$ and assuming $[\overline{P}]_c^{\text{sp}} = 10 \text{ mg/m}^3$ (11) may also be written as

$$L_c \text{ (mg P/m}^2 \cdot \text{y)} = 10 \cdot q_s (1 + \sqrt{\bar{z}/q_s}) \quad (11a)$$

(11a) expresses the loading tolerance in terms of the sole characteristics of morphometric properties of the lake cuvette — condensed in the term of mean depth \bar{z} —, and the hydraulic load $q_s = \bar{z}/\tau_w$ which — in es-

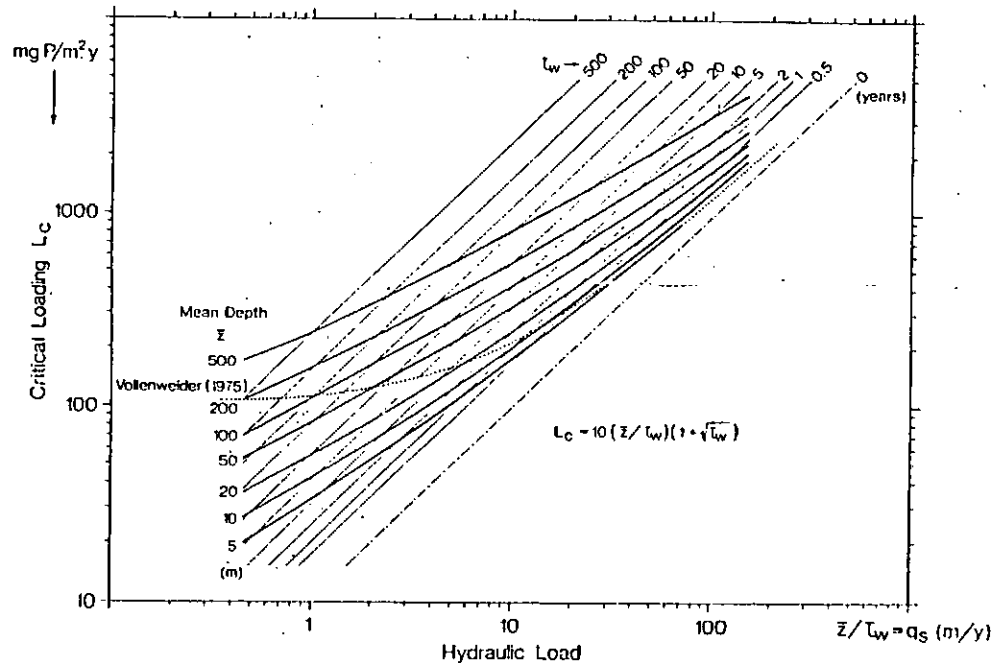


Fig. 6. - Critical Loading (lower limit) for phosphorus according to Eq (11a) plotted as function of hydraulic load.

sence — expresses the relationship between the hydrologic properties of the basin and the lake. This means that, in principle, the loading tolerance for phosphorus — or more generalized, the production capacity — of any lake can be understood as governed to a large extent by two independent functional properties only.

(11a) has been developed in the form of two equivalent diagrammes. In version 1 (Fig. 5) L_c is plotted against mean depth, and parameterized as a function of q_s ; in version 2 (Fig. 6), instead, L_c is plotted against the hydraulic load and split up in terms of mean depth. Although identical in content, each version has its proper merit.

Version (1) shows the characteristic flattening toward the left; this means that the loading tolerance of lakes of moderate mean depth is almost entirely governed by flushing; vice versa, version 2 shows that the loading tolerance of lakes, receiving a modest hydraulic load, is governed more by mean depth.

With (11) and (11a), therefore, a loading tolerance definition is obtained essentially different than that of Imboden. Remarkably, the same pattern — though not in absolute terms — has been derived by Snodgrass (1974) using a two-layer simulation model which — in contrast to that of Imboden — puts particular emphasis on the vertical exchange processes between the layers. Our method, and the method of Snodgrass, therefore, seem to be complementary.

On the other hand, it is also worthwhile to note that our new criteria, and those of Imboden, are practically identical for large, deep lakes as shown from Table 2; cf. column 6 versus (3) and (4). This would mean that the model assumptions made by Imboden are probably applicable for such lakes but would fail for shallower lakes.

Figure 6 supports further a version which I proposed earlier, and which was derived from simple inspection of lakes plotted in a $L_c - (\bar{z}/\tau_w)$ diagramme (cf. Vollenweider, 1975). In this diagramme, the loading criterion for separating oligotrophic from eutrophic lakes was assumed to be

$$L_c \text{ (mg/m}^2 \cdot \text{y)} \approx (100 \text{ to } 200) (\bar{z}/\tau_w)^{0.5} \quad (12)$$

which indeed is a shortcut well in agreement with (11a). Its usefulness has already been demonstrated by Michalski *et al.* (1973), Stockner *et al.* (1974) and several reports in preparation by G. Fred Lee and his collaborators. Its main weaknesses, of course, are the same as those of Eq. (4a).

6. - VERIFICATION.

Loading criteria cannot be verified, of course, in the proper sense of the term. However, any of the former equations implicitly contain the hypothesis that a relation exists between the actual phosphorus loading and production level attained by the lake(s) in question. Up until recently, it was impossible to discuss this aspect in quantitative terms be-

Table 2. - Phosphorus loading tolerance of selected deep lakes.

Lake	Mean Depth (Z)	Hydraulic Residence Time (τ_w) Years	Present Phosphorus Loading(*) g P/m ² .y	Trophic Conditions	Critical Loading Estimates L _C (g P/m ² .y)					
					(1)	(2)	(3)	(4)	(5)	(6)
Lake Superior ¹⁾	148	185	0.03(0.03)	0	0.50	0.11	0.11	0.12	0.09	0.10
Lake Michigan ²⁾	84	113	0.14(0.29)	0-m	0.36	0.11	0.08	0.09	0.09	0.10
Lake Huron ¹⁾	61	21	0.13(0.15)	0-m	0.30	0.13	0.17	0.16	0.17	0.12
Lake Erie ¹⁾	18	2.6	1.06(0.98)	m-e	0.14	0.17	0.1	0.18	0.26	0.09
Lake Ontario ¹⁾	84	7.9	0.65(0.86)	m	0.36	0.21	0.41	0.40	0.33	0.32
Lake Tahoe ³⁾	300	700	0.04	0	0.75	0.10	0.10	0.12	0.07	0.16
Lake Maggiore ⁴⁾	177	4	-3	+ m	0.55	0.54	1.31	1.33	0.67	1.5
Lake Léman ⁵⁾	154	12	0.7-1.9	+ m	0.51	0.23	0.59	0.57	0.36	0.47

(*)¹⁾ Sources: IJC Reports (1969; 1973)

Patalas (1972 figures in parenthesis)

²⁾ Lee 1974³⁾ Goldman (1974, unpublished)⁴⁾ Calderoni (personal communication)⁵⁾ Jaquet (personal communication)

(1) from Mean Depth (cf. Vollenweider (1968))

(2) from $0.1 + 0.01 (Z/\tau_w)$ (cf. Vollenweider 1975)(3) from $0.017 (Z/\tau_w)^{0.6} (Z)^{0.4}$ (present paper)(4) from $0.01 (Z/\tau_w) (1 + \sqrt{Z/q_s})$ (present paper)(5) from $0.1 (Z/\tau_w)^{0.5}$ (present paper)

(6) from Imboden (1974)

cause of lack of comparable data. With the initial results available from the OECD programme, however, a preliminary exploration is now feasible.

For this, Eq. (11) may be rewritten, not in the terms of critical loading and critical concentration, but simply relating phosphorus concentration to loading, i. e.

$$[P]_L = (L_p/q_s) \cdot \left(\frac{1}{1 + \sqrt{z}/q_s} \right) \quad (13)$$

In this (L_p/q_s) represents the average inflow concentration as discussed earlier. Its further implication shall be considered further below.

Several authors have shown that a relation exists between the spring overturn phosphorus concentration and the average chlorophyll build-up during the following season (Sakamoto, 1966; Dillon, 1974). According to this, we may postulate that (13) could be read in the sense of a production equation in which the LHS is measured by a biological entity such as chlorophyll.

In order to test this hypothesis, average epilimnetic chlorophyll concentrations for some 60 lakes have been plotted against

$$(L_p/q_s) \left(\frac{1}{1 + \sqrt{z}/q_s} \right);$$

(cf. Fig. 7). Although there are a few uncertain values (¹), the relationship is unquestionable, giving a correlation coefficient of 0.868. Also, the exponent derived from the least square fit, though slightly less than 1, deviates only marginally from unity.

In addition to this, each lake plotted has been characterized as oligo-, meso-, or eutrophic, according to the subjective judgement provided by the author. The above discussion on critical phosphorus loadings implies that oligotrophic lakes would be found to the left of the 10 mg/m³ mark, and eutrophic to the right of the 20 mg mark. This is indeed the case for the majority of lakes considered with a few noticeable exceptions. The data available do not allow, however, analysis of the nature of such uncertain lakes.

As a whole, however, Fig. 7 can be considered as valid proof for the realism of the new phosphorus loading criteria presented here. The spectrum of lakes covered includes shallow to deep, and little to highly flushed basin, and hence, represents a valid sample of lakes of the temperate zone. Lakes outside this zone, however, will probably have to be excluded from the application of this criteria.

(¹) The values plotted refer to a large part to summer averages; however, included also are a few yearly averages and averages for which it was not possible to clearly understand their time limits.

7. - PREDICTION OF CHLOROPHYLL FROM LOADING CHARACTERISTICS.

It is self-evident that the relationship in Fig. 7 can be used to predict biomass, in terms of chlorophyll, in relation to the specific phosphorus loading characteristics. In absolute terms, the prediction is probably quite good for low and medium productive lakes, yet less certain for highly productive lakes. According to the experience gained thus far in applying the prediction to lakes not included in the correlation analysis, one can expect that at least the peak values of chlorophyll measured

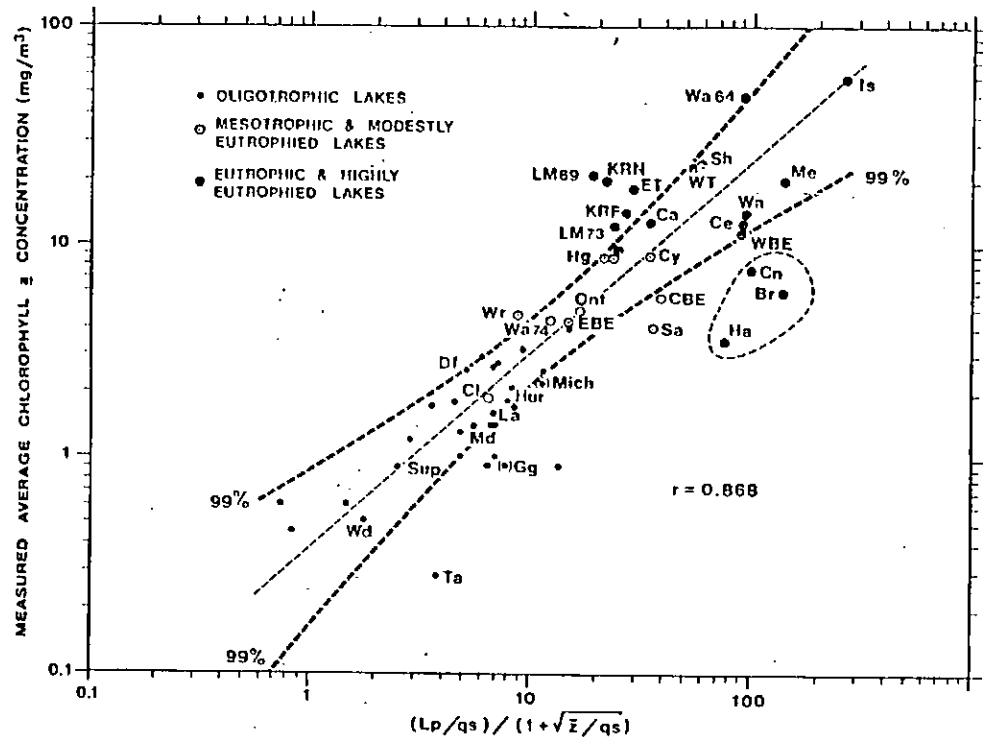


Fig. 7. - Statistical relationship between average epilimnetic chlorophyll concentration and phosphorus loading characteristics according to Eq. (13). Original data by courtesy of authors collaborating in the OECD Cooperative Programme on Eutrophication, North American Project: P. Brezonik, G. J. Brunskill, R. Carlson, N. Clesceri, D. Cook, R. Daley, M. Dickman, P. J. Dillon, W. T. Edmondson, C. R. Goldman, U. T. Hammer, L. Hetting, N. A. Jaworski, J. Kalff, J. Kerekes, G. F. Lee, K. Mahug, R. O. Megard, J. K. Neel, T. G. Northcote, R. Oglesby, C. F. Powers, J. Robinson, C. Schelske, D. Schindler, J. Shapiro, J. Stockner, S. J. Tarapchak, R. R. Weiler, C. Weiss, E. Welch, n. o. sources.

will fall within the 99% confidence limits given in Fig. 8. For lakes receiving a very high phosphorus load one has reason to believe that their production level is not solely controlled by phosphorus, and hence, one would expect the relationship to break down.

A specific application has been made to Lake Washington over the period of its recovery, cf. Fig. 9. During the period from 1957 to 1964 during which the load still increases, the summer chlorophyll also increases within the confidence limits of the prediction; after sewage diversion has been implemented, however, the measured chlorophyll values remain slightly above prediction until 1971 which may be interpreted as a lag phenomenon. From 1972 to 1974, the summer chlorophyll follows the variations of the loading characteristics practically on the line, including the 1957 pre-diversion situation. It has to be mentioned that the loading characteristics over all years have been derived from variations of the hydraulic discharge each year; mean depth has been treated constant.

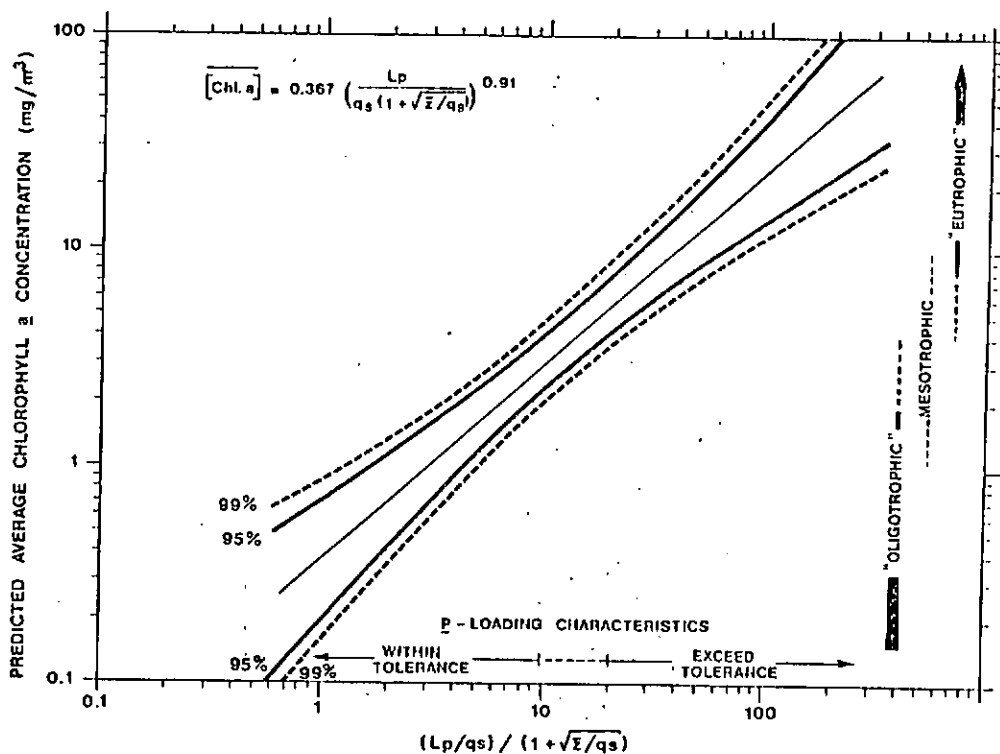


Fig. 8. - Prediction of average chlorophyll and trophic character of lakes relative to phosphorus loading characteristics.

8. - THE π_r RATIO.

Eq. 6 defines the π_r ratio as the quotient between the average lake concentration and the average inflow concentration. In principle, this definition extends to any substance flowing into a lake. For highly flushed lakes with but little sedimentation, this ratio is expected to approach unity, yet, with increasing involvement of the substance in question into the lake metabolism, this ratio can deviate more or less from 1. Values <1 signify that the substance has a positive net flow to sediments; conversely, values >1 signify that the net exchange with sediments is negative, i.e. the substance accumulates in the water phase of the lake system.

In the case of phosphorus, one would expect that, statistically,

$$\pi_r = [P]_L/[P]_I \sim \frac{1}{1 + \sqrt{z/q_s}} = \frac{1}{1 + \sqrt{\tau_w}}. \quad (14)$$

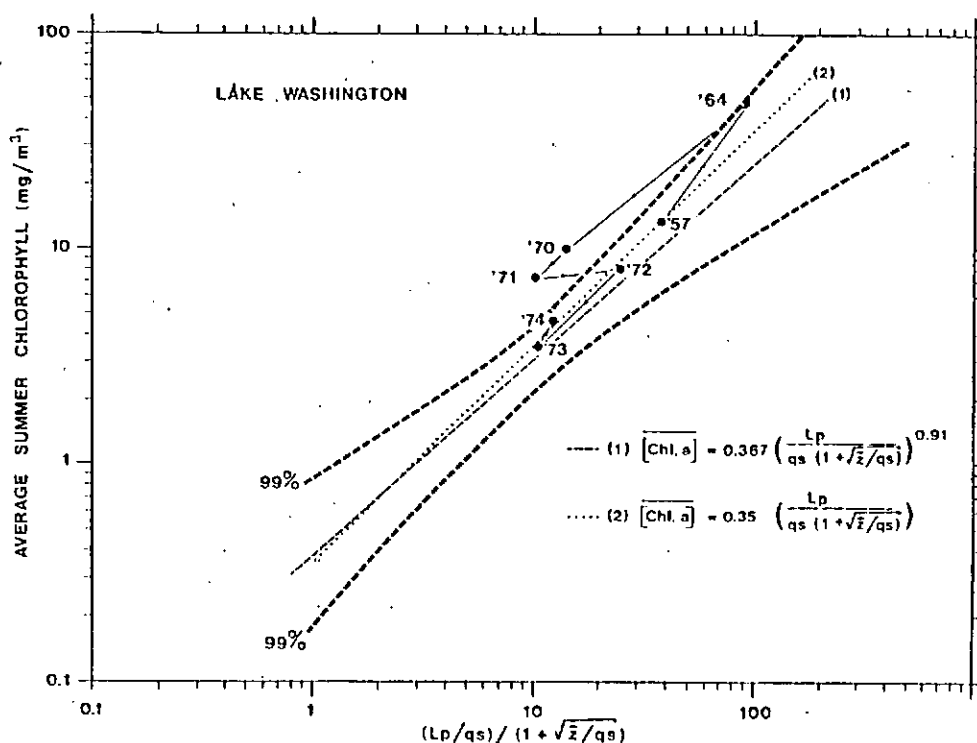


Fig. 9. - Application of the phosphorus loading concept to the evolution of Lake Washington prior and after sewage diversion. Original data by courtesy of Professor W. T. Edmondson.

Indeed, this relationship has proved to be extremely valuable. First of all, it serves as a guideline to judge the validity of the basic data. Strong deviations of π_r from the expected value make the basic data suspect. If, for example, in a highly flushed lake, π_r is significantly above the value from (14), one is entitled to believe that the loading has been underestimated; conversely, if the calculated π_r is lower than the value from (14), one has to suspect that the total loading has been overestimated. These considerations are particularly useful in cases where the loading has been obtained from indirect estimates.

If, however, the basic data are such that any uncertainty would have little effect on π_r , then this ratio gives additional information about the metabolic properties of a lake relative to phosphorus. In the case of π_r significantly smaller than $1/(1 + \sqrt{\tau_w})$, the accumulation of phosphorus in the sediments of the lake in question must occur at a rate above average; this situation may be caused by high settling rates of inorganic suspensions, or by phytoplankton having high specific density such as highly silicated diatoms. Strong positive deviations of π_r from (14), on the other hand, may signify sedimentation rates below average as caused by light tiny phytoplankton or mineral turbidities, or reduced capacity of sediments to take up phosphorus permanently as can be expected in highly eutrophic lakes. Indeed, in a few cases of highly eutrophied lakes, the π_r ratio has been found to be in excess of 1.

In this light, then, it is also possible to define loading criteria for phosphorus as average inflow concentrations. This is done by dividing both sides of Eq. (9b) and (11a) by $q_0 = \bar{z}/\tau_w$; the resulting relationship is shown in Fig. 10 which also includes a few specific examples. These, in addition to demonstrating the transition range from oligo- to eutrophy, corroborates also the correction which was deemed necessary in (9) to properly describe the character of highly flushed lakes. It is evident that the loading tolerance of lakes to the left of the diagramme which are highly flushed oligotrophic lakes, would erroneously be judged with Eq. (9), yet, they are reasonably comprehended within the boundaries of (11) ⁽¹⁾.

Used in this sense, the average inflow concentration gives a reference figure valid for the lake as a whole. However, the reference figure for the critical loading does not necessarily imply that local areas may not be affected adversely. Indeed, judging from the experience made on a number of lakes listed in Fig. 10, one would have to conclude that local

⁽¹⁾ Because of the washout effect which can be expected to occur at water residence times comparable with the mean life span of phytoplankton populations, it could be argued that the loading tolerance in terms of average inflow concentrations may increase somewhat at very high flushing rates, i. e. at residence times $< 1 \div 2$ weeks. Yet, at the present state of knowledge, no precise quantitative statements as to the magnitudes involved can be made, but the problem could be explored at the basis of appropriate dynamic growth models.

