Crandon Project: Addendum no. 1 to the preliminary engineering report for wastewater treatment facilities: mine water management contingency plan: scope ID 00C002. 2000

Foth and Van Dyke and Associates, Inc.
Green Bay, Wisconsin: Foth and Van Dyke, 2000

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March 13, 2000

Mr. Paul Luebke
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Ms. Char Hauger
U.S. Army Corps of Engineers
St. Paul District
190 Fifth Street East
St. Paul, MN 55101

Dear Mr. Luebke and Ms. Hauger:

Re: Crandon Project - *Addendum No. 1 to the Preliminary Engineering Report for Wastewater Treatment Facilities* ***Mine Water Management Contingency Plan***

Nicolet Minerals Company (NMC) is pleased to submit the enclosed report titled *Addendum No. 1 to the Preliminary Engineering Report for Wastewater Treatment Facilities* ***Mine Water Management Contingency Plan*** *(MWCP)*.

The MWCP has been prepared on behalf of NMC by Foth & Van Dyke and Associates, Inc. As noted on the attached distribution list, NMC has distributed the information to appropriate state and federal agencies, to local officials, and to various interested parties. It is our understanding that the Wisconsin Department of Natural Resources (WDNR) and the U.S. Army Corps of Engineers (USCOE) will be responsible for distribution of the document to their appropriate staff members.

The primary purpose of the MWCP is to describe the contingency plan that NMC will implement for water management in the unlikely event that mine inflow exceeds the current design capacity of the wastewater treatment and discharge systems. The MWCP has been prepared in response to the WDNR's review of the regional groundwater flow model by the U.S. Geological Survey (USGS). NMC has developed this contingency plan for mine inflow to demonstrate how its water management facilities could be modified to accommodate higher mine inflows.

If you or your staff have any questions regarding this document, please contact me at (715) 478-3393.

Sincerely,

Gordon Reid, P.E.
Manager of Engineering
Nicolet Minerals Company

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Crandon Project
Addendum No. 1 to the Preliminary Engineering Report for Wastewater Treatment Facilities

Mine Water Management Contingency Plan

Scope ID: 00C002

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Crandon Project
Addendum No. 1 to Preliminary Engineering Report for Wastewater Treatment Facilities

Mine Water Management Contingency Plan

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Appendices

1 Introduction and Objective

As part of the Crandon Project's *Environmental Impact Report* (EIR) (Foth & Van Dyke, 1995/1998a), Nicolet Minerals Company (NMC) completed a detailed assessment of potential groundwater inflow to its proposed underground zinc and copper mine and subsequent potential impacts to regional water resources. Based on groundwater model calibrations, NMC has estimated that the potential mine inflows will be approximately 450 gpm under best engineering judgement (BEJ) conditions to approximately 775 gpm under practical worst case (PWC) conditions. The BEJ mine inflow prediction represents the condition most likely to occur during mine operations. Based on these predictions, NMC selected an average mine inflow rate of 600 gpm as the design basis for the construction and operation of the wastewater treatment facilities, as described in the *Preliminary Engineering Report for Wastewater Treatment Facilities for the Crandon Project* (PER) (Foth & Van Dyke, 1995/1998b). On January 18, 1999, NMC signed a legally binding contract with the Town of Lincoln that the mine water discharge will not exceed 600 gpm average, as measured over any 30-day period. If pumpage exceeds this limit, NMC will promptly undertake appropriate remedial action and suspend mine development until mine water discharge falls below 600 gpm.

The PWC mine inflow prediction represents a very conservative assessment of potential mine inflow based on numerous conservative assumptions, as described in the EIR (Foth & Van Dyke, 1995/1998a). Given the conservative nature of the PWC prediction, it represents an inflow condition that, although possible, is highly unlikely. The PWC scenario was used as the basis for the *Crandon Project Surface Water Mitigation Plan* (SWMP) (Foth & Van Dyke, 1998a). The SWMP contains contingencies for expanding mitigation in the unlikely event that measured impacts to surface waters are greater than predicted.