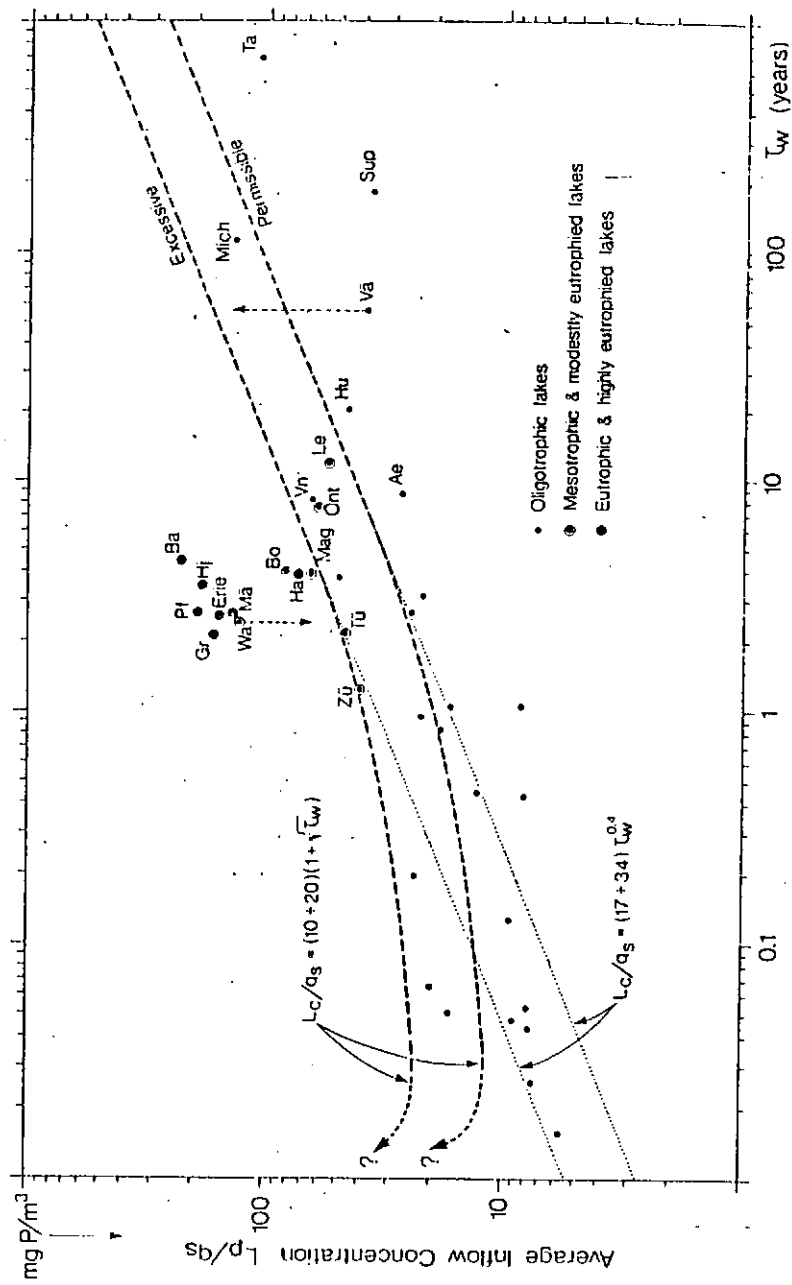


Fig. 10. - Generalization of the phosphorus loading concept in terms of average inflow concentration (L_p/q_S) plotted as function of water residence (filling) time. Lower and upper limits are given. Points plotted referring to a number of known examples; unidentified points (original data from Dillon 1974) referring to oligotrophic highly flushed Ontario lakes.

problems, such as increased growth of periphyton, noticeably *Cladophora*, would have to be expected for average inflow concentrations exceeding 40 to 50 mg P/m³. The implications for a number of cases have been discussed elsewhere (Vollenweider, 1975).

9. - DISCUSSION.

The improvement on available loading criteria for phosphorus was essentially possible only by including terms for water residence time and relative residence times of phosphorus in the lake. The importance of water residence time was recognized by Vollenweider (1968, 1969) and has been stressed further by Dillon (1974), Imboden (1974), Snodgrass (1974), Kerekes (1975), whereas the phosphorus residence time has been considered by Lerman (1974), Sonzogni and Lee (1974),

There are a number of aspects which are worth discussing separately. First, the agreement in principle of our results, with those derived from Snodgrass' theory appears to be more than accidental, although the underlying concepts used are quite different. Snodgrass follows the way designed by O'Melia (1972) and Imboden (l.c.), considering mass balances as depending on the internal thermal cycles and the mass exchanges taking place between vertically connected layers, whereas in our treatment, this dynamic aspect has been largely neglected and substituted for by a global treatment of the phosphorus residence time.

Although not supporting fully Imboden's qualitative conclusions, the partial agreement between his and our criteria for deeper not too highly flushed lakes could, in no sense, be expected *a priori*. Indeed, Imboden's mensural criterion refers to oxygen depletion in the hypolimnion during summer stagnation - arbitrarily set at $\Delta O_2 = 1 \text{ g/m}^3$, which is quite stringent - whereas the mensural criterion used here refers to an average phosphorus concentration of 10 mg/m³ arbitrarily selected at spring overturn time. Therefore, it appears that these criteria for lakes of the kind in question are limnologically compatible.

On the other hand, it is also clear why Imboden's criterion fails for shallow, highly flushed lakes. First, an oxygen depletion of 1 g/m³ for lakes having a hypolimnion of 5 m has quite a different meaning in terms of the epilimnetic productivity than the same rate would have for lakes having a hypolimnion of 100 m. In addition - and this seems to be the more stringent reason - the seasonal cycle and the vertical thermal structure, and their consequence for the intensity and rate of water and material exchange between superposed layers in shallow highly flushed lakes, is hardly comparable to that of deep modestly flushed lakes. The shear forces acting between the layers of a highly flushed shallow lake undoubtedly favour a higher rate of vertical oxygen transport to the hypolimnion over the period of stagnation (if such a stagnation ever occurs at all) as compared to deep lake situations. Accordingly, in the former case, the apparent hypolimnetic oxygen depletion rate represents a

balance value between supply and respiratory consumption, whereas in the latter case the same oxygen depletion rate will be closer to respiratory consumption alone. Therefore the hypolimnetic oxygen depletion rate, for a large spectrum of lakes, cannot be taken as a reference without due consideration of the connected vertical oxygen transport functions.

Conversely, the nutrient availability in the production carrying epilimnetic layers, seems to be more closely connected to productivity, at least in terms of average standing crop, generally. Difficulties will arise, however, with the variability of food transfer from phyto- to zooplankton in different lakes, and this may account for part of the statistical variability of the measured average phytoplankton standing crops (in terms of chlorophyll) in lakes of different limnological characteristics. The phytoplankton-zooplankton interrelationship in highly eutrophic lakes appears to be particularly dependent on the kind of species composition of the biota; hence, if the phytoplankton is composed primarily of species edible for zooplankton, one may find a relatively low phytoplankton standing crop vis-a-vis the standing crop of zooplankton and vice versa. As shown by Steel (1975), Shapiro *et al.* (1975) *a.o.*, this area needs careful consideration for improving our understanding of the trophic nature of lakes as depending on nutrient loadings.

Secondly, the relation of the relative phosphorus residence time, or π_r , as defined here, to the residence time of water, seems to integrate a number of basic limnological processes in balance. Implicitly, the same relationship has been found recently by Larsen and Mercier (1975) using data from a different set of lakes, concluding that the retention coefficient, statistically can be approximated by

$$R_c = \frac{1}{1 + \sqrt{\rho_w}}.$$

This, indeed, is identical to what one would derive from Eq. (6), defining

$$R_c = \frac{[P]_t Q - [P]_l Q}{[P]_t Q} = 1 - \pi_r. \quad (15)$$

Substituting $\pi_r = \bar{\tau}_p / \tau_w$ from Eq. (10), then

$$R_c = \frac{\sqrt{\tau_w}}{1 + \sqrt{\tau_w}} = \frac{\sqrt{\bar{z}/q_s}}{1 + \sqrt{\bar{z}/q_s}} = \frac{1}{1 + \sqrt{\rho_w}} \quad (16)$$

or

$$R_c = \frac{\sqrt{\bar{z}}}{\sqrt{q_s} + \sqrt{\bar{z}}}. \quad (16a)$$

Accordingly, R_c would be an increasing function of mean depth with an upper limit of 1 for very deep lakes ($\bar{z} \gg q_s$), but inversely related to q_s for lakes of comparable mean depth. The sense of this is obviously that

the likelihood of settling of particles, formed in, or reaching the lake from outside, decreases with the rate of flushing, whereas the dependency on mean depth relates to the probability of «trapping» of particles in the hypolimnion as well as the extent of mineralization of organic particles along their way of sedimentation.

Although (16) is in agreement with our previous deductions, one might suspect that R_e would be slightly underestimated for $q_s < 10$. As has been pointed out earlier, the statistical relationship between \bar{z}_0/τ_w versus τ_w gives an intersection of 0.6-0.7 for $\tau_w = 1$, yet $1/(1 + \sqrt{\tau_w})$ would give only 0.5. Indeed, Kirchner and Dillon's statistical analysis for Ontario lakes of some 10 m mean depth would give higher R_e values for $q_s < 10$, yet values for $q_s \geq 10$ are in fair agreement with ours. Contrary to this, Snodgrass' theory would give lower retention coefficients for $q_s > 10$, $\bar{z} > 100$ m. The only known case to the author, apt to provide a judgement whether or not (16a) or Snodgrass' estimates agree more closely with reality, is Lake Maggiore ($\bar{z} = 177$ m, $q_s = 44$ m, $\tau_w = 4$ y). The two values would be 0.67 and 0.32 respectively. From available data on loading and outflow (Calderoni in Barbanti *et al.*, 1974) R_e has been estimated to 0.6. Therefore, (16a) seems to give the better estimate of R_e for such lakes. However, prior to giving a final judgement, much more data from «extreme» lakes should be available.

The discussion of these two aspects indicates the limit of simple mass balance models. If further progress should be possible, then more complex models are needed. It seems to be particularly important to obtain a better hold on parameters which also exert an influence on loading tolerance, such as length of stratification, mixing cycles, depth of thermocline, hypolimnetic entrainment, water discharge and loading cycles, etc. Also, the trophic-dynamic interrelationships in the sense of Lindeman (1942) requires much more sophisticated analyses.

Attempts of this nature are underway in several places; however, there are a number of pitfalls to be avoided in order to significantly exceed what Riley *et al.*, (1949) have already prospected. In spite of the large amount of limnological literature on the subject the tropho-dynamic interrelations are still insufficiently understood; also, careful mass balance studies, broken down into monthly, or even timely closer episodes, are scant. In addition, much of the data used for «verification» of limnological models have been drawn from nonreliable or at least inappropriate data banks, and hence, this has hardly been to the advantage of model development.

Finally, it has to be stated that — whilst much effort has been put into the analysis of the phosphorus situation, the dynamics of other substances has been largely neglected. If we should arrive at more global criteria in the future relative to eutrophication, then, over the next period of time, more substantial studies are required for other nutrients, such as carbon, nitrogen, macro- and micro-elements, and their metabolic interactions.

Acknowledgement

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I wish further to thank Drs. C. Bennett, A. El-Shaarawy and J. Simons from the Canada Centre for Inland Waters who made valuable comments, but in particular, I wish to express my appreciation for the encouragement I have had over the past years from Dr. K. Patalas, Freshwater Institute, Winnipeg, and Dr. F. Lee, Institute of Environmental Science, University of Texas, Dallas, both of whom contributed essentially to the development and application of the nutrient loading concept.

Appendix 1. - USE OF FIGURES 5 AND 6, RESPECTIVELY FOR PREDICTING TOLERANCE LEVELS FOR PHOSPHORUS LOADINGS.

Enter diagrammes either with appropriate figures for mean depth \bar{z} (in metres), and hydraulic load q_s (in metres per year), or, alternatively, follow the isolines for the residence time $\tau_w = \bar{z}/q_s$ (in years) instead of separately calculating q_s . Both entries are equivalent.

Example:

Lake Ontario ($\bar{z} \approx 84$ m, $\tau_w = 7.9$ y, $q_s = \bar{z}/\tau_w = 10.6$ m/y).

The loading tolerance, from either Fig. 5 or 6, would approximatively be 350 and 400 mg P/m² · y. This value specifies the lower limit not to be exceeded to keep the lake in acceptable oligotrophic conditions. Excess loading over about twice the above values, i.e. 700-800 mg/m² · y would cause the lake to become eutrophic.

The loading of this lake, prior to the initiation of the phosphorus reduction programme which is now in full progress according to the US-Canada Agreement, was estimated to 680 mg/m² · y (cf. Anon. IJC Report, 1969). Indeed, the condition of the lake has been judged mesotrophic exhibiting pronounced problems of eutrophication inshore (extended growth of *Cladophora*). This is in agreement with the statement made in the paper (cf. p. 76) that local problems are to be expected, if the average inflow concentration exceeds 40 to 50 mg P/m³. From the data given here, the average inflow concentration would be

$$680/10.6 = 64 \text{ mg P/m}^3.$$

Appendix 2. - SYMBOLS.

	Meaning	Dimensions	
		CGS-System	Practical
V	Lake volume	L^3	m^3 ; km^3
Λ_0	Lake surface	L^2	m^2 ; ha ; km^2
z	Depth	L	m
\bar{z}	Mean depth	L	m
Q_y	Total yearly water discharge	L^3T^{-1}	m^3/y ; km^3/y
q_s	Hydraulic load ($Q_y/\Lambda_0 = \bar{z}/\tau_w$)	LT^{-1}	m/y
τ_w	Water filling time (Water « Residence time ») ($V/Q_y = \bar{z}/q_s$)	T	y
φ_w	Flushing coefficient ($1/\tau_w$)	T^{-1}	$1/y$
P_λ	Total amount phosphorus in lake	M	t ; kg ; g
$[P]_\lambda$	Phosphorus concentration in lake	ML^{-3}	g/m^3 ; mg/m^3
$[P]_\lambda^{sp}$	Lake concentration at spring overturn (P_λ/V)	ML^{-3}	g/m^3 ; mg/m^3
\mathcal{L}_p	Total phosphorus loading	MT^{-1}	t/y ; kg/y ; g/y
L_p	Specific surface loading (\mathcal{L}_p/Λ_0)	$ML^{-2}T^{-1}$	$g/m^2 \cdot y$; $mg/m^2 \cdot y$
l_v	Specific volumnar loading (\mathcal{L}_p/V)	$ML^{-3}T^{-1}$	$g/m^3 \cdot y$; $mg/m^3 \cdot y$
$[\overline{P}]_i$	Average inflow concentration ($\mathcal{L}_p/Q_y = l_v/q_s$)	ML^{-3}	g/m^3 ; mg/m^3
L_c	Critical specific loading	$ML^{-2}T^{-1}$	$g/m^2 \cdot y$; $mg/m^2 \cdot y$
$[P]_c^{sp}$	Critical lake concentration at spring overturn	ML^{-3}	g/m^3 ; mg/m^3
σ_p	Sedimentation coefficient of phosphorus	T^{-1}	$1/y$
$F_p(z)$	Flux of phosphorus through the z - horizon	$ML^{-2}T^{-1}$	$g/m^2 \cdot y$; $mg/m^2 \cdot y$
τ_p/τ_w	« Residence time » of phosphorus relative to « Residence time » of water	0	
π_r	Ratio between average lake concentration and average inflow concentration (e.g. $[P]_\lambda/[\overline{P}]_i$)	0	
R_c	Retention coefficient	0	

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REVIEW

RECENT ADVANCES IN ASSESSING IMPACT OF PHOSPHORUS LOADS ON EUTROPHICATION-RELATED WATER QUALITY

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INTRODUCTION

Eutrophication—excessive fertilization, which is manifested by excessive growths of planktonic (suspended) and attached algae, and aquatic macrophytes (water weeds), can have significant deleterious effects on the beneficial uses of lakes, impoundments, estuarine, and marine waters. Lee (1973), who reviewed in detail the causes, impacts and common control options for eutrophication, pointed out that excessive growths of aquatic plants can interfere with the use of waters for domestic and industrial water supply, irrigation, recreation, fisheries, etc. Further, it has recently been determined for some waterbodies that there is an apparent relationship between the degree of eutrophication and the amount of trihalomethanes formed during chlorination of the water during treatment for domestic use. Trihalomethanes are chloroform-like compounds which if ingested in large amounts, are known to be carcinogenic to animals. The widespread applicability of the apparent relationship between trophic status and trihalomethane formation, however, is not known at this time.

As discussed by Krenkel *et al.* (1979), excessive growths of aquatic plants in a waterbody can also cause deterioration of water quality downstream of the affected waterbody. Of particular concern in this regard is the release from a reservoir of deoxygenated hypolimnetic (bottom) waters which can result from excessive growths of algae. Such releases can cause obnoxious odors as well as impair the fishery potential of downstream waters.

Because of the potential and realized water quality deterioration associated with excessive growths of algae and other aquatic plants, considerable effort is being expended in the U.S. and abroad to control eutrophication. Presented below is a discussion of the recent advances in assessing eutrophication-related water quality and in developing and evaluating eutrophication control options with emphasis on phosphorus load control.

APPROACH TO EUTROPHICATION MANAGEMENT

Cause of Eutrophication

The most cost-effective approach toward eutrophication management is usually to eliminate or reduce the cause of the excessive aquatic plant growths. Algae, as well as other aquatic plants, need a wide variety of chemical constituents, in addition to sunlight, for growth. Ordinarily, all constituents needed for their optimum growth based on the light available, are present in aquatic systems in surplus amounts compared to the organisms' needs, except for nitrogen and/or phosphorus. It is generally the rate of supply of algal available forms of nitrogen or phosphorus which limits the maximum algal biomass reached in a waterbody. When the load of the nutrient which is limiting algal growth is decreased, a decrease in the maximum algal biomass that is produced would be expected. As discussed herein, it usually requires a fairly substantial reduction in the available load of the limiting nutrient to cause a noticeable improvement in eutrophication-related water quality characteristics. Common sources of aquatic plant nutrients to waterbodies include domestic wastewater treatment plant effluent, direct and indirect (tributary streams) runoff from land, and atmospheric precipitation and dry fallout. The amounts of N and P that are derived from these sources can be readily and fairly reliably estimated using techniques described in a subsequent section of this paper.

There are several elements which are basic to developing an effective, in terms of both cost and water quality improvement, eutrophication-nutrient management program, each one of which must be evaluated on a site specific basis. First, efforts should be oriented to controlling the nutrient which limits maximum algal biomass in the waterbody of concern. Most commonly, focus is placed on phosphorus control since it most often is the limiting nutrient and methods to control its loading from domestic wastewaters, often a substantial source, are readily available and relatively inexpensive to implement. It is important to note that even in waterbodies in which algal growth is limited by nitrogen or some other

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factor, phosphorus load reduction can result in improved water quality if the phosphorus load reduction is sufficiently large to drive the waterbody to P limitation, and then reduced sufficiently beyond that to decrease algal growth. Second, if phosphorus control efforts are practiced, they should be directed toward those sources of phosphorus which permit the greatest reduction in algal available phosphorus and which are most readily controllable. Lee *et al.* (1980) provide a state-of-the-art summary of the availability to stimulate algal growth, of various forms of phosphorus and of the phosphorus derived from various sources, as well as methods of evaluating the availability of phosphorus from a particular source. Finally, a quantitative assessment should be made of the improvement in water quality—beneficial uses of the water that can be achieved by implementing P management options. While some espouse the approach to P load reductions to waterbodies that "every little bit helps", there is no technical justification for that approach and following it will not lead to the most cost-effective, technically sound, yet environmentally protective eutrophication control programs. Rather, it can readily lead to the public's spending large amounts of money in the name of "pollution control" with little improvement in water quality beyond that which could be attained using a rational, technically defensible approach to nutrient load assessment and control. To assess the impact on water quality that will result from a given control program, a verified method should be used to relate the phosphorus load and projected change in load to the eutrophication-related water quality response (in terms of beneficial uses and public perception) of the waterbody.

Numerous attempts have been and are currently being made to quantify the cause-effect coupling between nutrient load and eutrophication-related water quality response, to create models which could be used as a basis for eutrophication-related water quality management. Such models, if properly formulated and verified, would allow determinations to be made of the changes in water quality that would result from given changes in nutrient load as well as projections to be made of the water quality that will exist in an impoundment that has not yet been created. Most of this modeling effort has been directed towards formulating "dynamic" models which are mathematical representations of the key physical, chemical, and biological processes governing algal growth in waterbodies. While it is possible to develop a set of differential equations which can describe algal population dynamics in relation to nutrient loads for a given set of circumstances, few of these models have demonstrated capability for predicting response when the nutrient loads are substantially altered or when applied to a waterbody other than the one upon which it was developed. The Canada Centre for Inland Waters (1979) has recently conducted a review of "dynamic" eutrophication models developed for

Lake Ontario and concluded that such models have, at this time, limited predictive capability for water quality management purposes. An alternate approach to the dynamic model is the statistical modeling approach such as the Vollenweider OECD models described below. Occasionally reference will be made to the statistical eutrophication models of Dillon & Rigler (1974) and Larsen & Mercier (1976). As discussed by Rast & Lee (1978), these models have the same technical foundation as the Vollenweider-OECD models. However, the correlations or lines of best fit for these models were not developed on as broad a data base as those of the OECD load-response models discussed in this paper.

OECD Eutrophication Modeling Approach

Background

The Organization for Economic Cooperation and Development (OECD) Eutrophication Study was undertaken to quantitatively define the relationship between the nutrient (phosphorus) load to a waterbody (lake, impoundment, or estuary) and the eutrophication-related water quality response of the waterbody to that load. This recently-completed five year study involved the examination of P load and response characteristics of about 200 waterbodies in 22 countries in Western Europe, North America, Japan, and Australia. The 34 U.S. waterbodies or parts thereof included in the OECD Eutrophication Study Program were selected because a considerable body of information on their nutrient loading, hydrological and morphological characteristics was already available and individuals who had investigated these waterbodies were willing to assemble the information on them. Lee had the contract with the U.S. EPA to critically review these data and evaluate the waterbodies' overall load-response characteristics.

Vollenweider (1975), through his work with the OECD, developed a model which described a relationship between the phosphorus load to a waterbody and the relative, general acceptability of the water for recreational use. Figure 1 shows this model applied to the U.S. OECD waterbodies. Rast & Lee (1978) and Lee *et al.* (1978a) found, as Vollenweider (1975) had for a smaller group of primarily European lakes, that when the annual areal P load to a lake or impoundment is plotted as a function of the quotient of the mean depth (volume ÷ surface area) and hydraulic residence time (filling time), waterbodies which were eutrophic (highly fertilized) tended to cluster in one area, as the oligotrophic waterbodies (poorly fertilized—lower amounts of aquatic plant growth) clustered in another area. As the phosphorus load to a P-limited waterbody having a given mean depth/hydraulic residence time quotient is reduced, eutrophication-related water quality is generally improved, provided the reduction is sufficiently large. It is important to note that the "Excessive" and "Permissible" lines in Fig. 1 do not represent sharp

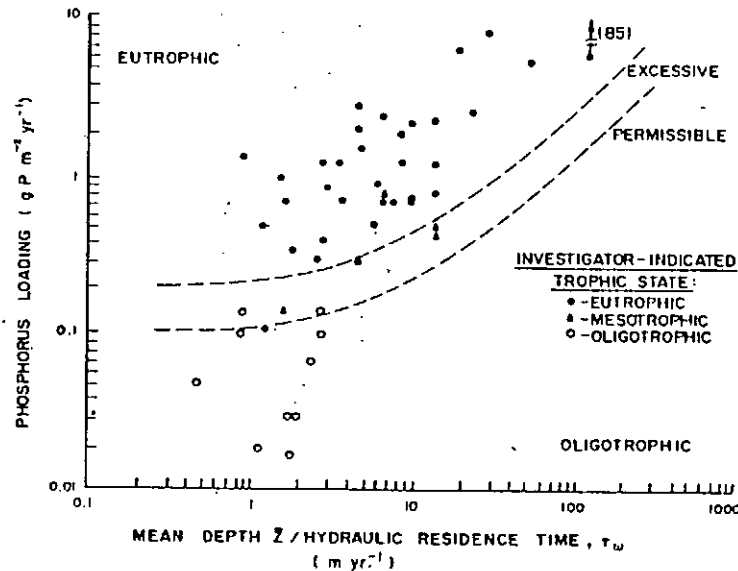


Fig. 1. U.S. OECD data applied to Vollenweider P loading-mean depth/hydraulic residence time relationship (after Rast & Lee, 1978).

boundaries of water quality between oligotrophic and eutrophic waters. For any set of waterbodies having a given mean depth/hydraulic residence time quotient, there is a gradation in water quality along the vertical, with waterbodies having better water quality plotting toward the bottom and those having poorer water quality, toward the top.

Vollenweider (1976) subsequently developed a statistical correlation between the areal annual P loading $[L(P)]$ to a waterbody normalized by mean depth (\bar{z}) and hydraulic residence time (τ_w), (i.e. $[L(P)/(\bar{z}\tau_w)]/(1 + \sqrt{\tau_w})$), and the eutrophication response of the waterbody as measured by mean chlorophyll concentration, for a small group of waterbodies. Based on the data from the U.S. OECD waterbodies, Rast & Lee (1978) and Lee *et al.* (1978a) substantiated that general relationship and defined it for a greater number of waterbodies, deriving a somewhat different line of best fit than Vollenweider's. They also expanded Vollenweider's concept and developed correlations between the normalized P load and Secchi depth (water clarity), and between the normalized P load and hypolimnetic oxygen depletion rate. Figure 2 shows these three load-response relationships for the U.S. OECD waterbodies.

Results of recent work

Since the completion of the U.S. part of the OECD Eutrophication Study, the authors and their associates have evaluated the P load-response relationship for another approx. 40 U.S. waterbodies, including a dozen Tennessee River System impoundments, and a variety of waterbodies in Indiana, Wisconsin, Colorado, Wyoming, and Texas. These additional waterbodies were not selected *per se* in any way, but rather are the group of waterbodies which the authors have

been able to investigate and evaluate over the past several years for nutrient load eutrophication response relationships. The results of these specific studies are in various stages of publication and include Lee *et al.* (1978b, 1979, 1980), Newbry *et al.* (1979, 1980), Lee & Meckel (1978), Horstman *et al.* (1980), Lee & Jones (1981c), Jones & Lee (1978), and Lee & Archibald (1981).

The locations of the approx. 80 U.S. waterbodies evaluated to date are shown in Fig. 3. Figure 4 shows the lines of best fit for the P load-chlorophyll, P load-Secchi depth, and P load-hypolimnetic oxygen depletion rate relationships for these U.S. waterbodies and the associated 95% confidence intervals for the data. (Except for the confidence limits, Figs 2(C) and 4(C) are identical, as no additional P load-hypolimnetic oxygen depletion rate couplings were available.) As can be seen by comparison of Figs 2 and 4, the regressions developed based only on the U.S. OECD data are essentially identical to those developed by including the additional lakes and impoundments.

Further, the remainder of the approx. 200 waterbodies have also been found to follow the general relationships shown in Figs 2 and 4 (Clasen, 1979; Vollenweider & Kerekes, 1980). It therefore appears that these relationships have a wide-spread applicability to most waterbodies—those in northern as well as southern climates, shallow as well as deep waterbodies, lakes as well as impoundments, large waterbodies and small waterbodies. Contrary to some assertions which have been made by the U.S. Army Corps of Engineers (1979) for example, and as shown in Fig. 4, the OECD eutrophication studies have demonstrated that as a group, lakes do not respond differently to nutrient input than impoundments; the same load-response regressions are applicable to both lakes and impoundments. However, as discussed

below, there is a number of characteristics of impoundments as well as some lakes and estuaries, which may necessitate modification of input parameter values.

Based on the results of the OECD eutrophication study and the subjective opinions of the individual investigators regarding the limnological trophic conditions of their waterbodies, Vollenweider (Vollenweider & Kerekes, 1980) developed distributions of the probability that a waterbody will be classified in a certain limnological trophic category, given a certain

chlorophyll concentration, Secchi depth, or total P concentration, among other parameters. Following the probability distribution for chlorophyll and through the U.S. waterbody load-response couplings shown in Fig. 4 the authors developed the response parameter values in Table I which tend to be indicative of certain trophic designations for waterbodies. It is important to understand, however, that these values should not be strictly applied for the classification of waterbodies, especially for water quality management purposes. Lee *et al.* (1981c) discuss how this table and

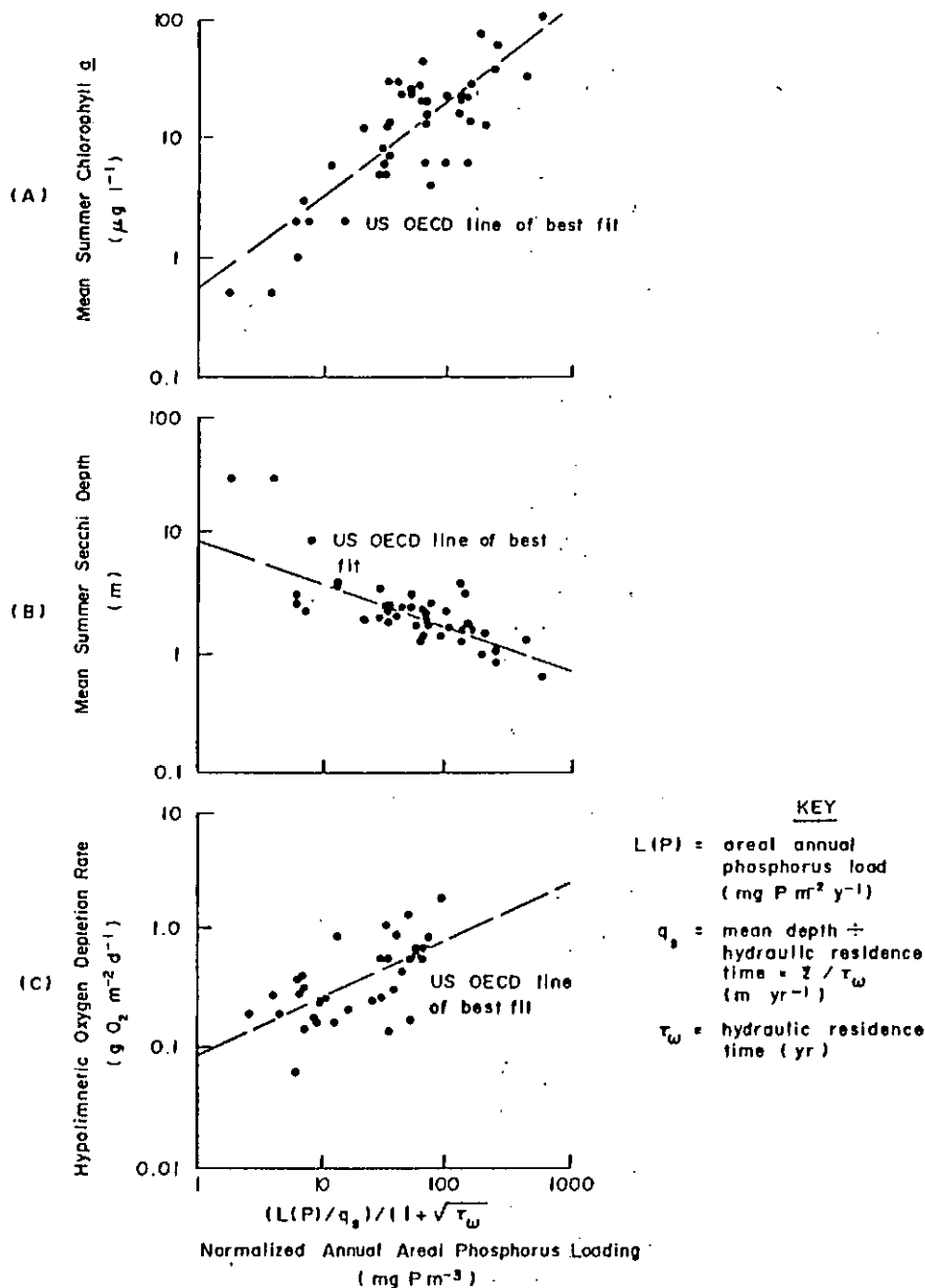


Fig. 2. Normalized P load, chlorophyll concentration, Secchi depth, hypolimnetic oxygen depletion rate relationships based on U.S. OECD data (after Lee *et al.*, 1978a).

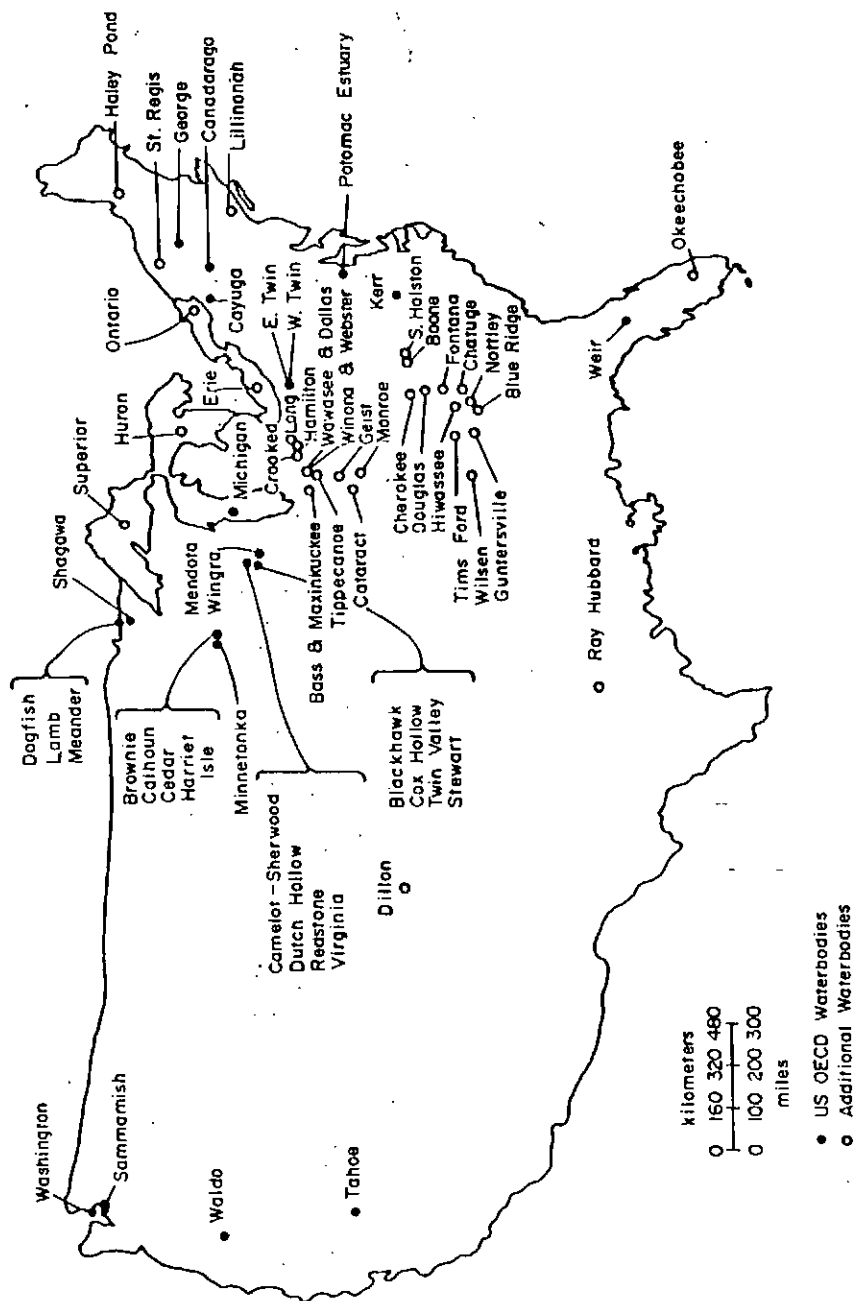


Fig. 3. Locations of U.S. waterbodies for which load-response relationships have been evaluated.

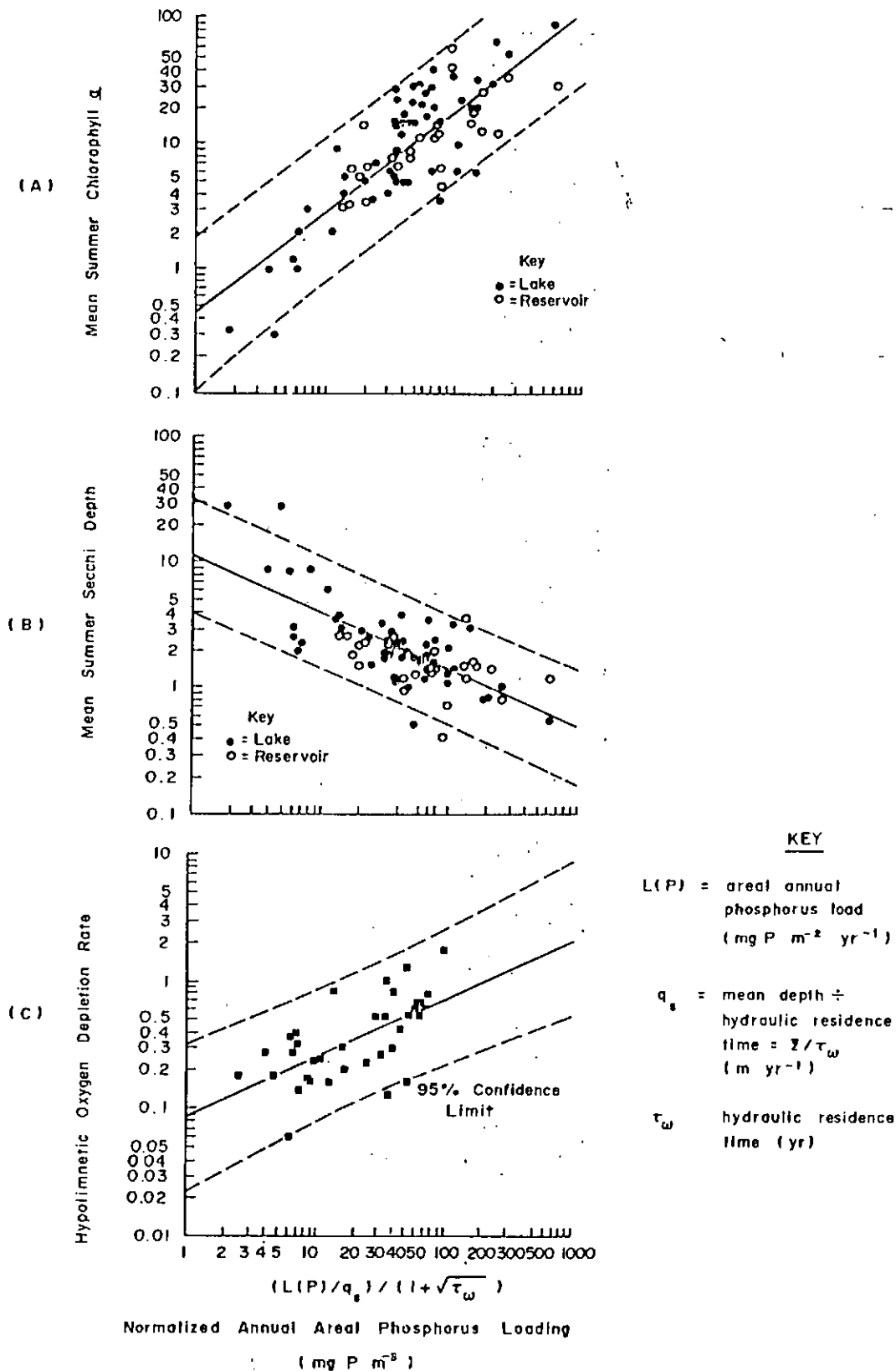


Fig. 4. Updated P load-eutrophication related water quality response relationships for U.S. waterbodies.

Table 1. Limnological classification of trophic status of lakes and reservoirs

Classification	Average planktonic algal chlorophyll ($\mu\text{g l}^{-1}$)	Average Secchi depth (m)	Average in lake total phosphorus ($\mu\text{g l}^{-1}$)
Oligotrophic	< 2.0	> 4.6	< 7.9
Oligotrophic-mesotrophic	2.1-2.9	4.5-3.8	8-11
Mesotrophic	3.0-6.9	3.7-2.4	12-27
Mesotrophic-eutrophic	7.0-9.9	2.3-1.8	28-39
Eutrophic	≥ 10	≤ 1.7	≥ 40

After Lee *et al.* (1981c).

the OECD eutrophication modeling approach should be used in conjunction with desired beneficial waterbody uses, for water quality management—PL 92-500 Section 314 A classification purposes. They stress the fact that for most applications, planktonic algal chlorophyll concentration tends to be the most reliable eutrophication-related water quality indicator; for those waterbodies having average amounts of inorganic turbidity and color, Secchi depth is an adequate secondary parameter. The total P concentrations in a waterbody should not, in general, be used as an indication of water quality response since, as discussed by Lee *et al.* (1981c) as well as others, unless the P is used in the production of undesirable amounts of algae it does not typically cause an impairment of beneficial uses of the water.

Lee & Jones (1979) have further extended the Vollenweider-Rast & Lee-OECD nutrient load-response concept and developed a relationship between P load and fish yield (Fig. 5). While additional data are

needed to better define the correlation, the relationship is clear. It should be noted that this relationship does not explicitly address the types or quality of fish present. While the overall fish yield will increase with increased fertilization, there is a point at which in certain waterbodies, the aquatic plant growth would be sufficient to significantly deplete hypolimnetic oxygen and preclude the survival of cold water fish types such as salmonids (trout). Further, some highly eutrophic waterbodies tend to have large populations of stunted pan fish. Therefore, a plot of yield of desirable fish vs normalized P loading would likely have two breaks in it, one corresponding to the load which causes sufficient algal growth to cause complete hypolimnetic oxygen depletion and the other corresponding to the load resulting in stunted fish growth. It is important to note that while in more developed countries, the management emphasis is on control of eutrophication-related water quality to reduce the production of aquatic plants in order to improve the recreational-aesthetic quality and treatability of the water for drinking, in many lesser developed countries, the emphasis will likely be on maximizing production of fish for food without rendering the waters unsuitable for other purposes. The latter philosophy can now be handled at least in a rudimentary way, through the Lee and Jones relationship (Fig. 5). As discussed above and in detail by Lee & Jones (1979) however, the use of this approach for estimating edible fish abundance must be made cautiously.