The purpose of this document is to describe the contingency plan that NMC will implement for mine inflow and water management in the unlikely event that mine inflow exceeds the current design capacity of the wastewater treatment and disposal systems. Based on the Wisconsin Department of Natural Resources' (WDNR) review of the regional groundwater flow model (U.S. Geological Survey, 1999), NMC believes that the WDNR's "worst case predictions" of mine inflow will exceed 1,200 gpm. Although NMC does not concur with the WDNR's analyses (U.S. Geological Survey, 1999), it has elected to develop this contingency plan for mine inflow to demonstrate how its water management facilities could be modified to accommodate higher mine inflows. In doing so, NMC has elected to use a value of 1,500 gpm as the basis for the development of this plan. It should be noted that 1,500 gpm is not the maximum inflow that could be handled by the expanded wastewater treatment facilities (WWTF) and soil absorption system (SAS). Additional treatment and disposal capacity could be implemented by following the framework described in this plan.

The objective of this Mine Water Management Contingency Plan (WMCP) is to incorporate contingency measures that will allow NMC to implement modifications to the project to treat and discharge excess water in the unlikely event that the measured mine inflow exceeds the design capacity of the WWTF and SAS systems. Accordingly, this report addresses the following topics:

- A description of the underground mine development process as it relates to mine dewatering, mine inflow control, and the various stages of mine dewatering that correlate to implementation of the WMCP (Section 2 of this report).
- An evaluation of alternatives for the WMCP and provisions of the selected approach for the WMCP (Section 3 of this report).
- A description of engineering modifications to the WWTF and discharge facilities that would be required during implementation of the WMCP (Section 4 of this report).
- The data analysis and decision process that will be followed to determine the need for implementing the WMCP during various stages of mining and the schedule for implementation of the WMCP once a decision has been made to implement (Section 5 of this report).

2 Mine Development and Initial Groundwater Inflows

2.1 Underground Mine Development and Dewatering

Mine development will be conducted using a variety of water control methodologies. These methodologies will consist of several components, including:

- preservation of natural geologic flow barriers
- grouting and testing
- controlled drainage
- mine sequencing
- mine water discharge system

These components are discussed in more detail in Sections 2.2 through 2.7.

2.2 Preservation of Natural Geologic Flow Barriers

The massive saprolite/till layer at the bedrock interface is a significant, natural flow barrier that will act as a natural control on the seepage of groundwater into the mine. A key feature of the design of the underground mine is preserving the integrity of the massive saprolite/till layer. Accordingly, NMC will maintain a 200 foot crown pillar during the initial years of mining. In the uppermost part of the mine where the ore and host rock may have been somewhat weakened by weathering, controlled cut-and-fill methods will be employed during the latter stages of mining to reduce the crown pillar thickness to 100 feet. This extraction method involves the controlled removal of horizontal lifts of ore approximately 13-16 feet thick. The voids created will be used to test for possible inflows. Once assured that the next lift can be excavated under controlled conditions, the voids will be tightly backfilled with cemented paste tailings prior to mining of the next lift. Studies completed by Agapito Associates (1996, 1997) have shown that the planned mining will not cause strains in the crown pillar that will affect its hydrologic integrity. Moreover, other studies have shown that the potential for subsidence over the mine is negligible (John D. Smith Engineering Associates Limited, 1982; Agapito Associates, 1996). As such, the integrity of both the crown pillar and massive saprolite/till layer will be preserved.

In addition to preserving the hydrologic integrity of natural flow barriers, NMC will implement water inflow control measures during the sinking of the main shaft and ventilation shafts. Ground freezing will be used around the shafts to prevent water inflow as the shaft is developed through the glacial overburden. The shaft will be sunk into low permeability hanging wall rock. Cover grouting will be completed, followed by the installation of a concrete liner to stabilize the walls.

Section 4.8 and 4.9 of the *Mine Permit Application* (Foth & Van Dyke, 1995/1998c) (MPA) provide additional details on proposed mining methods to be employed.