Recently, the predictive capability of the OECD modeling approach has been verified by Rast *et al.* (1981). They examined the P loads and waterbody response (chlorophyll) for the approx. 10 waterbodies to which the P loads had been substantially changed, and on which sufficient "before" and "after" P load change data were available. Using the OECD eutrophication modeling approach and pre-change data, the post-change chlorophyll, Secchi depth, and hypolimnetic oxygen depletion rate values were estimated and compared with reported measured values for the respective characteristics after the P load reduction. As shown in Fig. 6, the steady state load-response coupling for any given waterbody tends to move parallel to the U.S. waterbody regression line. The response predicted based on a perfectly parallel re-

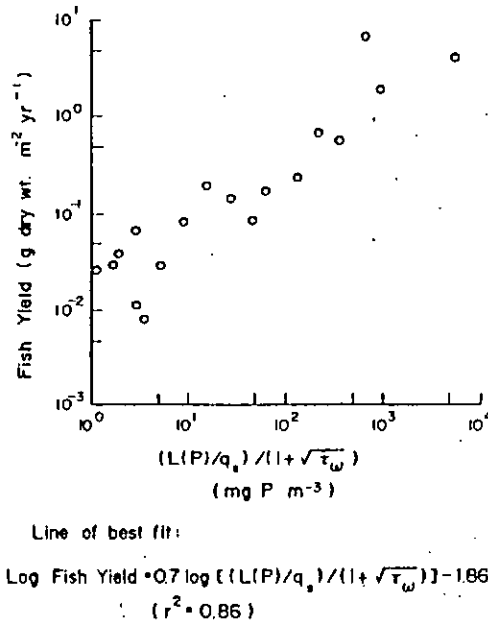


Fig. 5. Relationship between normalized P load and fish yield (after Lee and Jones, 1979).

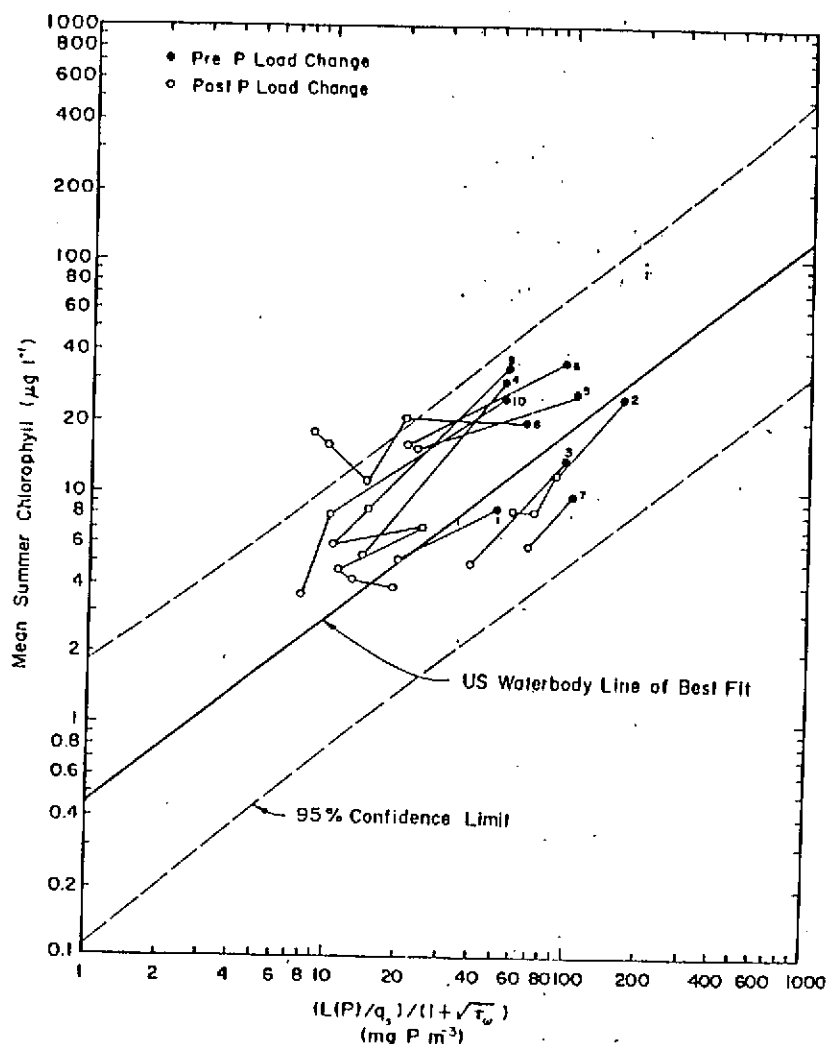


Fig. 6. Application of U.S. OECD load-response model before and after P load changes (after Rast *et al.*, 1981).

sponse in general compared favorably with the measured values. Rast *et al.* (1981) discussed the deviations and likely reasons for them; they are related to the model constraints presented below. While the confidence intervals around the lines of best fit may appear large, this does not affect the predictive capability when one load-response coupling for a waterbody is known. It should be noted that the Vollenweider-OECD eutrophication modeling approach has an inherent verification within it since the models were developed based on the load-response characteristics of approx. 300 independent waterbodies having a variety of characteristics.

Rast *et al.* (1981) as well as Sonzogni *et al.* (1976), discussed the response time necessary to achieve the predicted steady state response to the altered P load. Provided no other substantial load changes occur, new, essentially steady state conditions will be

reached after a period equal to three times the phosphorus residence time of the waterbody. They emphasized that the residence time of P, a nonconservative chemical, is usually substantially shorter than that of a conservative chemical or the hydraulic residence time.

Model constraints

In their current forms, the OECD load response relationships have some constraints. They are generally applicable only to waterbodies in which algal growth is limited by phosphorus. They can only be used to assess eutrophication as manifested by planktonic algal growth and have no applicability to other water quality problems such as sanitary quality, toxicants, siltation, etc. Hydraulic residence time during the growing season must be at least 2 weeks, i.e. sufficient time for algae to grow in response to a load of

nutrients. Further, the P load-Secchi depth relationship was developed for waterbodies having only small to moderate amounts of inorganic turbidity and color. Those waterbodies having a high inorganic particulate load or large amounts of suspended sediments or color would be expected to plot below the lines of best fit in Figs 2(B) and 4(B). The variation in the turbidity of various waterbodies plotted in Figs 2(B) and 4(B) probably accounts for much of the scatter about the lines of best fit.

There is also a number of waterbody characteristics peculiar to impoundments and some other waterbodies which must be evaluated before applying this approach, and adjusted if deemed appropriate. The depth of water in some reservoirs can be highly variable over an annual cycle. Depending on the depth-time pattern, it may be necessary, for example, to use the mean depth commonly found in early summer instead of full pool mean depth, in the models. For some reservoirs, water withdrawal is made from bottom waters. This may impact the nutrient loading since it may affect normal nutrient recycling by selectively removing hypolimnetic waters containing elevated phosphorus concentrations. Reservoirs and some lakes often have one or more arms or extensions. If these are clearly physically or chemically distinguishable from the main body of the reservoir, they are likely to have discrete nutrient load-response relationships and should be considered separately. It should also be noted that these arms often trap substantial parts of the P load to the reservoir; in the case of Lake Ray Hubbard (Lee *et al.*, 1978b), one of the arms trapped about 90% of the inflowing P load. If this occurs, the P load to the main body should be adjusted accordingly. Finally, because of the nature of the systems in which reservoirs are typically constructed, many tend to contain large amounts of suspended inorganic matter. As discussed previously, such reservoirs should be evaluated to determine if the inorganic turbidity is sufficient to alter algal growth in response to nutrient load or to affect the Secchi depth measurement.

Because of the extensive number and variety of types of waterbodies to which the Vollenweider-OECD modeling approach is applicable, if a waterbody does not appear to fit these relationships, it is usually an indication that either the value for one or more of the parameters has been incorrectly estimated or the users did not follow the procedures outlined by Rast & Lee (1978) of evaluating the waterbody's characteristics for any properties which would cause the nutrients to be used in a way different from the 300 waterbodies upon which the models were based (such as excessive inorganic turbidity, the presence of arms or bays which would trap nutrients, nutrient limitation, hydraulic-hydrologic characteristics, etc., as discussed previously) and making technically appropriate alterations to account for these properties. As in the case of the U.S. OECD study, for the several waterbodies which at first at first did not

appear to have a load response coupling similar to the other waterbodies, there were definable reasons for their behavior which could be corrected for in a technically valid manner for each waterbody based on knowledge of how the characteristic affects general nutrient utilization by planktonic algae, or by correcting errors in the data. It should be noted that such adjustments are technically appropriate to make and do not involve arbitrarily altering numbers to force the waterbody to fit. They need to be made under certain circumstances because this load-response modeling approach was developed based on theoretical nutrient utilization in completely mixed systems having a fairly simple morphology. The regressions (lines of best fit) were developed from data on waterbodies with similar "simple" characteristics or with unusual characteristics normalized. It would not be appropriate to use this approach for a waterbody having "unusual" characteristics without making such modification.

EXAMPLE OF THE APPLICATION OF OECD EUTROPHICATION MODELING APPROACH FOR WATER QUALITY MANAGEMENT

Rast & Lee (1978) provided an extensive discussion of how this modeling approach can be used for water quality management by applying it to a "hypothetical waterbody" (Figs 7 and 8) with assigned mean depth of about 18 m, hydraulic residence time of about 2.5 years, and having about 55% of its P load from point sources (i.e. domestic wastewater inputs). These characteristics are similar to those of Lake Erie during the mid-1970s. Figure 7 shows that by reducing the point source P load to the waterbody by 90% (which can readily be done through chemical precipitation of phosphorus at domestic wastewater treatment plants), there could be some relative improvement in the recreational acceptability of this waterbody. Further improvement would be seen after reduction of the nonpoint source P load in addition to the point source control.

By using the relationships shown in Figs 2 or 4, the improvement in water quality in the hypothetical waterbody can be determined in terms of chlorophyll concentration, Secchi depth, and hypolimnetic oxygen depletion rate. Figure 8 shows, for example, that 90% point source P load reduction would cause the chlorophyll concentration to decrease from an initial $7 \mu\text{g l}^{-1}$ to about $4 \mu\text{g l}^{-1}$; additional P load reduction from control of nonpoint sources by 40% would result in a mean chlorophyll concentration of about $3 \mu\text{g l}^{-1}$. Changes in chlorophyll concentration on this order (i.e. from 7 to $3\text{--}4 \mu\text{g l}^{-1}$) could be perceived by the public as improvements in water quality. Figure 8 also shows that the P load reduction that could result from detergent P bans in the watershed (i.e. 20-25% reduction of the domestic wastewater effluent P load) would not result in a perceptible change in eutrophication-related water quality in the lake.

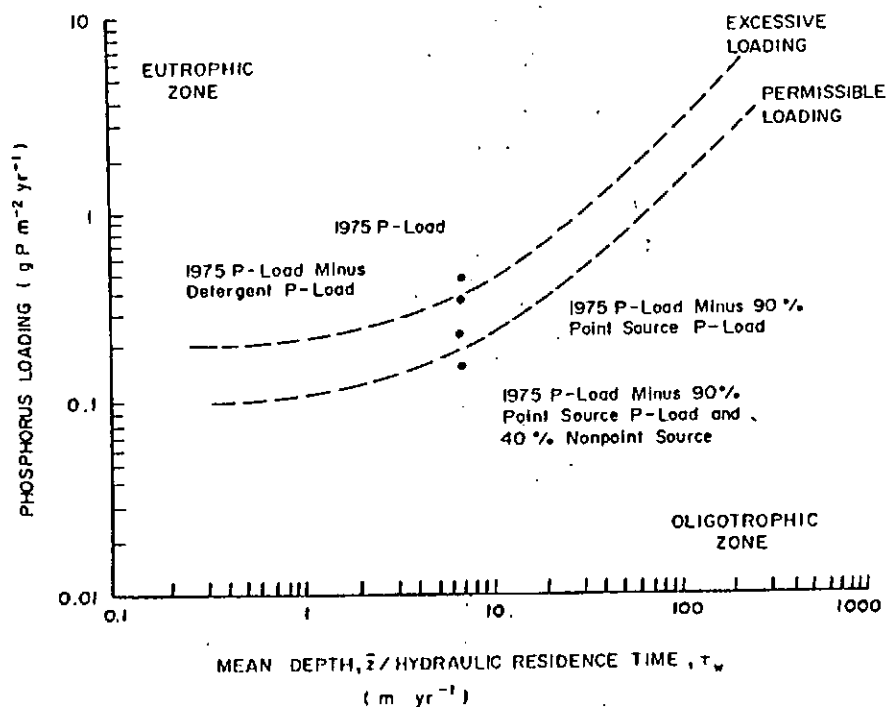


Fig. 7. "Hypothetical Waterbody" under different P load conditions applied to Vollenweider P loading-mean depth/hydraulic residence time relationship (after Rast & Lee, 1978).

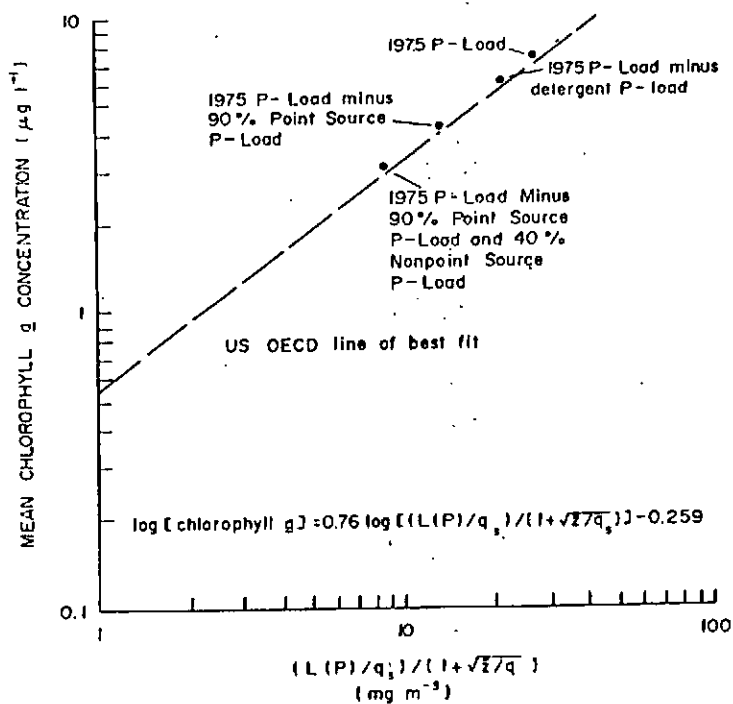


Fig. 8. U.S. OECD normalized P load Chlorophyll concentration relationship applied to "Hypothetical Waterbody" under different P load conditions (after Rast & Lee, 1978).

The authors and their associates have made use of this Vollenweider-Rast and Lee-OECD modeling approach in eutrophication-related water quality assessment and management for a number of reservoirs in the U.S. and Europe. For example, it was used for about a dozen Tennessee Valley Authority impoundments (Newbry *et al.*, 1979, 1980); the Great Lakes (Lee *et al.*, 1979); Lake Ray Hubbard, a water supply-recreation impoundment near Dallas, TX (Lee & Meckel, 1978; Lee *et al.*, 1978b); a group of waterbodies in Indiana (Lee & Archibald, 1981); Dillon Reservoir near Breckenridge, CO (Horstman *et al.*, 1980); as well as the Potomac Estuary (Lee & Jones, 1981c). It is currently being used by the Spanish government under the direction of the authors, as a basis for developing a eutrophication control program for Spain's more than 700 waterbodies.

Through their work with the American Water Works Association Quality Control in Reservoirs Committee, Lee & Jones (1981a,b) and Lee *et al.* (1981a) developed a series of manuals to aid in the use of the OECD eutrophication models for water quality management. One manual describes in detail how to estimate nutrient loads to waterbodies based on land use in the watershed; another describes how to assess the limiting nutrient and factors that should be considered in making this assessment as part of a water quality management program; and the third discusses the minimum waterbody and watershed monitoring programs to collect requisite data to apply the OECD eutrophication models. Lee *et al.* (1981b) have recently developed a step-by-step manual providing instruction on the proper application of this modeling approach for water quality management. Further, Lee *et al.* (1981c) have provided a discussion of how it can be used to comply with PL 92-500 Section 314 A lake classification-restoration requirements.

Estimating nutrient loads

Because of the importance of developing appro-

priate P load estimates in the use of the OECD (or any other) eutrophication modeling approach for evaluating the efficacy of nutrient load management options as part of eutrophication management programs, a summary of key points in nutrient load estimations is presented below. Lee *et al.* (1981a) and Rast & Lee (1981) present more detailed discussions on this topic and should be consulted for further information.

In order to use the Vollenweider-OECD eutrophication modeling approach to predict future water quality characteristics or to predict characteristics of impoundments which have not been constructed, it is necessary to make estimations of nutrient loadings. As discussed previously, the nutrient load to a waterbody generally consists of that derived from domestic wastewater input (point source), and that derived from nonpoint sources such as land runoff and the atmosphere. It is well established that over an annual cycle, the sediments are a sink for phosphorus (Lee *et al.*, 1977). In the U.S., the typical load of nutrient P in domestic wastewater treatment plant effluent (from plants not practicing P removal) is on the order of 1-1.5 kg P person⁻¹ year⁻¹. To estimate the amounts of nutrients coming from the land and the atmosphere, Rast & Lee (1978) developed, based on the literature and the U.S. OECD nutrient load and land use information, a set of nutrient export coefficients which have been found to have general applicability throughout much of the U.S. These are presented in Table 2. The export coefficient for a particular land use is multiplied by the area of land in the watershed devoted to that use, to determine the annual load from that type of land. With this information it is possible to estimate with an adequate degree of reliability for modeling associated with management purposes, the eutrophication-related water quality impact of, for example, increased urbanization (both in terms of increased nutrient input from land runoff and increased domestic wastewater treat-

Table 2. Watershed nutrient export coefficients

Watershed land use	Watershed export coefficient (g m ⁻² yr ⁻¹)
A. Total phosphorus	
Urban	0.1
Rural/agriculture	0.05
Forest	0.01
Other:	
Rainfall and dry fallout	0.02
B. Total nitrogen	
Urban	0.5 (0.25)*
Rural/agriculture	0.5 (0.2)*
Forest	0.3 (0.1)*
Other:	
Rainfall	0.8
Dry fallout	1.6

* Export coefficients used in calculating nitrogen loadings for waterbodies in the western U.S.

After Rast and Lee (1978).

ment plant nutrient load) on eutrophication-related water quality; of other altered land use or agricultural practices; of phosphorus removal at domestic wastewater treatment plants (90% removal of P in the effluent can be readily accomplished by precipitation with iron or aluminum salts); of decreased P content in laundry detergents (detergent P ban could result in a 20-25% reduction in domestic wastewater treatment plant effluent P concentration); and of the relative merits of various nutrients management strategies. It is important to note that while the Vollenweider-OECD models are based on total P load, because of the model formulation and the data base used, for many waterbodies the availability of phosphorus from various sources is in large part accounted for.

CONCLUSIONS AND RECOMMENDATIONS

The OECD Eutrophication Study nutrient load-eutrophication response models provide water quality managers with a reliable, relatively easy-to-use tool for developing eutrophication-related water quality management programs. Significant advantages of the OECD modeling approach over the "dynamic" modeling approaches used or proposed for use include the facts that the OECD approach has minimal data requirements and has demonstrated predictive capability.

The Vollenweider-OECD nutrient load-eutrophication response models should be incorporated into all 314-A lake and reservoir classification and management activities. This would include establishment where necessary of a water quality monitoring program such as that outlined by Lee & Jones (1981b).

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3.2.- DIMENSIONADO DEL ESTABLECIMIENTO DEPURADOR

3.2.1 - PARAMETROS EXTERNOS

El tamaño de una planta depuradora biológica urbana para eliminación de la materia orgánica está fundamentalmente gobernado por los siguientes parámetros externos:

- carga orgánica
- caudal de pico

los cuales, a su vez, surgen de otros que prevén las características del servicio o de la explotación:

- población conectada equivalente
- carga orgánica per cápita
- dotación o efluente per cápita
- factores de pico

En tanto que la concentración o fuerza del líquido cloacal puede utilizarse como parámetro de control.-

Dada su tradición, se utilizará como medida de la carga orgánica la DBO_5 , como masa de oxígeno, referida a la unidad de tiempo, o a la unidad de volumen y, a veces, al habitante equivalente.-

En este esquema, las previsiones del Anteproyecto Preliminar se traducen en las siguientes:

a) Parámetros del Servicio

Población un módulo.....	80.000 hab.
Carga orgánica per cápita.....	90 gr/h/día
Dotación media anual.....	318 l/h/día
Factor de pico estacional.....	1,55
Dotación media estacional.....	493 l/h/día
Factor de pico diario.....	1,20

b) Parámetros Externos para Diseño

Carga orgánica.....	7.200 Kg/ DBO_5 /día
Caudal de pico.....	1.972 m3/hora

c) Parámetro de Control

Concentración media anual.....	283 p.p.m.
Concentración media estacional.....	182 p.p.m.

COMENTARIO:

- El tamaño del 1er. módulo se juzga acertado. Debe tenerse en cuenta que para su eficiente aprovechamiento deberá emprenderse, simultáneamente, la ampliación de la red. Empero, aún cuando esas ampliaciones se retrasacen, la planta podría operar con toda eficiencia ya que cada módulo constará de dos - calles.-
- El tamaño definitivo del establecimiento ($80.000 \times 3 = 240.000$ hab.) a partir del año 2010 (o algo más tarde) que sería utilizado a pleno a partir del año 2020, impresiona algo generoso o excesivo.-

La ejecución del tercer módulo conforme a las pautas de este proyecto 1989 resulta incierta, debido no solo a la impresión de toda proyección sino también al progreso en la tecnología de depuración que cabe esperar en los años venideros. No obstante, la pretensión de resolver la totalidad del proyecto con las técnicas que hoy juzgamos más acertadas, crean condicionamientos al proyecto inmediato. Sobre todo en cuanto a la utilización del terreno. En materia de tratamiento de barros, por ejemplo, se pueden preveer progresos importantes en cuanto a la mecanización de los mismos. Además, la propia explotación de la planta durante 20 años brindará con seguridad conclusiones muy útiles a la hora de su ampliación.

- La carga per cápita adoptada de 80 gr/hab/día es, a nuestro juicio, alta. No se compadece con las habitualmente observadas en ciudades del país, aún las de alto grado de desarrollo. Como los Consultores lo señalan (Inf.Fiñal, p.4) en países altamente industrializados como Bélgica y Holanda se utiliza entre 54 y 65 gr/hab/día y en el Reino Unido se adopta entre 50 y 60; en tanto que en países en desarrollo, se utilizan entre 35 y 60 gr $\text{DBO}_5/\text{h/día}$ y en Brasil 50. Se advierte asimismo que en S.C. de Bariloche no hay fábricas

polucionantes de importancia.-

El 15.10.86 se extrajo muestra compensada en la estación de bombeo cloacal, la que arrojó una concentración de 81 p.p.m. DBO_5 (Est.Preliminar p.249).-

En ese mismo día, se aforaron los caudales en la E.B. hallándose un caudal medio de 631 m³/h (Est.Preliminar p. 220). La población conectada se evaluó en 19.000 habitantes, más los equivalentes al matadero que faena 50 vacunos diarios. Entonces:

. Población conectada en 1986.....	19.000 hab.
. Población equivalente del matadero.....	6.000 hab.
. Caudal medio diario al 15.10.86.....	631 m ³ /h
. DBO_5 muestra compensada del 15.10.86.....	81 p.p.m.

Resulta: $\frac{631 \times 24 \times 81}{25.000}$ 49 gr/h/día

Es decir que un valor como el adoptado, de 80 gr/hab/día sugiere un exceso de cobertura (Inf. Final, p.5) sobre todo si al momento de diseñar cada una de las etapas, se aplica un nuevo incremento que resulta de adoptar 180 ppm y, simultáneamente, 500 l/hab/día lo que representa una carga per cápita - de:

$$500 \text{ l/hab/día} \times 0,18 \text{ gr/l} = 90 \text{ gr/hab/día}$$

Cabe aclarar, además, que en estos días el Matadero Municipal se trasladará a sus nuevas instalaciones con tratamiento propio.-

- Coincidimos con la apreciación del caudal de pico formulada en el Anteproyecto Preliminar en el que se propone un valor de 2.000 m³/h para cada etapa de 80.000 habitantes.-

Ese caudal coincide con la capacidad máxima de transporte -- del colector costero existente en condiciones extremas de -- funcionamiento, esto es, con la boca de registro N° 16 (cruce Nireco) desbordando.-

Preferimos en cambio llegar a ese valor a partir de una dotación algo menor de 500 l/hab/día pero afectada de un factor de pico mayor. En la práctica, esta discrepancia no tiene -- ningún efecto en el dimensionado de la planta.-

Conclusión: Se advierte que una serie de posiciones de cobertura (crecimiento, concentración, dotación) se han -- potenciado para dar como resultado una carga orgánica que juzgamos algo excesiva. En cuanto a la -- carga hidráulica hay coincidencia. En concreto, -- proponemos trabajar sobre los siguientes números:

a) Parámetros del Servicio

. Población un módulo.....	80.000 hab.
. Carga orgánica per cápita.....	60 gr/h/día
. Dotación media estacional.....	400 l/h/día
. Factor de pico diario.....	1,5

b) Parámetros Externos para Diseño

. Carga orgánica.....	4.800 KgDBO ₅ /día
. Caudal de pico.....	2.000 m3/h

c) Parámetro de Control

. Concentración a caudal estacional.....	150 p.p.m.
--	------------

3.2.2 - PARAMETROS INTERNOS

Llamaremos así a los parámetros propios de la planta depuradora que cambian o varían según el proceso y el punto de funcionamiento que se adopte como hipótesis de trabajo.-

Descartada la derivación de las aguas al Río Limay, coincidimos con el Anteproyecto Preliminar en la elección del proceso (oxidación prolongada) y dentro de esa categoría coincidimos -- también con la elección del punto de funcionamiento, en tanto que propondremos algunas variantes en el diseño y dispositivos de las cámaras de aireación.-

El punto de funcionamiento viene caracterizado por los siguientes parámetros, principalmente biológicos:

- X = concentración de sólidos en el líquido mezclado, en términos de SST o de SSV por unidad de volúmen
- C_v = carga volúmica o carga orgánica diariamente aplicada por unidad de volúmen
- C_m = carga másica o carga orgánica diariamente aplicada por -- unidad de sólidos suspendidos totales
- I = índice de crecimiento de lodos, en literatura anglosajona SGI
- θ_c = edad del lodo o tiempo de residencia celular
- OC/L = relación oxígeno consumido o demandado vs. carga orgánica aplicada

Es decir que siguiendo a Vosloo, cuando no se indique lo contrario, se hará predominar:

- Sólidos suspendidos totales sobre Sólidos suspendidos volátiles
- Carga orgánica aplicada sobre Carga orgánica utilizada

Estos parámetros no son todos libres. Solo hay dos parámetros libres, los demás son dependientes. Elegiremos como parámetros libres a θ_c y X . Los demás quedan vinculados o bien por leyes que gobiernan los procesos biológicos o bien por las propias definiciones convencionales. Las leyes pueden ser descriptas mediante modelos matemáticos o como resultado de observaciones de campo, forma esta que, a la hora de diseñar, se impone por su fuerte apoyatura experimental. Esas relaciones empíricas entre parámetros, pueden buscarse en distintas fuentes, tales como: normas y manuales (por ej. ASCE-WPCF en Estado Unidos o -- ATV en Alemania Federal), tratadistas o autores clásicos (por ej. Imhoff o Metcalf-Eddy) o investigadores (por ej. Downing o Vosloo).

Desde mediados de la década del 70 y hasta ahora, los criterios expuestos por Vosloo sobre la base de observaciones realizadas por Wurhman, Downing y Hopwood cobraron gran prestigio --

entre diseñadores, en gran parte por su sencillez y pragmatismo. Hemos completado esa información con los criterios de ATV, que son más explícitos en materia de tratamiento biológico de líquidos completos o brutos, es decir sin sedimentación primaria.-

Los parámetros dependientes, se obtienen entonces al aplicar:

$$I = f_1 (\theta_c)$$

$$C_m = \frac{1}{I \theta_c}$$

$$C_v = C_m \times$$

$$OC/L = f_2 (\theta_c)$$

Además, Vosloo propuso un factor de pico para calcular la capacidad de oxigenación, que tiene en cuenta la falta de uniformidad en que la carga orgánica ingresa a la planta. Normalmente lo hace con fuertes picos (por caudal y por concentración) en horas del mediodía. Ese factor de pico también depende del punto de funcionamiento, es decir de θ_c :

$$\mu = \frac{\text{Tasa de Oxigenación Máxima}}{\text{Tasa de Oxigenación Media}}$$

$$\mu = f_3 (\theta_c)$$

En definitiva, se transcriben los parámetros que nos proponemos utilizar, en comparación con los que se desprende del dimensionado realizado en el Anteproyecto Preliminar:

- Parámetros Libres

	Utilizado en el Ant. Preliminar	Propuesto para el Ant. Definitivo
θ_c (días)	25	25
X (Kg SST/m3)	4	4
X (Kg SSV/m3)	-	3

- Parámetros Dependientes

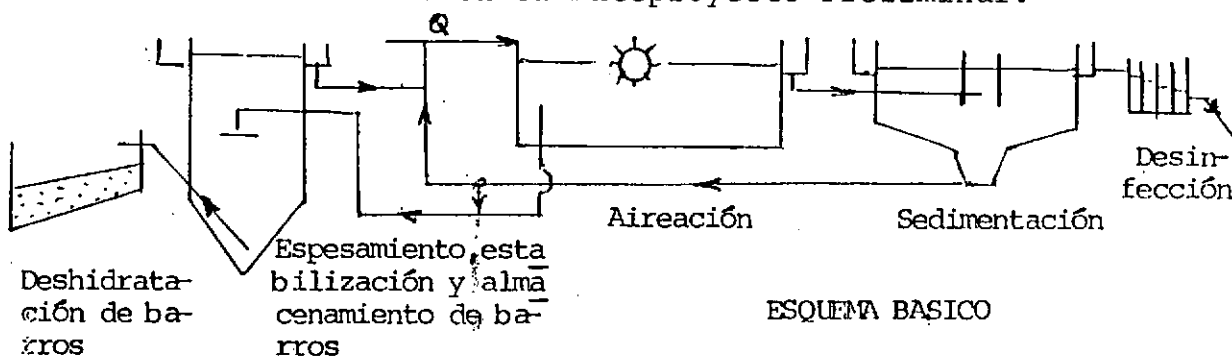
	Utilizado en el Ant. Preliminar	Propuesto para el Ant. Definitivo
$I \left(\frac{\text{Kg SST sintetizados}}{\text{Kg DBO}_5 \text{ aplicada}} \right)$	0,8	0,666
$C_m \left(\frac{\text{Kg DBO}_5 \text{ aplicada}}{\text{Kg SST día}} \right)$	0,05	0,06
$C_m \left(\frac{\text{Kg DBO}_5 \text{ aplicada}}{\text{Kg SSV día}} \right)$	0,08	0,08
$C_v \left(\frac{\text{Kg DBO}_5 \text{ aplicada}}{\text{m}^3 \text{ día}} \right)$	0,24	0,24
$OC/L \left(\frac{\text{Kg O}_2}{\text{Kg DBO}_5 \text{ aplicada}} \right)$	1,479	1,8
μ	NO	1,33

Comentario:

- En general, hay buena coincidencia en el tipo de planta depuradora que se pretende.-
- Nos inclinamos por una mayor capacidad de aireación que la prevista en el Anteproyecto Preliminar.-
- Como también se ha propuesto trabajar con una carga externa menor, la capacidad de aireación será a la postre casi coincidente.-

3.2.3 - PROCESO DE DIMENSIONADO

En este apartado se dimensionan las principales unidades de -- proceso para ir conformando el esquema general y su inserción en el terreno disponible. Se obvian los cálculos hidráulicos y los diseños de detalles. También se citan las unidades equivalentes desarrollados en el Anteproyecto Preliminar.-



DIMENSIONADO:

a) Reactores aeróbicos para 80 habitantes

	<u>Ant.Prel.</u>	<u>Ant.Def.</u>	<u>Ref.</u>
- Volumen.....	30.000	20.000	(1)
- Cantidad (Nº).....	2	2	
- Volumen unitario (m3).....	15.000	10.000	
- Profundidad (m.).....	4,50	3	
- Superficie Neta (m2).....	3.333	3.333	

b) Aireadores para 80.000 Habitantes

- Tasa de Oxigenación de campo, pico (Kg O ₂ /hora).....	443	480	(2)
- Tasa de Oxigenación Standard (Kg O ₂ /hora).....	654	670	
- Rendimiento al Eje (Kg O ₂ /KWh).....	2	2	
- Rendimiento en Línea (Kg O ₂ /KWh).....	1,75	1,6	
- Potencia al Eje (KW).....	327	335	
- Potencia en Línea (KW).....	372	420	
- Tasa de Oxígeno por metro de rotor (Kg O ₂ /h m).....		8	
- Longitud de rotor (m).....		84	(3)
- Densidad de Potencia Disipada (W/m3).....	10,9	16,75	(4)

c) Sedimentadores para 80.000 Habitantes

- Carga hidráulica superficial o velocidad ascensional (m/H).....	1	1	
- Superficie total (m2).....	1.973	2.000	(5)
- Cantidad (Nº).....	2	2	
- Superficie Unitaria (m2).....	987	1.000	
- Diámetro (m).....	35,45	35,68	
- Factor de recirculación en operación normal.....	1	0,8	
- Carga másica media (Kg/m2 h).....		4,8	(6)
- Carga másica pico (Kg/m2 h).....		6,13	(7)
- Velocidad de aproximación (m3/diam.).....		218	(8)

d) Silo de Barros para 80.000 Habitantes

	Ant.Def.	(Ref.)
- Caudal de purga con licor mezclado (m3/día)	800	(9)
- Caudal másico purgado (Kg/día)	3.200	(10)
- % de Metanización sobre los SST (%)	40	
- Permanencia buscada (días)	20	
- Masa acumulada en el silo (Kg)	38.400	(11)
- Concentración media estimada (Kg/m3)	20	
- Volúmen (m3)	1.920	(12)
- Altura (m)	18	
- Diámetro (m)	12	

e) Playas para 80.000 habitantes

- Materia seca anual (Kg/año)	700.800	(13)
- Carga sobre playas de secado (Kg/m2.año)	100	
- Superficie necesaria (m2)	7.000	

ACLARACIONES DEL DIMENSIONADO

Recordemos que:

λ : Carga orgánica entrante	4.800 Kg DBO ₅ /día
Q: Caudal de pico	2.000 m ³ /hora
Q _m : Caudal medio	1.333 m ³ /h
C _v : Carga volúmica	0,24 Kg DBO ₅ /m ³ .d
OC/L: Relación oxígeno-carga orgánica	1,8 Kg O ₂ /Kg DBO ₅
μ : Demanda de oxígeno pico/media	1,33
θ_c : Edad del lodo	25 días

De las referencias anteriores:

$$(1) V = \frac{L}{C_v} = \frac{4.800}{0,24} = 20.000 \text{ m}^3.$$

$$(2) CO = \mu \text{ OC/L } L = 1,33 \times 1,8 \frac{4.800}{24} = 479 \text{ Kg O}_2/\text{h}$$

$$(3) LR = 670/8 = 83,75 \text{ m.}$$

$$(4) DP = P_{\text{eje}}/V = \frac{335}{20.000} \times 1.000 = 16,75 \text{ W/m}^3$$

$$(5) S = \frac{Q}{VA} = \frac{2.000}{1} = 2.000 \text{ m}^2$$

$$(6) q_{\text{medio}}^* = \frac{(1 + R/Q_m) Q_m X}{S} = \frac{1,8 \cdot 1.333 \cdot 4}{2.000} = 4,8 \text{ Kg SST/m}^3 \text{ h}$$

$$(7) q_{\text{máx.}}^* = \frac{Q + R/Q_m Q_m}{S} X = \frac{2000 + 0,8 \cdot 1.333}{2.000} 4 = 6,13 \text{ Kg SST/m}^2 \text{ h}$$

$$(8) v_{\text{ap}}^{**} = \frac{Q}{2 \pi D} = \frac{2.000}{2 \times 3,14 \times 35} = 218 \text{ m}^3/\text{día m}$$

$$(9) Q_{\text{purga}} = \frac{V}{\theta_c} = \frac{20.000}{25} = 800 \text{ m}^3 \text{ IM/día}$$

$$(10) q_{\text{purga}} = Q_p \cdot X = 800 \times 4 = 3.200 \text{ Kg SST/día}$$

$$(11) M.B. = 3.200 (1 - 0,4) 20 = 38.400 \text{ Kg SST}$$

$$(12) V.S. = \frac{38.400}{20} = 1.920 \text{ m}^3$$

$$(13) M.S. = (1 - 0,4) 3.200 \times 365 = 700.800 \text{ Kg/año}$$

(*) Se acepta hasta 5 Kg/m² h para la carga media y hasta 7 Kg/m² h para la de pico (Metcalf-Eddy, 1985).

(**) Se acepta hasta 250 m³/día m (M.E. p.570)

COMENTARIO:

En líneas generales, hemos coincidido tanto en los parámetros internos como en el proceso de cálculo con el Anteproyecto -- Preliminar. La discrepancia más significativa radica en la apreciación de la capacidad de aireación, tema en el que arribamos a una potencia algo mayor para una carga orgánica menor.-

En cuanto al diseño del reactor, nos inclinamos por el típico de aireadores de eje horizontal, es decir, de flujo orbital y mezcla completa. El sistema carrousel está patentado por D.H.V. y conducirá seguramente a una restricción muy fuerte de la -- oferta (conocemos una patente muy similar estadounidense sistema Activox, a favor de Eric Johnson, con quien D.H.V. disputa los derechos).-

El sistema carrousel ha ganado mucho mercado en los países bajos debido sobre todo a la mayor profundidad que se le permiten a sus reactores, lo que entraña una economía de espacio -- valiosa en esos países. Otra ventaja que se le atribuye es el buen rendimiento en desnitrificación debido a la sucesión de regiones anóxicas, pero está bien demostrado que estos efectos no son privativos de los aireadores de eje vertical, sino que se pueden lograr también con aire comprimido o con aireadores de eje horizontal a condición de proveer un arreglo adecuado.-

Un aspecto interesante del sistema carrousel es la concentración de la potencia de aireación que se logra: requiere pocas máquinas y muy grandes. Esto hace caer los costos de inversión. Pero como todo tiene su contrapartida, ese menor costo de inversión se paga de dos maneras: con mayor fraccionamiento de la potencia se consigue mejor rendimiento energético -- (no es económico elevar mucho el nivel de oxígeno disuelto) y mayor flexibilidad (otro factor de economía para regímenes regulados) y también es más fácil y económico proveer máquinas en "stand by" para atender los inevitables períodos de mantenimiento.-

En el planteo del Anteproyecto Preliminar, para conseguir 500 CV en línea se instalan dos máquinas de 125 CV en cada reactor. En nuestro planteo, para conseguir 560 CV en línea se -- instalan cuatro máquinas de 70 CV en cada reactor. Con ese número de máquinas no es necesario proveer de stand by.-

En cuanto a la etapa de sedimentación, hemos coincidido en -- fraccionar la misma en dos calles con sedimentadores de 35 m. Es ese un diámetro crítico para el sistema convencional de barrido.-

Pudo optarse por un sedimentador de 50 m. (y posiblemente no falten esas iniciativas al momento de cotizar) pero a nuestro juicio, para conseguir eficiencia en ese diámetro se impone - el barredor de succión, que es más complicado y de oferta más restringida. Además se perderían las ventajas de tener dos calles por etapa.-

Como era de esperar, con concentraciones de 4 Kg SST/m³ y sedimentadores de 35 m. de diámetro comienza a mandar la carga másica superficial, que se ubica en los límites de lo aconsejado con una recirculación del 80%. Este hecho no aparece destacado en el Anteproyecto Preliminar debido a un error de --- planteo que se comete al no tener en cuenta la recirculación (p.226, Inf. Final).-

Es probable que se trabaje con recirculaciones mayores (por - ej. 100%) y ante eventualidades, más aún. Ello obligaría a tomar algunos recaudos en el diseño del sistema de barrido, tales como puente de dos radios y sedimentadores más profundos o con pronunciada pendiente de fondo.-

Southern African Branch

Some Factors Relating to the Design of Activated-Sludge Plants

By P. B. B. Vosloo, B.Sc., M.SA.Chem.I., M.SA.I.Chem.E. (Fellow)

INTRODUCTION

During the last decade or two, there has been a considerable increase in the knowledge and understanding of the activated-sludge process, with the result that it is now possible to predict, with reasonable accuracy, what effect various design parameters and methods of operation will have on the efficiency of the process.

Among the more important advances have been the application, mainly by Downing and his co-workers^{1,2,3,4,5} at the Water Pollution Research Laboratory, of the theories of population dynamics to the micro-organisms inhabiting an activated-sludge system, and the almost universal recognition of the importance of the sludge loading rate as the main parameter in the successful operation of an activated-sludge plant.

POPULATION DYNAMICS

A given organism, under a fixed set of conditions which are suitable for its growth, will multiply at a fixed rate, normally expressed in terms of a "growth constant", which is defined as the rate of increase in concentration per unit concentration present. For example, if the growth constant of a certain organism under certain defined conditions is given as 0.18 day^{-1} , it means that the total number of individuals of that type of organism will, under the specified conditions, increase by 18 per cent per day.

None of the aerobic organisms that perform the useful functions in an activated-sludge system is present in the influent in sufficient numbers to be of any practical value, but has to be developed and maintained in the sludge by natural multiplication. At the same time, as they increase by natural multiplication some of them are constantly being lost in the surplus sludge which is drawn off. (In a practical system it must be assumed that the wastage of surplus sludge is exactly equal to the production of new sludge in the system). From this it follows that, if the operating conditions are such that the rate of production of new sludge is greater than the growth constant of a certain organism, that organism cannot develop in the system, because the number of new individuals produced per day

would not be sufficient to make up for those lost in the surplus sludge.

The validity of these arguments has been proved beyond doubt in the case of the nitrifying organism, *Nitrosomonas*, by the experiments of Downing and his co-workers^{1,2}. It must be realized, however, that exactly the same principles apply to all the different types of organisms found in an activated-sludge system. In a high-rate process, where the rate of production of new sludge is also high, only those organisms with a high growth constant under the conditions pertaining in the system can develop, or, having been established under a different set of conditions, can be maintained under the new conditions.

This is of the utmost importance in research work on the activated-sludge process. If the operating conditions are changed so that there is a change in the rate of production of new sludge, it may take quite a long time for the sludge to adapt itself to the new conditions. If, for example, the rate of production of new sludge is increased to a value just higher than the growth constant of a certain organism which is present in large numbers in the sludge, this organism will gradually be washed out of the system, but it may take weeks before it has disappeared altogether. It is obvious, therefore, that the composition and properties of an activated sludge depend upon its immediate past history, rather than upon the conditions to which it is subjected at the moment.

SLUDGE LOADING RATE

It seems logical to accept that a given number of aerobic organisms, under a given set of conditions, will be able to oxidize only a limited amount of organic material, even if they are supplied with an excess of oxygen. In other words, the amount of biologically oxidizable material supplied per unit weight of micro-organisms in unit time can be considered as one of the major factors governing the efficiency of the treatment process.

The amount of biologically oxidizable material supplied in unit time can be expressed with a fair degree of accuracy as kg BOD applied per day, but unfortunately it is not practicable to make even a

rough estimate of the actual weight of micro-organisms in a system. These are confined almost entirely to the solid phase, however, and as the weight of solids can be estimated quite easily it is convenient to accept the latter as a measure, albeit only a very approximate one, of the weight of organisms present. In the absence of a better measure, therefore, the food to micro-organisms ratio may be expressed in terms of *kg BOD applied per day per kg mixed liquor suspended solids (MLSS) in the system*. In the rest of this paper, sludge loading rates will be expressed in these units, without specifying the units every time.