2.3 Grouting and Testing

Grouting may be defined as the process of pumping materials into cracks or voids in geologic materials, generally for the purpose of strengthening them and/or reducing their permeability. The grout is generally clay or cement-based. Other approved additives may be used to improve the set-up time, penetration, and effectiveness of the grouts. A significant degree of hydraulic isolation is typically achieved through a well designed grouting program. A detailed discussion of grouting and monitoring for the Crandon project is included in Section 4.8.6 and Appendix B of the MPA (Foth & Van Dyke, 1995/1998c). Note that the effectiveness of the grouting will be evaluated prior to the commencement of mining, per the plan contained in Appendix B of the MPA (Foth & Van Dyke, 1995/1998c). Typical grouting techniques to be employed at the Crandon Project are outlined below.

2.3.1 Cover Grouting

Cover grouting will be applied in conjunction with shaft sinking and development drives or drifts in the hanging wall to reduce excessive water inflow. Cover grouting will also be completed during development of the grout drift at the 260 foot level within the Crandon Formation. Cover grouting involves drilling holes from the face or from a drilling gallery near the face. These holes are grouted to form a cover over the future excavation. Advance of the grout drift through the hanging wall contact and along the strike of the ore body is expected to require extensive cover grouting.

2.3.2 Blanket Grouting

Blanket grouting will be used to isolate mining in the more permeable Crandon Formation from the water bearing overburden and weathered rock. A grout blanket will be placed at the base of the crown pillar. Grouting from the grout drift will provide a more effective grout blanket than grouting from surface, since the varying sub-horizontal orientation of the underground grout holes will intersect a greater number of fractures compared to vertical holes drilled from surface. In addition, the grout drift can serve as a monitoring gallery to assess the performance of the grout blanket as mine development and stoping operations progress. If monitoring indicates a potential for higher inflows, additional grouting will be implemented.

A grouting and access drift will be developed from the East Ventilation Shaft on the 260 foot level, at the base of the crown pillar. Development of the 10 foot by 10 foot grouting drift will be conducted under a grout cover, where necessary. Probe holes will be drilled ahead of the face to detect the presence of water and to indicate the extent of cover grouting. Blanket grouting in the base of the crown pillar will begin concurrent with, and following completion of, the grout drift development.

To determine appropriate grout mixtures for each grouted section, packer testing will be employed at prescribed intervals for each grouting stage to determine the water take for each stage of grout advancement. Grout mixtures will be adjusted to match conditions encountered in each stage. Water inflow encountered while drilling will be controlled with a collar pipe grouted in the borehole and connected to a diverter and stuffing box designed for that purpose.

Grout cutoff fans will be constructed, as appropriate, along the length of, and at the ends of, the grout area to encapsulate the active mining areas in a grout blanket, and thereby isolate the mining operations by keying into lower permeability rock in the footwall and hanging wall formations. The grout blanket is ultimately expected to extend over the full length and width of the ore body.

Monitoring boreholes will be drilled to detect seepage. These holes will extend from the grouting drift to a point approximately 20 feet beyond the edge of the grout blanket. Monitoring boreholes will be installed in the strongly weathered sections of the grout drift where seepage is most likely. The grouting drift and monitoring boreholes will be maintained throughout the life of mining operations to monitor the grout blanket's effectiveness, and to provide access if additional grouting is needed.

2.4 Controlled Drainage

Controlled drainage will be extensively used in the Crandon Project, primarily in the pre-production period and in the initial years of mining. Stored water in the stoping blocks will be drained or dewatered prior to mining. Prior to mining a stoping block, a delineation drilling program will be established to (a) define grade and geometry of the stopes, and (b) permit draining of the planned mining area. The rate of controlled drainage will be governed by the design capacity in the WWTF and SAS and the overall mine inflow.

As described above, probe holes will be drilled in advance of the hanging wall and grout drift headings. In addition, fans of drill holes will be completed from the grouting drift for purposes of completing the grout blanket. NMC will frequently packer test these holes and the delineation drill holes to further assess the hydraulic characteristics of the bedrock as the development process progresses. This additional data will be used to supplement the existing database on bedrock hydraulic conductivity. The expanded database will subsequently be factored into the analysis on the need to implement the contingency plan.