There is a tendency, especially in the USA, to express the loading rate in *kg BOD per day per kg mixed liquor volatile suspended solids (MLVSS)*, but the additional complication hardly seems justified. In the treatment of normal waste waters, there is no reason to believe that the figure for volatile suspended solids provides a better measure of the number of organisms than that for total suspended solids does. If it is argued that the use of the MLVSS value makes provision also for abnormal cases, such as those where the sludge may be "diluted" with inorganic material, it is as well to remember that the sludge could also be "diluted" with inert *volatile* matter, such as cellulose fibre, and that the presence of the latter would probably upset the figure for the weight of organisms per unit weight of solids just as badly as the presence of asbestos fibre would.

Relationship Between Sludge Loading Rate and Various Other Factors

New sludge production and sludge age. The rate of production of new sludge is most conveniently expressed as a growth constant, or as a percentage increase in existing weight per day. The sludge age, of course, is the reciprocal of this. For example, if the weight of sludge increases by 20 per cent per day (which means that the surplus sludge wasted per day must amount to 20 per cent of the weight of sludge in the system) the sludge age is 5 days.

The sludge loading rate is the main factor controlling the rate of production of new sludge. The relationship between the two was demonstrated by Hopwood and Downing⁶ in 1965, when they reported on the results of a series of experiments carried out on settled domestic sewage from Stevenage. The present author has collected and recalculated data from various sources, including the results obtained by Wuhrmann⁷ from an exhaustive series of tests carried out on a very weak settled sewage from Zürich over the years 1957-61. If in each case the percentage increase in the weight of sludge per day is plotted against the sludge loading rate, it is found that the curve with the best

fit corresponds very closely to the original one published by Hopwood and Downing⁶. Admittedly there is a great deal of scatter in the points, but this is only to be expected. For any particular value of the sludge loading rate the points for the sludge growth rate may lie between 0.7 and 1.4 times the value on the curve.

Some of the scatter of the points may be due to the presence or otherwise of large numbers of predators in the sludge, a factor which appears to be beyond the control of either the designer or the operator of the plant. If it were possible to omit the results obtained in cases where large numbers of predators were present, it is possible that the remaining points may lie much closer to the best fitted curve.

For settled sewage mainly of domestic origin, the relationship between sludge loading rate and sludge growth rate, obtained from plotting all the available data, is given very approximately by the following table. The sludge age and sludge growth index (weight of sludge produced per unit weight of BOD applied) were deduced for each sludge loading rate and are included in Table 1.

TABLE 1. RELATIONSHIP BETWEEN SLUDGE LOADING RATE, RATE OF PRODUCTION OF NEW SLUDGE, SLUDGE AGE AND SLUDGE GROWTH INDEX

Sludge loading rate (kg BOD per day per kg MLSS)	Rate of production of new sludge (percentage increase in existing weight per day)	Sludge age (days)	Sludge growth index (weight of sludge produced per unit weight of BOD applied)
0.1	5	20	0.50
0.2	15	6.7	0.75
0.3	25	4.0	0.83
0.4	35	2.9	0.88
0.5	45	2.2	0.90
0.6	55	1.8	0.92
0.7	65	1.5	0.93
0.8	75	1.3	0.94
0.9	85	1.2	0.94
1.0	95	1.0	0.95

This relationship will probably vary with temperature, but the variations that can be produced by other unknown factors are so great that it would be difficult to demonstrate the variations due to temperature changes. Furthermore, it must be realized that the growth rate of the sludge at a given loading rate must depend upon the nature of the waste to be treated. For example, no data are available for the rate of production of new sludge in extended-aeration plants, where the feed consists of crude sewage. It is reasonable to assume that, per unit weight of BOD, crude sewage contains a larger proportion of unoxidizable suspended material than settled sewage does, and that therefore, at equivalent sludge loading rates, the sludge growth rate should be higher than in the case of settled sewage.

As will be shown later, this relationship can be a powerful tool in the design of activated-sludge systems, and an appeal is made to research workers to establish and report on values for different types of waste.

BOD of effluent. While the sludge loading rate affects the degree of purification that takes place in an activated-sludge system, it is not the only factor that does so, and there is therefore only an approximate correlation between it and the percentage removal of BOD.

Downing^{4,8} has collected and plotted data from various sources, including those of Wuhrmann⁷, and finds that, below a sludge loading rate of about 0.4 a 90-95 per cent reduction in BOD can be expected, while at higher loading rates there is a steady decrease in the percentage removal of BOD.

Haseltine⁹ also collected operating data on numerous conventional activated-sludge plants in the USA, and plotted curves of lb BOD removed per 100 lb sludge solids against lb BOD applied per 100 lb sludge solids. He draws the following conclusion: "The curves indicate that the activated-sludge process alone, exclusive of primary treatment, can be expected to yield about 90 per cent BOD reduction, so long as the daily 5-day BOD load does not exceed about 50 lb per 100 lb of suspended solids in the aeration tank. If higher loads are applied, lower percentage reductions result". This conclusion is almost identical with that reached by Downing.

It would appear that if one aims at a BOD reduction of more than 90 per cent in the activated-sludge plant alone, exclusive of primary treatment, it is imperative that the sludge loading rate be kept below 0.4.

Nitrification. If the principles governing the population dynamics of an activated-sludge system are considered, as discussed briefly above, and it is accepted that there is a definite relationship between sludge loading rate and rate of production of new sludge, it is obvious that, for any given set of circumstances, there must be a sludge loading rate below which *Nitrosomonas* will multiply in the sludge, but above which they will be "washed out" systematically in the surplus sludge.

In order to establish what this critical sludge loading rate is, it is only necessary to know the growth constant of *Nitrosomonas* under the conditions prevailing in the system. For example, if the growth constant of *Nitrosomonas* were 0.25 day⁻¹, the critical sludge loading rate would be

0.3 kg, since at this loading rate the growth rate of the sludge is also 25 per cent per day.

Unfortunately, the growth constant of *Nitrosomonas* is influenced by a number of factors, and can assume widely different values in different circumstances. Firstly, it is very sensitive to the presence of toxic or inhibiting substances in the waste water. There is evidence, for instance, that its value in river waters is much higher than in sewage. Secondly, it varies with pH. Thirdly, it depends very largely on temperature. Downing and Knowles⁵ state that in mixtures of activated sludge and domestic sewage in which the pH value is in the range from about 7 to 8, the growth constant of *Nitrosomonas* (in days⁻¹) will vary with the temperature, t°C, in the range from 5°C to 25°C, approximately, according to the empirical equation:

$$k = 0.18e^{0.12(t-15)}, \quad (R = 0.18 \text{ at } t = 15^\circ\text{C})$$

i.e. k increases by about 13 per cent for each 1 degC rise in temperature.

The values calculated from this equation are given in Table 2.

TABLE 2. RELATIONSHIP BETWEEN GROWTH RATE OF *Nitrosomonas* AND TEMPERATURE

Temperature (°C)	Growth rate of <i>Nitrosomonas</i> (percentage increase in existing number per day)
10	10
15	18
20	33
25	60

From Tables 1 and 2 the critical loading rate above which nitrification cannot be maintained can be deduced for any temperature that may be encountered in practice in South Africa. For example, if nitrification is to be maintained at 20°C, where the rate of increase of *Nitrosomonas* is 33 per cent per day, the sludge loading rate must not exceed 0.38 kg, where the rate of production of new sludge is also 33 per cent per day. If the mean temperature drops to 15°C, however, the sludge loading rate will have to be kept below 0.23 kg if nitrification is to be maintained.

Settleability of sludge. The settleability of the solids in an activated-sludge system is usually expressed in terms of the sludge volume index (SVI). This is defined as the volume, in ml, occupied by 1 g of dry solids in the sludge layer formed after 30 min quiescent settlement. The solids concentration in the sludge layer can, of

course, be expressed as $\frac{10^6}{\text{SVI}}$ mg/l.

$$= 25\%/\text{day} \approx 25\text{ g/g MLSS} \cdot \text{d}$$

$$\text{the threshold} = \text{sludge age}$$

The value of the SVI is of the utmost importance, since in the final analysis it is this that determines the maximum MLSS concentration that can be maintained, and therefore the minimum volume of aeration tank required to keep the sludge loading rate below the desired value.

There is, of course, a definite relationship between the concentration of mixed liquor suspended solids (MLSS), the concentration of return sludge suspended solids (RSSS) and the sludge return ratio (p). It is obvious that, under steady conditions, the weight of solids entering the secondary sedimentation tank in unit time must equal that leaving it in unit time. In other words:

$$\text{MLSS} (1 + p) = \text{RSSS} \times p$$

$$\text{or MLSS} = \frac{p}{1 + p} \text{RSSS}.$$

If the settling conditions and the sludge thickening time in the secondary sedimentation tank were the same as those in the cylinder used for determining the SVI, the value of RSSS would be equal to $\frac{10^6}{\text{SVI}}$. This is not the case, however, and all that can be said is that:

$$\text{RSSS} = r \times \frac{10^6}{\text{SVI}} \text{ mg/l,}$$

where r is a factor depending mainly upon the retention period of the return sludge in the sludge layer in the secondary sedimentation tank, and to some extent also upon the actual depth of this sludge layer (as distinct from its volume). The retention period in the sludge layer in the settling tank naturally depends on the rate of flow of return sludge (i.e. on p), and on the volume of that proportion of the tank occupied by sludge in the process of being thickened.

Substituting for RSSS in the previous equation, one arrives at the relationship:

$$\text{MLSS} = \frac{pr}{1 + p} \times \frac{10^6}{\text{SVI}} \text{ mg/l,}$$

which clearly demonstrates the effect of SVI and sludge return ratio on MLSS concentration. It must be clearly understood that, while a high SVI will lower the figure for the maximum attainable MLSS concentration, a low SVI will not necessarily result in a high MLSS concentration. Unless the draw-off of excess sludge is stopped or reduced when the SVI decreases, this decrease will merely result in a "shrinkage" of the sludge layer in the final sedimentation tank, a consequent reduction in the retention period of the return sludge in this layer, and a decrease in the value of the factor r , leaving the MLSS concentration unchanged.

Under these conditions the operator is at liberty, of course, to increase the MLSS concentration by temporarily reducing the wastage of excess sludge. This will increase the volume of the sludge layer, the retention period, the value of r and so also the MLSS concentration.

If there were a fixed relationship between the sludge loading rate and the SVI, and if the values of r under different operating conditions were known, it would be possible to predict with certainty what the maximum attainable MLSS concentration at the desired sludge loading rate and the given sludge return ratio would be, and one would be able to install the minimum size of aeration tank in the certain knowledge that the weight of solids in the system would never decrease below the required amount.

Unfortunately there is practically no information on the value of r for different operating conditions. In the author's experience it would appear that it can be as high as 2.0 where the SVI is high and the retention period of the return sludge in the sedimentation tank sludge layer is 4 or 5 h, while it may be as low as 0.5 where the SVI is low and the retention period in the sludge layer is only 20 or 30 min. A secondary sedimentation tank designed for effective clarification of the final effluent will always be capable of providing the return sludge with a retention period of at least $1\frac{1}{2}$ h in the sludge layer, provided the sludge return ratio does not exceed about 1.5 and the sludge layer is allowed to come up to about half the tank volume. Under such conditions it may be assumed that r will have a value of about 1.2.

While the sludge loading rate undoubtedly has an effect on the SVI, there does not seem to be a fixed relationship between the two. The Water Pollution Research Laboratory^{10,11} has published at least two curves showing how the SVI varied with the sludge loading rate in various experiments carried out by the Laboratory. These two curves agree only in so far as they indicate that the highest values for the SVI (300 and 250 respectively) were obtained at sludge loading rates between 0.2 and 0.5. The shapes of the curves are different, however, and the points are so scattered about the mean curve that it is impossible to predict with any degree of certainty what the SVI would be at a given sludge loading rate.

A similar curve was published by Stewart and Ludwig¹² in the USA in 1962. It shows quite low values for the SVI at sludge loading rates below 0.35, with a rapid rise thereafter to a maximum of 380 at about 0.5, followed by a slow decrease to a minimum value at loading rates above 3.0. The values are still very high at 0.8 and 1.0, where the

curves of the Water Pollution Research Laboratory show low values.

In this connection, Haseltine⁹ makes the following statement:

An average Mohlman Index (SVI) of about 100 is generally considered very satisfactory at conventional activated-sludge plants of the diffused-air type. However, Pearce and his committee show that even at plants where the average index is about 100 the daily index may range from a minimum of 30 to a maximum of 300 to 400. The maintenance of a constant BOD-to-solids loading would probably reduce much of that variation but, in view of the highly variable BOD input to most plants, such an ideal situation is impossible of attainment. Therefore the designer should provide for satisfactory operation at a Mohlman Index considerably above 100. It is suggested that moderate to large plants be designed for an index of 250, and small plants, where continuous skilled operation is improbable, be designed for an index of 350.

Because of the evidence that the SVI tends to increase to high values at sludge loading rates above about 0.3, it would seem advisable to design for loading rates below this value. If this is done, it will probably be unnecessary to make provision for SVI values in excess of 250 ml/g.

Therefore, even if a satisfactory effluent could be obtained at a sludge loading rate above 0.3, no advantage would be gained by designing for such a higher sludge loading rate. The aeration tank volume would have to be about the same as before, since the possibility of having a lower MLSS concentration than before would have to be taken into account. This argument does not apply to high-rate plants, since it is possible to have low SVI values at sludge loading rates of 0.8 and higher.

Oxygen requirements. On mixing sewage with activated sludge most of the colloidal and suspended material is adsorbed on to the sludge. Some of this adsorbed material may be oxidized in the aeration tank, but some of it is removed from the system with the surplus sludge without having been oxidized completely. Consequently, the longer the sludge is retained in the system (i.e. the greater the sludge age), the greater must be the proportion of the adsorbed suspended and colloidal material that is oxidized, and the smaller the proportion that is drawn off with the surplus sludge.

It follows that the weight of DO required per unit weight of BOD removed in the system must depend to a large extent on the sludge age, or in other words on the rate of production of new sludge, which itself is governed by the sludge loading rate.

At high sludge loading rates, where a large amount of surplus sludge is formed and therefore a large proportion of the BOD-producing material is drawn off from the system, less than 1 kg of DO is required per kg of BOD removed. At low sludge loading rates, on the other hand, the adsorbed suspended and colloidal material may be retained in the system for longer than 5 days, and therefore undergo more extensive oxidation and consume more oxygen in the process than they would have done in a 5-day BOD test. In such cases, therefore, more than 1 kg of DO will be required per kg of BOD removed.

Although a great deal of information has been published on oxygen requirements in the activated-sludge process, very little of this is presented in such a form that one can deduce a relationship between the weight of oxygen required per unit weight of BOD removed and the sludge loading rate. Apart from this, much of the information that has been published is contradictory.

In attempting to estimate the oxygen requirements it will be advisable to omit the oxygen required for nitrification and to estimate this separately for those cases where nitrification can occur. The reason for this is that there will be a discontinuity in the curve for total oxygen requirement against sludge loading rate at the loading rate below which nitrification can take place under the conditions prevailing in the plant.

From the available evidence it appears that, for carbonaceous oxidation alone, the ratio of DO required to BOD removed is about 1.6 at the one extreme, where the sludge age is 20 days or more, and about 0.65 at the other extreme, where the sludge age is 1 day or less. To obtain one point in between, it seems reasonable to assume that for a

TABLE 3. RELATIONSHIP BETWEEN SLUDGE LOADING RATE, SLUDGE AGE, AND DISSOLVED OXYGEN REQUIREMENTS FOR CARBON, (A)

Sludge loading rate (kg BOD applied per day per kg MLSS)	Sludge age (days)	Dissolved oxygen requirements (kg per kg BOD removed)
0.10 and less	20 and more	1.60
0.15	10.0	1.38
0.20	6.7	1.22
0.25	5.0	1.10
0.30	4.0	1.00
0.40	2.9	0.88
0.60	1.8	0.74
0.80	1.3	0.68
1.00 and more	1.0 and less	0.65

sludge age of 5 days the oxygen consumed would be somewhat more than that consumed in a 5-day BOD test, since in the latter, without any specially developed aerobic culture like that used in the activated-sludge process, the oxidation is probably less efficient than in an activated-sludge system. It is therefore assumed that at a sludge age of 5 days 1.1 kg of DO is required per kg BOD removed. Since the sludge loading rates giving rise to sludge ages of 1, 5 and 20 days, respectively, are 1.0, 0.25 and 0.1 kg BOD per day per kg MLSS, it follows that for the latter loading rates, in that order, the oxygen required for carbonaceous oxidation alone must be 0.65, 1.1 and 1.6 kg per kg BOD removed. Interpolation between these three points gives the values set out in Table 3.

APPLICATIONS TO DESIGN OF CONVENTIONAL ACTIVATED-SLUDGE SYSTEMS

In the design of any activated-sludge system the main items to be decided upon are size of aeration tank, rate of sludge return, capacity of aeration equipment, and size of final sedimentation tank. These will now be considered in turn for the hypothetical case of a conventional activated-sludge system required to treat 1.1 mil gal/d (5000 m³/d) of settled sewage with a BOD of 200 mg/l.

Size of aeration tank and sludge return ratio.

In order to calculate the size of aeration tank to be provided it is necessary to decide on the maximum sludge loading rate that can be applied to produce an effluent of the desired quality. The quality of effluent to be expected at various sludge loading rates was discussed above, and the conclusion was reached that the sludge loading rate has to be kept below 0.4 kg if better than 90 per cent BOD removal is to be obtained. If nitrification is to be maintained, even when the temperature drops to 15°C, the sludge loading rate should be kept to below 0.23, and if high SVI values are to be avoided it should be kept to below 0.30.

In South Africa, the regulations framed under the Water Act No. 54 of 1956 require, *inter alia*, that effluents shall contain not more than 10 mg/l of ammoniacal nitrogen. In order to achieve this, an activated-sludge system would have to be capable of nitrification in all normal circumstances. For most parts of the country this means that nitrification would have to be maintained, even if the temperature dropped to 15°C. Consequently, the sludge loading rate would have to be kept to below 0.23.

In the hypothetical case under consideration

the applied load would be 1000 kg BOD per day, and if the sludge loading rate is not to exceed 0.23 the system would at all times have to contain at least 4350 kg of MLSS. During operation, a proportion of the solids would always be in the sedimentation tank, so the aeration tank would not necessarily have to contain the full quota of solids required. Attempts have been made to estimate the weight of solids that the sedimentation tank could normally be expected to contain, but this depends on so many factors (SVI, relative sizes of aeration and sedimentation tanks, variations in flow rates and in sludge return ratios during the day, etc.) that it is impossible to arrive at even an approximate estimate. In any case, the solids in the sedimentation tank usually constitute quite a small proportion of the total weight of suspended material, and it is therefore suggested that these solids be left out of reckoning and be regarded as providing an additional factor of safety. In the case of an influent with a very low BOD the sedimentation tank will be large compared to the aeration tank, and in such a case the designer may wish to make some allowance for the solids in transit in the sedimentation tank.

In the case in point it is assumed that the aeration tank alone will have to contain at least 4350 kg of solids at all times, which means that it must be large enough to contain this amount under the most unfavourable conditions, i.e. when the SVI is at its maximum and the MLSS concentration at its minimum value, the latter still being the maximum attainable under the unfavourable conditions. This MLSS concentration can be calculated from the previously derived equation:

$$MLSS = \frac{pr}{1+p} \times \frac{10^6}{SVI} \text{ mg/l.}$$

It was decided earlier that the maximum SVI to be provided for is 250 ml/g, and that the value of r in these circumstances will be approximately 1.2. In order to calculate the maximum attainable MLSS concentration, therefore, one merely has to select a value for the sludge return ratio, p , making this as large as is practicable or useful. It is necessary at this stage to consider the factors affecting the choice of p .

When the SVI increases, the sludge layer in the secondary sedimentation tank will expand, in order to increase the retention period of the return sludge in this layer, raise the value of r and maintain the MLSS concentration. This process will continue until the sludge level reaches the overflow level of the tank, after which a further increase in the SVI will result in the carry-over of solids in the effluent and a decrease in the MLSS concentration.

At this stage an increase in the sludge return ratio would increase the factor $\frac{p}{1+p}$, tending to increase the MLSS concentration or to arrest its decrease. It must be borne in mind, however, that an increase in the sludge return flow will decrease the retention period in the sludge layer and so decrease the value of r , partially undoing the benefit derived from increasing p . As p is increased further, a stage will be reached where the increase in the value of $\frac{p}{1+p}$ will be neutralized completely by the concomitant decrease in r . It is not certain at what value of p this stage is reached, but the consensus of opinion seems to be that there is little benefit in increasing the sludge return ratio beyond 1.0 or 1.5. This becomes quite credible if one considers the following.

With a reasonably high MLSS concentration such as 4000 mg/l, and a low sludge return ratio such as 0.5, the sedimentation tank has to concentrate the sludge to 12 000 mg/l ($\frac{1+p}{p} \times \text{MLSS}$), which may be very difficult unless the SVI is low. If the sludge return ratio is increased to 1.5, the tank only has to concentrate the sludge to 6700 mg/l, which is quite a reasonable figure. A further increase in the value of p to 2.0 will only allow the return sludge concentration to drop to 6000 mg/l, which is rather a small benefit considering the relatively large increase in the sludge return ratio. With a high SVI it may be no more difficult to concentrate the sludge to 6700 mg/l with a sludge return ratio of 1.5 than to concentrate it to 6000 mg/l in the shorter retention period in the sludge layer occasioned by increasing the sludge return ratio to 2.0.

It must be realized that, in the equation, p does not refer to the average sludge return ratio (which is the ratio of the sludge return rate to the average dry-weather flow), but to the actual ratio that applies at any particular time. If the sludge return rate is kept constant over the 24 h, the value of p will increase during periods of low flow, resulting in an increase in the MLSS concentration and a reduction in the amount of solids contained in the sedimentation tank. This increase will be less than the anticipated amount, however, due to a decrease in the value of r , occasioned by a reduction in the volume of the sludge layer in the sedimentation tank. During periods of high flow, on the other hand, the value of p will decrease, leading to a decrease in the MLSS concentration in the aeration tank and an increase in the amount of suspended matter held in the sedimentation tank. The latter factor will increase the value of r , which will limit the decrease in MLSS concentra-

tion to some extent. If the incoming flow rate is high enough and remains so for a sufficient length of time, the sedimentation tank may fill up with sludge, resulting in a loss of solids by carry-over with the effluent, unless the return sludge rate is increased in order to raise the value of p . Fortunately the transfer of solids to the sedimentation tank is a gradual process and in most cases the incoming flow will have dropped before the sedimentation tank is full of sludge.