2.5 Mine Sequencing

The sequencing of the mining activities affords another degree of operational flexibility to control and evaluate mine inflow and, if need be, implement the WMCP. During the initial stages of mining, mine inflow is likely to be low, as operational activities will be occurring primarily in the hanging wall drifts and grout drift. This early pre-production period defines the first stage of mine development as it relates to mine inflow. The second stage of mine inflow relates to the draining and dewatering of the stoping blocks. During this second stage of mining, mine inflows would be expected to increase. The third stage of mining relates to the actual removal of ore from the stoping blocks, during which time sustained inflows are likely to reach their maximum value.

NMC has built into the mine sequence sufficient flexibility to conduct the underground development activities to minimize mine inflow to manageable levels. For instance, if high mine inflows are expected to occur in a particular development area based on data obtained from the cover grouting program or draining program, mining activities can be shifted to other areas of the mine until such time that the sustained inflows are controlled so as not to exceed the capacity of the WWTF or SAS.

2.6 Mine Water Drainage System

A mine water collection and pumping system is required to accumulate and discharge water from the mine. The components of the mine water discharge system for the Crandon Project include mine water ditches, boreholes, and pipelines to carry water from the mine working areas to the mine collection and settling sumps. Clean water sumps, pump stations, pump control systems, and water discharge piping complete the system. Control of the pumping will be automated through level control switches. As discussed in Section 4.8 of the MPA (Foth & Van Dyke, 1995/1998c), drainage water from the sumps will be reused as utility water whenever practicable for drilling, dust control, and cooling.

2.7 Initial Mine Inflows

As described above in Sections 2.1 through 2.6, the development of the underground mine will progress through several stages as it relates to mine dewatering. Underground mine development will involve three main phases from a hydrogeologic standpoint. The first phase includes sinking of the main shaft and east ventilation shaft (ground freezing will be used to cut off flows to the shafts in the glacial overburden), development of the initial hanging wall drifts, and the development of the grouting drift at the 260 foot level in the Crandon Formation. With the exception of the grout drift, this first phase of development will take place in the competent low hydraulic conductivity hanging wall rock north of the ore body. The grouting drift is a small opening within the Crandon Formation that will be completed under cover grouting. With cover grouting, groundwater inflow to the grout drift will be minimal. The key point is that the drifts developed in the three year pre-production period will be engineered access ways through the rock in which grouting is used to control groundwater percolation into the mine workings. This type of development is a common engineering practice, and will result in a condition whereby limited inflows would be expected to occur. As a result, during hanging-wall development, sustained inflows to the workings will likely be well below the capacity of the water management system. Sustained inflows are defined as a 30-day rolling average of water pumped from the mine as measured at the production shaft, plus or minus other sources such as utility water, etc. Higher inflow conditions, such as those likely to be predicted by the WDNR based on their analysis of bedrock and till/saprolite hydraulic characteristics (USGS, 1999), pertain to a later mining time period. Because the hanging wall rock is much less permeable than the Crandon Formation, and because the shaft and drift openings will be much smaller openings than mining blocks, the initial inflows during this period will almost certainly be below those predicted for the fully-open zinc mine.

Two important facts related to groundwater inflow should be noted about this initial development period. First, the water management facilities are focused on "sustained" inflows, which ignores the initial inflow that may occur due to localized storage release as a section of a shaft or drift is excavated. It is expected that just after excavating one of these sections, inflow to the new opening may be high, but only for a very short time (minutes to hours). Secondly, NMC will have an upper limit on the rate of water removal from the workings that is established by the capacity of the water management system. Where an excavated development heading initially produces excessive inflows due to the drainage of stored water, the rate of development would be slowed and/or dewatering of the undeveloped drift heading would be reduced by grouting techniques to avoid exceeding the system capacity. Section 5.1 of this report addresses the decision process and criteria by which the contingency plan would be implemented during this period of mine development if, in the unlikely event, higher sustained mine inflows would occur due to higher hydraulic conductivities in the bedrock and till/saprolite.