If 1.5 is accepted as the maximum useful value of p , and this is substituted for p in the above equation, together with the previously selected values for SVI and r , it is found that the maximum attainable MLSS concentration under unfavourable conditions will be 2880 mg/l. This means that, with an SVI of 250 ml/g, it should be possible to maintain an MLSS concentration of 2880 mg/l at average dry-weather flow. At low flows the MLSS concentration will rise above this figure, but at a wet-weather peak of $3 \times \text{d.w.f.}$ the value of p will decrease to 0.5 and the MLSS concentration will tend to decrease to 1600 mg/l. It will not quite reach this figure, however, as the value of r will tend to increase.

At times when the SVI is below 250 it will be possible to operate at higher MLSS concentrations, and therefore at lower sludge loading rates. It will also be possible, during periods of high wet-weather flow, to maintain an MLSS concentration reasonably close to the desired figure. If however such flows coincide with high SVI values, or they are sustained for too long a period, it may become necessary to bypass part of the flow in order to prevent loss of sludge, which would lead to higher sludge loading rates and most probably to further increases in the SVI, thus setting up a vicious circle.

In the hypothetical case under consideration, if provision is made for a maximum sludge return rate of 1.5 times average dry-weather flow, i.e. a sludge return ratio of 1.5, it may be accepted that there will be times when the maximum attainable MLSS concentration will be 2880 mg/l. In order to ensure that, even at such times, the aeration tank still contains 4350 kg of suspended solids; its volume will have to be 1510 m³, which corresponds to a volumetric loading rate of about 0.66 kg BOD per day per m³ aeration tank (41 lb per 1000 ft³).

A reduction in the sludge return ratio would give rise to an increase in the value of r . If it were possible, with a sludge return ratio of 1.0, to get the value of r up to 1.44 when the SVI is 250, the maximum attainable MLSS concentration would remain unchanged at 2880 mg/l and the necessary aeration tank volume would still be 1510 m³. Even

if r could be increased to 1.44 in the same sludge layer volume in the sedimentation tank as before, the retention period in the sludge layer would be 50 per cent longer, which would certainly not be an advantage. At the same retention period as before and therefore at the same value of r , the maximum attainable MLSS concentration when the SVI is 250 would be only 2400 mg/l, and the aeration tank volume would have to be increased from 1510 m³ to 1810 m³.

It must be pointed out that the operator of an activated-sludge plant is not necessarily tied down to the sludge loading rates envisaged by the designer. He may, for example, reduce the MLSS concentration and increase the sludge loading rate in order to reduce the oxygen requirements and so save power. Under favourable conditions, e.g. a temperature of 20°C, this could probably be done without detriment to the BOD of the effluent, or loss of nitrification. The sole aim of the designer should be to provide a plant that will enable the operator to produce an acceptable effluent, even when conditions are unfavourable.

Capacity of aeration equipment. Earlier on, an estimate was made of the weight of DO required per kg BOD removed at various sludge loading rates. From Table 3 the daily oxygen requirement for carbonaceous oxidation can be estimated. For example, in the hypothetical case used above, where the flow is 5000 m³/d, the BOD of the influent is 200 mg/l and the sludge loading rate is 0.23, the oxygen requirement would be 1.15 kg per kg BOD removed. If one assumes a maximum BOD reduction of 95 per cent, the weight of BOD removed per day would be 950 kg, which would require 1090 kg of DO. To this must be added the oxygen required for nitrification. Supposing the influent contains 30 mg/l of ammoniacal nitrogen, 20 mg/l of this, or a total of 100 kg/d, would have to be converted into nitrate. This would require 100×4.5 , or 450 kg of oxygen per day, making the total oxygen requirement 1540 kg/d. The average hourly requirement would therefore be 64.2 kg, but it would not be sufficient to install aeration equipment with this capacity only since there can be considerable fluctuations in the oxygen demand over the 24 h of the day. The peak flow may be twice the average, and the BOD of the influent during peak hours may be $1\frac{1}{2}$ times the average. This means that the peak load may amount to 3 times the average load, while the minimum load may be only about $\frac{1}{3}$ of the average.

If aeration were applied at the average required rate over the full 24 h there would be long periods during the day when the DO concentration would drop to zero, while there would be times during off-peak periods when the concentration

would rise to quite high values, and considerable energy would be wasted in trying to introduce oxygen into a liquid in which the concentration is already approaching saturation.

It is essential, therefore, that provision be made for varying the rate of oxygenation over the 24 h in relation to the variations in the incoming load. The variations in rate of oxygenation need not be in direct proportion to the variations in the incoming load, however, in view of the considerable balancing capacity that the aeration tank and the sludge solids have. The oxygen reserve in a large aeration tank containing 2 or 3 mg/l of DO is not insignificant, while a large amount of activated-sludge solids can adsorb quite a lot of suspended and colloidal material for oxidation at a later stage. The lower the sludge loading rate the greater will be the volume of the aeration tank and the weight of activated sludge solids in it, and presumably therefore also the greater will be the balancing effect on the oxygen demand.

No information on this aspect of aeration could be found in the literature, and once again there seemed to be no alternative but to put forward estimates that appeared to be reasonable. Such estimates are given in Table 4, together with those for the weight of DO required per unit weight of BOD removed, which are repeated from Table 3.

TABLE 4. RELATIONSHIP BETWEEN SLUDGE LOADING RATE, DISSOLVED OXYGEN REQUIREMENTS AND RATE OF OXYGENATION

Sludge loading rate (kg BOD applied per day per kg MLSS)	Dissolved oxygen requirements (kg per kg BOD removed)	Ratio of maximum to mean oxygenation rate	Ratio of minimum to mean oxygenation rate
0.10 and less	1.60	1.5	0.5
0.15	1.38	1.6	0.5
0.20	1.22	1.7	0.5
0.25	1.10	1.8	0.5
0.30	1.00	1.9	0.5
0.40	0.88	2.0	0.5
0.60	0.74	2.2	0.5
0.80	0.68	2.4	0.5
1.00 and more	0.65	2.5	0.5

From Table 4 it appears that, in the hypothetical case under consideration, the maximum capacity of the aeration equipment should be 64.2×1.75 , or about 112 kg DO per hour, and that provision should be made for varying the capacity between this figure and a lower limit of 64.2×0.5 , or about 32 kg per hour.

With the reasonably reliable DO meters that are available today it has become possible to adopt automatic control of the aeration equipment in order to maintain any desired DO concentration in the mixed liquor. Alternatively, a DO meter may be installed with electrodes in various sections of the aeration tank and an indicating dial and selector switch in a convenient centralized position. This would enable the operator to control the aeration equipment manually. Dissolved oxygen should be maintained at about 2 mg/l throughout the aeration tank at all normal times, but it may be allowed to drop to 0.5 mg/l at peak demand periods, and to rise to 3 mg/l at times of minimum demand.

Size of sedimentation tank. A high sludge volume index is one of the most serious problems that can be encountered in the operation of an activated-sludge plant. It may result in the carry-over of sludge with the effluent, leading to a reduction in the amount of sludge in the system and a consequent increase in the sludge loading rate. This, in turn, usually causes a further increase in the SVI, resulting in a greater loss of sludge.

This snowballing effect may be triggered off by a loss of sludge occasioned by an abnormally high flow through the plant for an hour or two, at a time when the SVI is somewhat above normal. Because of this danger it is recommended that the upward velocity in the final sedimentation tank, based on the rate of flow of the effluent leaving the tank, should not be higher than 4 ft/h (1.2 m/h) at peak dry-weather flow, or 6 ft/h (1.8 m/h) at peak wet-weather flow, with the proviso that, should the latter condition result in a carry-over of sludge, it should be possible for the operator to reduce the rate of flow through the plant to the point where this will no longer occur.

If, in the case under consideration, the peak dry-weather flow is taken as twice the average, the secondary sedimentation tank has to have an area of 348 m². If the effective depth is 3 m and the sludge layer is allowed to occupy half this depth, the retention period of the return sludge in the sludge layer will be about 1.7 h at times when the return sludge rate is 1.5 times the average dry-weather flow. This should allow the value of r to rise to at least 1.2.

CONCLUSION

The approach to the problems connected with the design of activated-sludge plants, as set out above, is not claimed to be an ideal one. It is being put forward, however, in an attempt to simplify and rationalize a rather complicated subject, and to

highlight the regions in which additional experimental data would be of the greatest value.

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The above Paper was presented for discussion at a meeting of the Southern African Branch held in Johannesburg on 19 June 1969.

DISCUSSION

Mr. D. W. OSNORN (Johannesburg City Council), opening the discussion, said that Mr. Vosloo had given an excellent summary of current literature on activated sludge, had indicated areas of deficiency in our knowledge, and had drawn on his own extensive practical experience to fill some missing gaps.

Reliable data on the rate of growth in extended-aeration plants were required, but plant-scale derivation of this information was rendered difficult by the inability of works operators to obtain a representative sample of the incoming sewage.

Factors affecting the SVI were still not clearly understood, and it was not impossible that the high intake (and also wastage) of certain carbohydrate foods (mainly maize meal) by African races could have an

adverse effect on the operation of activated-sludge plants in South Africa. This needed investigating.

Because of the temperature susceptibility of the growth constant of *Nitrosomonas*, he enquired whether Mr. Vosloo would recommend an increase in MLSS concentration during winter operation as a compensatory technique to maintain nitrification, or whether he would rely on the initial design of the plant being such that nitrification would be maintained even at low winter temperatures.

In regard to oxygenation, Mr. Osborn complimented the author on introducing the concept of diminishing oxygen requirements for diminishing sludge age, but enquired whether oxygen requirements so calculated would result in the supply of sufficient air to the mixed liquor to ensure adequate mixing.

The concept of sludge age as used by the author differed from, and was perhaps more logical than, the definition given in many textbooks.

The procedures outlined in the paper were adequate for determining the total volume of aeration tank required, but did not indicate the desirability or otherwise of splitting this capacity into two or more units. Mr. Vosloo's views on the separation of the carbonaceous and nitrogenous oxidation steps were therefore solicited.

The paper was a useful South African contribution to the subject, and data presented were likely to become the basis of design for future activated-sludge plants, at least in that country.

Mr. E. J. HALL (Johannesburg City Council) said that increasing the MLSS concentration to about 5000 mg/l in order to reduce foaming would mean that, on a 1:1 recirculation basis, the return sludge would have to contain about 10 000 mg/l or 1 per cent of solids. He considered this to be too high.

Mr. P. G. J. MEIRING (Pretoria) said that Mr. Vosloo's paper was an excellent preparation for the future, as South Africa would soon see many activated-sludge plants in operation. Mr. Meiring considered that there were many places in South Africa where the average temperature of the mixed liquor would drop to below 15°C in winter. It would be uneconomical to design plants to maintain nitrification down to, say, 10°C and it was hoped that the authorities would not insist that the ammonia nitrogen in purified effluents be kept below 10 mg/l throughout the winter months. He also enquired about the difference between total oxidation and extended-aeration.

REPLY TO DISCUSSION

Replying to Mr. Osborn, Mr. Vosloo agreed that it would be difficult to obtain reliable data on sludge

growth rates in extended-eration plants, but the problem could possibly be overcome by carrying out laboratory investigations with homogenized crude sewage, as had been done by the Water Pollution Research Laboratory.

There was little or nothing a works operator could do by way of operating procedures to overcome the difficulties caused by low temperatures. The designer had to allow for this by providing sufficient aeration tank capacity to enable the operator to carry the proper amount of sludge required for preventing the loss of *Nitrosomonas* in winter. During the summer months the operator could reduce the MLSS concentration in order to lower the oxygen consumption, but there was no harm in running on a high MLSS concentration, provided sufficient oxygen could be supplied. This would decrease the amount of surplus sludge to be disposed of.

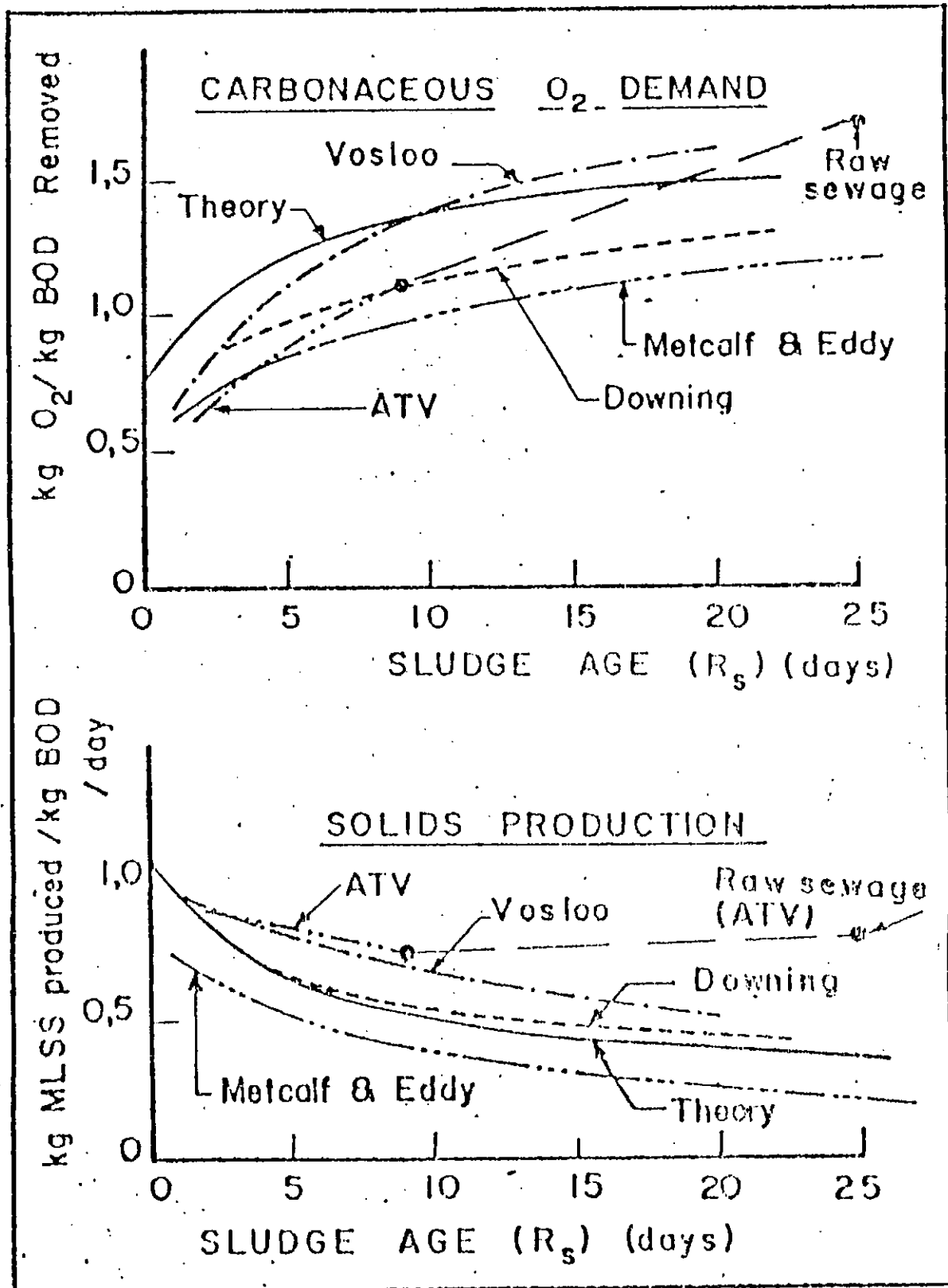
Mr. Vosloo did not consider that the lower oxygen requirement at low sludge age would lead to difficulties in ensuring adequate mixing. It had to be borne in mind that a low sludge age usually went hand in hand with a smaller than normal aeration tank, so that the intensity of aeration would not necessarily be lower than in the case of a large volume of air in a large aeration tank.

Regarding carbonaceous oxidation and nitrification, a two-stage process would be highly desirable to meet South Africa's stringent effluent standards. There should be a high-rate process for the first stage to remove the carbonaceous load, followed by a second stage for removing ammonia by nitrification. Each stage would have to carry its own sludge, which meant that each stage would also have to have its own sedimentation tanks.

In reply to Mr. Hall, Mr. Vosloo said that 10 000 mg/l of suspended solids in returned sludge was not uncommon, but not enough was known as yet about the conditions that were necessary to ensure that such a figure could be maintained at all times, and it could not be used as a basis for design. It would probably be extremely difficult to maintain an MLSS concentration of 5000 mg/l at times when the SVI was above 200 ml/g.

In reply to Mr. Meiring, he said that he saw no real difference between total oxidation and extended-aeration. The former was probably intended to apply to an extended-eration process employing a particularly low sludge loading rate, but actual total oxidation was out of the question. It appeared that, no matter how low the loading rate or how long the aeration period, ordinary sewage would always produce at least 0.3 kg of excess sludge per kg BOD applied. If this excess sludge was not drawn off at some stage or other, it would be shed in the form of finely-divided solid material carried over with the effluent. While it appeared to be common practice in the United States to allow this to take place, it would not be permissible in South Africa where very strict effluent standards applied.

*See also 40.5.176.1
Sludge for disposal*



Comparación de criterios de diseño para demanda de oxígeno y producción de barros por día (Fuente: "The activated sludge process, Part 1: Steady state behaviour", Marais, G.R. y Ekama, G.A., Water S.A., Vol. 2, N° 4, 1976).--

3.3.- REMOCION DE FOSFORO

De lo que surge del punto 3.1, las urgencias para la implementación del tratamiento de eliminación de fósforo no son apremiantes, aunque deben preverse. En el interín, otras medidas para reducir la carga sobre el lago podrán ser estudiadas.

Los parámetros adoptados conducen a estimar una concentración de P de:

$$C = \frac{2 \text{ gP/h./día}}{400 \text{ l/h./día}} = 0.005 \text{ gP/l} = 5 \text{ ppm P}$$

y como hemos planteado una eficiencia de remoción del 75%, el efluente no deberá contener más de 1 ppm de P.

Se puede llegar a ese umbral de dos maneras:

- a) por precipitación química, principalmente de ortofosfatos.
- b) mediante procesos biológicos basados en la "captación lujuriosa", un mecanismo celular poco conocido hasta 1986.-

El método químico de eliminación puede llevarse a cabo mediante adición de cal (pH alto), cloruro o sulfato férrico, sulfato de aluminio o aluminato sódico (pH bajo). El fósforo es eliminado de la fase líquida mediante una combinación de precipitación, adsorción e intercambio. Como tratamiento terciario, requiere importantes instalaciones y disponibilidad de terreno. Como tratamiento conjunto, la elección del reactivo queda reducida prácticamente al cloruro férrico, un insumo de bajo costo en zonas industrializadas (es un subproducto del decapado). En cambio, el sulfato de aluminio impone altos costos. Además, la asociación con la aireación extendida no nos parece clara, dado que la única forma de eliminar los fosfatos es conjuntamente con el barro biológico y esa operación está atada a la necesidad de mantener una concentración apropiada en el reactor.

La eliminación de fósforo por vía biológica está en pleno desarrollo y ha despertado gran expectativa. Por ahora los procesos mejor estudiados están protegidos por patente, tales como:

Bardenpho	Eimco Process Equipment Co.
Phostrip	Biospheric Inc.
A/O	Air Products & Chemical Inc.

Particularmente interesante nos parece en este caso el --- "PHOSTRIP" debido a que:

- puede ser incertado sin dificultad en el circuito hidráulico de la aireación extendida (particularmente de distribución funcional) aún cuando requiera obras de cierto porte.
- el fósforo es eliminado con el sobrenadante de elutriación, independientemente de los barros biológicos que pueden ser integralmente recirculados.
- puede precipitarse, a la postre, con cal, un insumo de bajo costo.
- una alternativa que se nos ocurre interesante es la elutriar con agua subterránea de bajo contenido de P disponiendo el sobrenadante en el suelo, donde los mecanismos de fijación son muy eficientes, mediante riego de superficies de extensión suficiente.

Como desventaja, se señala las todavía escasas referencias existentes sobre el tema. Además, un desarrollo consistente de este tema no podría ser resuelto dentro de los límites impuestos a este Anteproyecto.

CONCLUSION: A la luz de los resultados del pto. 3.1 la eliminación del fósforo comienza a ser importante a partir del año 2010 o cuando S. C. de Bariloche tenga 184.000 habitantes. Hasta entonces, la acción debe orientarse a:

- seguimiento de la investigación y de los progresos en el tema.
- seguimiento del estado trófico del lago mediante mediciones sistemáticas a cargo del D.P.A.

- investigaciones propias utilizando el Establecimiento Depurador como centro.
- promoción y fomento de medidas tendientes a -
contener o limitar el consumo de agentes tenso
activos ricos en polifosfatos (p.ej. regla -
mentación de lavaderos, desgravación de cier -
tos detergentes, educación y difusión, etc.).