As the first phase of mine development is progressing, the second phase of underground development, from a hydrogeologic perspective, will begin. The second phase of development is the ore dewatering or draining period. During this period of time, drill holes will be drilled from the hanging wall drifts into the ore body to serve two purposes. The first is to provide detailed delineation of the ore grade, while the second is to dewater the ore body prior to mining. Not all holes will be used for draining water from the ore body. Only those holes that are water bearing will be collared with control valves and used to drain the ore body at a controlled rate. As the drilling program and dewatering program progresses, inflow to the underground mine will gradually increase. The ore draining/dewatering period of mine development represents a second point in the development of the mine for which decision criteria will be needed to determine if the contingency plan described in this report will need to be implemented. These decision criteria are described in Section 5.1.

The third and final stage of mine development, from a hydrogeologic perspective, begins with the actual removal of ore from the mining stopes. It is during this phase of mining that the highest sustained inflows into the mine will likely occur. The higher inflow predictions that are likely to result from the WDNR's predictions, based on their analysis of the bedrock hydraulic conductivity and saprolite/till permeability (USGS, 1999), correlate to this condition of mining. It is important to note that NMC's best engineering judgement (BEJ) and practical worst case (PWC) predictions correlate to this condition of the mine life and conservatively assume that all of the mineable zinc ore has been dewatered. During this stage of mining, yet a third set of decision criteria will be used in a decision process to determine if the contingency plan needs to be implemented.

Note that an inherent assumption in the implementation process during these three stages of mine development is to implement the plan in advance of mine inflow actually exceeding the capacity of the water management system. It is in NMC's interest to implement the contingency plan prior to inflow exceeding the capacity of the system, since failure to do so could result in a temporary pause in mine development. The decision criteria described in Section 5.1 have been developed with this goal in mind.

3 Contingency Plan Alternatives and Selected Approach

If primary, secondary, and tertiary grouting, as described in the MPA (Foth & Van Dyke, 1995/1998c) fail to control groundwater inflow, two basic alternatives can be considered with respect to development of the WMCP. The first alternative is development of a WMCP that focuses on treating and disposing of mine inflows that exceed the design capacity described in the PER (Foth & Van Dyke, 1995/1998b). The second alternative is to implement other engineering controls to limit mine inflow to the design capacity of the WWTF and SAS. Each of these basic approaches is described below.

3.1 Water Management Alternatives

The following sections describe the wastewater treatment and disposal alternatives considered for addressing contingency flows, up to an assumed mine inflow rate of 1,500 gpm, which is assumed to be equal to or greater than the upper bound practical worst case groundwater inflow simulation, as developed by the WDNR.

3.1.1 Wastewater Treatment Facilities

The Crandon Project wastewater treatment facilities will be designed and constructed based on the maximum wastewater flow condition as presented in the PER (Foth & Van Dyke, 1995/1998b). In the unlikely event that evaluation of data collected during the early stages of mine development indicates the probability of a maximum mine inflow in excess of that presented in the PER, expansion of the wastewater treatment facilities will be required. Since the treatment facilities will be constructed and placed into operation prior to development of data which would support higher mine inflow estimates, any treatment facility expansion will require construction of additional treatment trains.

For purposes of preparing this contingency plan, the maximum projected inflow to the mine is estimated at 1,500 gpm. This level of mine inflow could result in a projected flow to the WWTF of 1,757 gpm during wet weather conditions, as shown in the overall mill water balance for contingency plan flow conditions (Figure 3-1). This compares to the PER-based maximum mine inflow and WWTF inflow values of 600 gpm and 726 gpm, respectively. Consequently, under the maximum contingency plan flow condition, an additional 1,031 gpm of treatment capacity would be required.