4 - PLANTEO DE SUBALTERNATIVAS

Se han desarrollado seis (6) alternativas de implantación general mostradas en otros tantos planos que se adjuntan.-

La programación en etapas obliga a pronunciarse por una de las dos formas de organizar el Establecimiento: a) con distribuciones unitarias y b) con distribuciones funcionales.-

Las distribuciones unitarias son aquellas en que cada etapa se comporta como una planta autónoma. No hay grandes conductos comunes, se recircula el propio barro y las unidades se asocian por etapas, no por función. A favor de ella se pueden anotar las siguientes ventajas: Hay mínimos problemas de interferencia en el momento de ampliar la planta, los conductos de recirculación son cortos y no están sometidos a regímenes diversos, con la segunda etapa construida se crean condiciones ideales para la optimización y la investigación pudiéndose operar una etapa como testigo y la otra con las variantes que se investigan. Como desventajas, presentan las siguientes: ninguna unidad puede emplearse en socorro de las otras etapas y por consiguiente las reservas (máquinas en stand by como bombas de recirculación) se multiplican, hay operaciones que resultan descentralizadas, la operación es menos flexible y el refuerzo selectivo de alguna de las etapas del proceso es muy difícil de lograr. La distribución unitaria se utiliza generalmente en plantas pequeñas o medias. La subalternativa D responde a esta categoría.-

En la distribución funcional las unidades semejantes se asocian en distintas áreas. Se consigue con ello las siguientes ventajas: se gana en confiabilidad y flexibilidad y son más fácilmente corregibles algún desajuste inicial de los tamaños (son "todas para todas") y las operaciones resultarán, en el futuro, más centralizadas.-

En el caso de Bariloche, la flexibilidad resulta particularmente atractiva, por las dudas que hay sobre la concentración (dilución) de los futuros líquidos.-

Suele achacarse a este dispositivo la desventaja de grandes -- distancias de transporte de los barros y compromisos difíciles de superar en los esquemas y perfiles hidráulicos, pero se estima posible evitar esos inconvenientes mediante la utilización de canales lo cual requiere mayor elaboración arquitectónica del proyecto. Las alternativas B a F transitan por esta -- tendencia, la que se impone en plantas grandes.-

En las alternativas B a D se ha hecho uso de la totalidad del predio, lo cual evidentemente favorece al conjunto, pero ello implica expropiar tres lotes que tienen obras construídas, aunque de escaso valor. El resto de las alternativas prescinde de esos 3 lotes.-

En todos los casos se ha considerado que, a tamaño completo, -- los barros serán deshidratados mecánicamente. No obstante, se confía que durante la primer etapa (y quizá también en la segunda) la disposición en eras de secado resultará práctica.-

Ello es posible dentro de los límites del predio a condición de hacer uso transitorio de las áreas destinadas a futuras unidades de proceso, pero las instalaciones de las eras tendrían -- que ser removidas.-

Finalmente, la alternativa F se programó sobre la base de un -- cambio en el punto de funcionamiento de las etapas construídas (1ra. y 2da.) para afrontar los requerimientos del tamaño completo. Ese cambio de funcionamiento consiste en:

- Reducir la edad del lodo a 15 días, con lo que la carga másica se eleva a $0,10 \text{ Kg DBO}_5/\text{Kg SST día}$ y con $X = 3,5 \text{ Kg/m}^3$ la carga volúmica a $0,36 \text{ Kg DBO}_5/\text{m}^3 \text{ día}$.-

Es decir se ingresa a la zona de activación, en reemplazo de la de oxidación prolongada.-

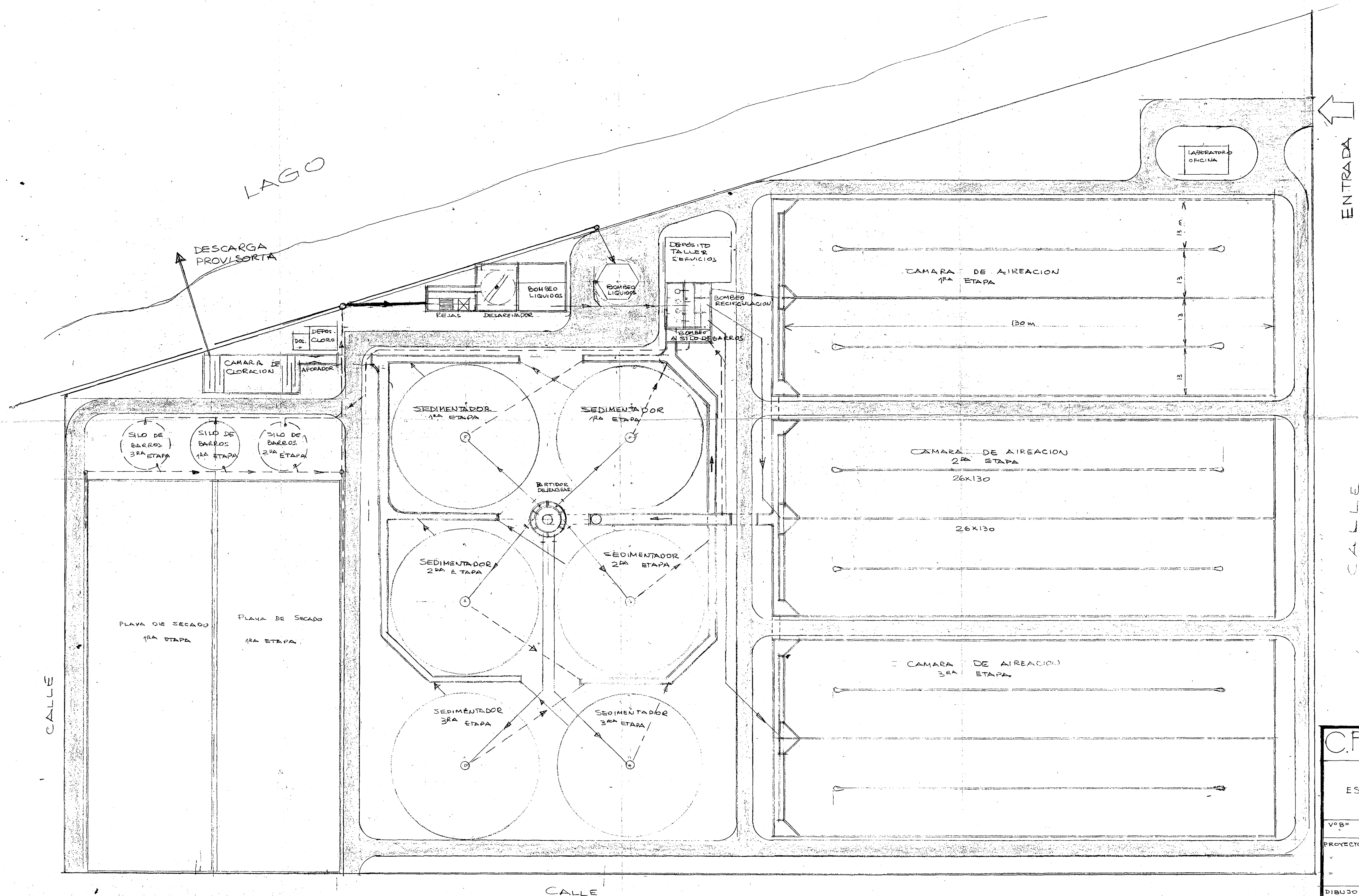
- La relación OC/L caerá a $1,6 \text{ Kg O}_2/\text{Kg DBO}_5$, por lo que no se llegará a compensar los efectos de la mayor carga orgánica. Será necesario, posiblemente, implementar una máquina más -- por reactor, cosa perfectamente factible.-

- Con este punto de funcionamiento, se obtendrá un barro más --

floculento, y por consiguiente, la velocidad ascensional podrá ser aumentada a 1,2 m/h. Como consecuencia del incremento de carga hidráulica podrá ser absorbido agregando sólo un sedimentador igual a los anteriores.-

- Según ATV, el índice de crecimiento I se mantendrá al mismo valor, pero como es mayor la carga aplicada, habrá mayor producción de barro, y serán más vitales, es decir menos estabilizados. Será necesario por consiguiente perfeccionar el tratamiento de los mismos.-
- Ese perfeccionamiento podrá ser hecho de dos maneras: o bien por vía biológica implementando calefacción y recolección de metano en los digestores o bien confiando a los progresos en el tratamiento químico-mecánico de los barro (por ejemplo, coagulación con polielectrolito y filtro banda u otro).-
- Las eras de secado serán autosuficientes para las dos primeras etapas y de inclinarse la solución por vía de la digestión anaeróbica, en la tercer etapa será posiblemente necesario socorrer a las playas con máquinas centrifugadoras.-

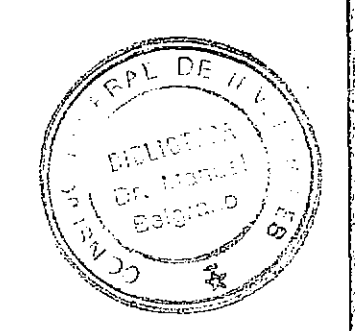
Demostrada de esta manera la factibilidad de esta interesante alternativa, se tomarán las previsiones en cuanto a las obras comunes de pretratamiento y bombeo. En este, como en los demás casos, sería ilógico proyectar por anticipado, sobre la base de hipótesis de trabajo inciertas y con las tecnologías actuales.-



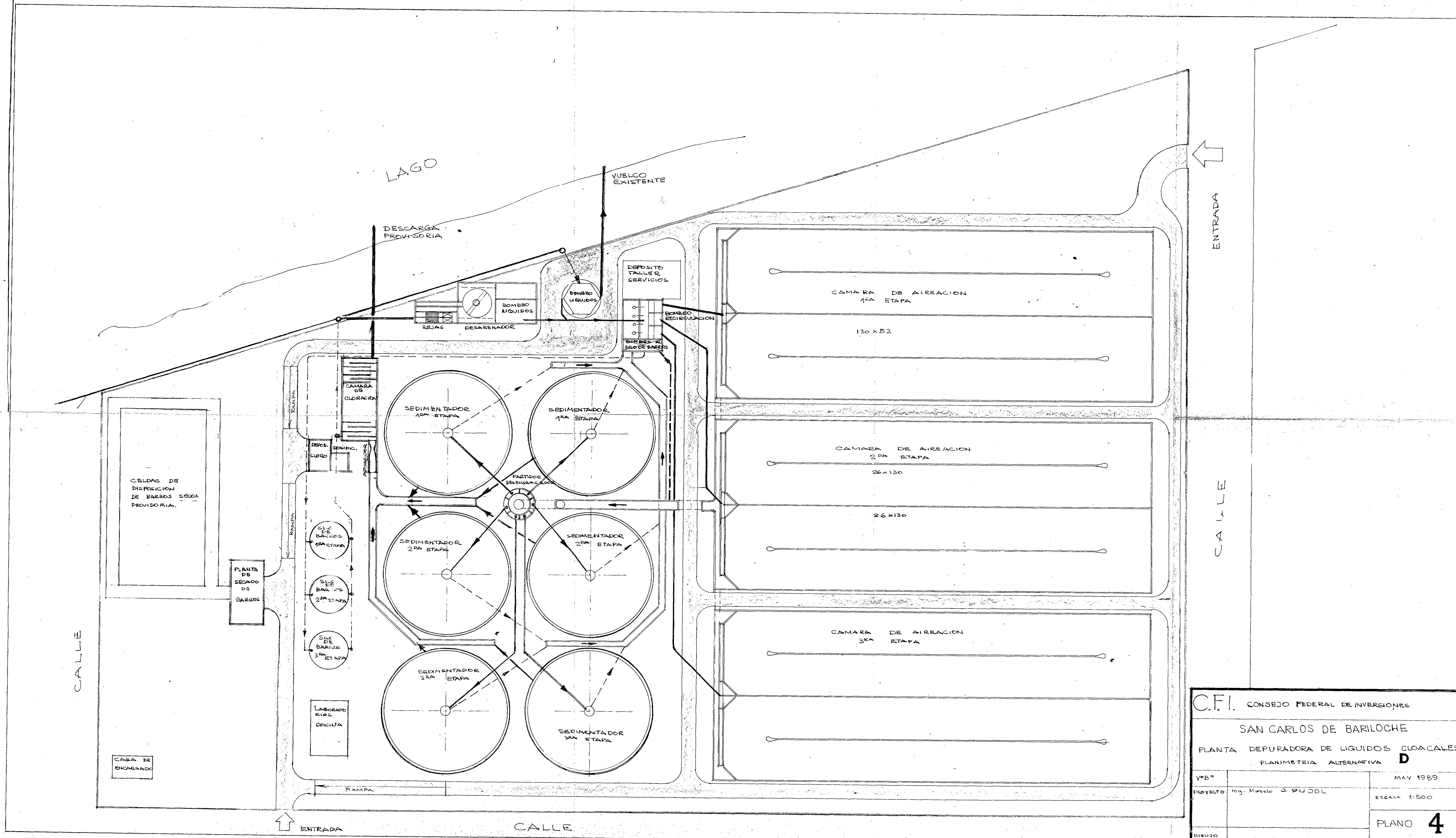
INTA

CALLE

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SAN CARLOS DE BARILOCHE ESTABLECIMIENTO DEPURADOR CLOACAL PLANIMETRIA ALTERNATIVA B		
VºBº		FECHA MAYO 1985
PROYECTO	INGº MARCELO J. PUJOL	PLANO 2
DIBUJO		
		ESCALA 1:500



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PLANIMETRIA ALTERNATIVA D		
Yo Bº		MAY 1989
PROYECTO	Ing. Marcelo J. PUJOL	ESCALA 1:500
DIBUJO		PLANO 4

DESCARGA
EFLUENTE
DEPURADO

CORTE 1-1 ESCALA 1:100

LAGO

DESCARGA
EXISTENTE

CELAS DE DISPOSICION
TRANSITORIA DE BARROS
DESHIDRATADOS

PLANTA DE
DESHIDRATADO DE
BARROS

DEPOSITO
DE CLORO
DOSA
DE CL

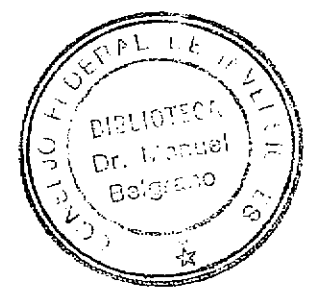
CAMARA
DE
CLORACION

CALLE

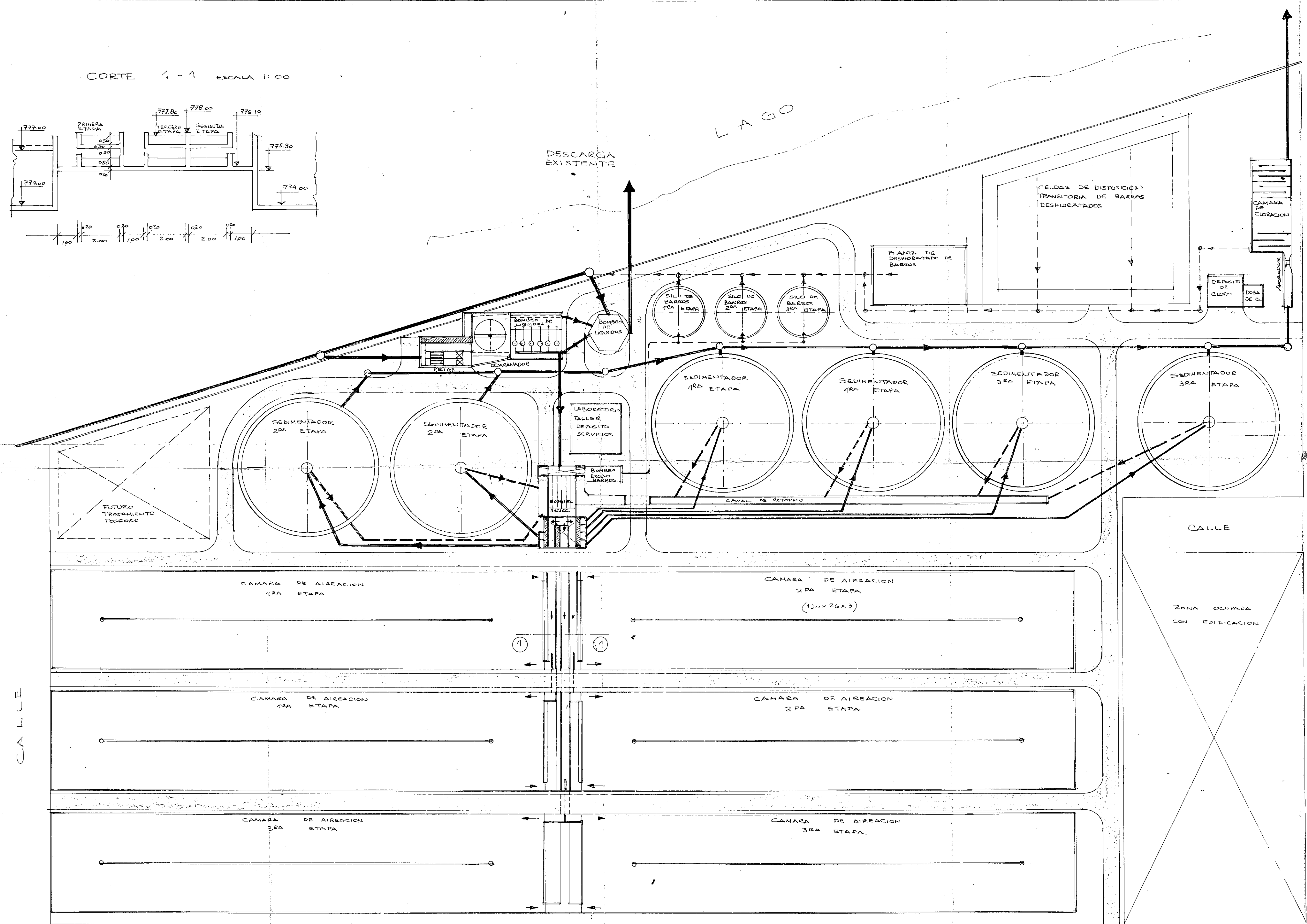
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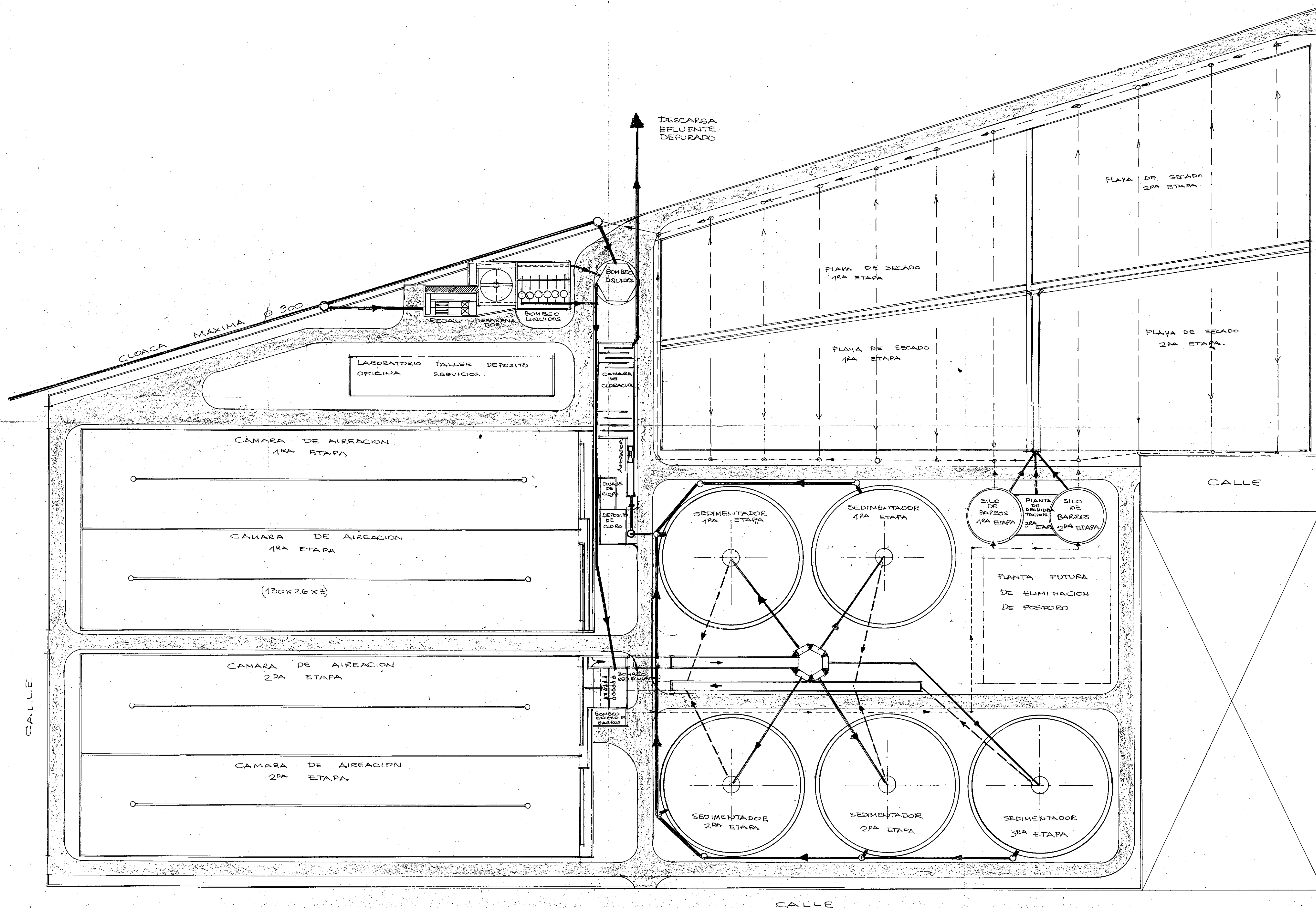
ZONA OCUPADA
CON EDIFICACION

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VºBº		FECHA MAYO 1989
PROYECTO	INGº MARCELO J. PUJOL	PLANO 5
DIBUJO		ESCALA 1:500





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PUJOL, MARCELO O/F. 331.9/P32.1

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SAN CARLOS DE BARILOCHE
ESTABLECIMIENTO DEPURADOR CLOACAL
PLANIMETRIA ALTERNATIVA F

Vols	PROYECTO ING° MARCELO J. PUJOL	FECHA: MAYO 1983
DIBUJO		PLANO 6
		ESCALA 1:500