The WWTF design basis, as presented in the PER (Foth & Van Dyke, 1995/1998b), includes two treatment trains, each having a capacity of 370 gpm. Additional treatment capacity could be provided by adding one or more treatment trains to the WWTF. Alternative treatment system configurations for one, two, or three additional treatment trains, sized to accommodate flows associated with 1,500 gpm of mine inflow, were evaluated. Considerations applied in the evaluation of the number and capacity of additional treatment trains to be constructed include the following:

- area requirements
- operational complexity
- maintenance requirements
- operational flexibility
- design/construction time requirements
- capital cost
- operation and maintenance cost

Based on consideration of these factors, an additional three train configuration was selected. Each train would be sized to match the PER treatment train capacity of 370 gpm. While the three train configuration will have relatively higher capital cost, operating cost, area requirement, and operational complexity, it offers significant advantages with respect to operational flexibility and design/construction time requirements. Design/construction times would be minimized through reuse of engineering designs and shop drawings developed for construction of the initial WWTF.

3.1.2 Treated Wastewater Discharge Facilities

At a mine inflow rate of 1,500 gpm, taking into account additions and losses to the system, the flow to the SAS is 1,532 gpm. Treated wastewater discharge of the flow would be through the SAS, as described in the *Preliminary Engineering Report for the Crandon Project Soil Absorption System* (SAS PER) (Foth & Van Dyke, 1998b). Another potential alternative if adequate disposal volume were not available in the SAS would be the use of continuous mitigation on soft water lakes. A description of these treated wastewater discharge alternatives follows.

3.1.2.1 Area H Soil Absorption System

The location of Area H and alternate SAS sites are shown in Figure 1-2 of the SAS PER (Foth & Van Dyke, 1998b). Area H is the selected SAS site for discharge of treated wastewater under the current project design. The numerical analysis described in the SAS PER (Foth & Van Dyke, 1998b, Appendix J) has shown that the aquifer thickness that will transmit water from the SAS will be about 70 feet. By conservatively assuming an aquifer thickness of 45 feet in the design calculations, and by redesigning the piping system in the cells and limiting the mounded groundwater beneath each cell to within 2 feet of the base of the cell, a hydraulic capacity exceeding 1,500 gpm can be achieved at the Area H SAS. A description of the design enhancement and resulting flow handling capabilities of Area H is presented in Section 4.2 of this report.

3.1.2.2 Other Options

Several other options exist for the discharge of treated wastewater. These include the potential SAS at Area A and continuous mitigation. Area A was a potential site evaluated in the SAS PER (Foth & Van Dyke, 1998b), and could potentially handle a modest amount of water. However, given that mitigation systems will exist at the time of operations, continuous mitigation would likely be a preferable option in the highly unlikely event that additional discharge capacity is required.

The discharge of mitigation water is intended to replace the water from streams and lakes that have been impacted by the mine development. As mining operations influence the local groundwater level, the rate at which groundwater is naturally discharged to a stream may be reduced. In some cases, where a lake is naturally discharging to the groundwater, the rate of discharge may increase as the groundwater elevation drops. At the present time, the discharge of mitigation water is planned to occur only during low stream flow or low lake level conditions. If the water flow drops below the established public rights flow (PRF) for streams and/or if the lake levels drop below the public rights stage (PRS), mitigation water will be added to compensate for the loss of water attributed to the groundwater drawdown.

A contingency plan alternative for handling a greater volume of treated wastewater is for the continuous discharge of mitigation water year round to soft water bodies. This may include Little Sand Lake, Deep Hole Lake, and Skunk Lake. This addition of water during higher lake level conditions would not present a detriment to the surface waters, as it would only offset the loss due to mine dewatering. Thus, lake levels will not increase above what would normally occur without the mine operating. In this case, the soft water discharged from the wastewater evaporation process would be discharged to the soft water lakes on a year round basis. This will reduce the flow that would need to go to the SAS. The discharge of mitigation water to lakes and streams is part of the permit applications. All the discharge points will be covered under a WPDES discharge permit.

3.2 Mine Inflow Control Alternatives

Section 3.1 describes engineering considerations associated with a contingency plan focused on treatment and discharge of water resulting from higher mine inflow conditions. An alternative approach for the contingency plan could be the implementation of engineering controls to minimize groundwater inflow to the mine. Currently, NMC has incorporated grouting into the mining plan as the main engineering control for minimizing groundwater inflow. Slurry walls and artificial ground freezing are engineering controls used successfully to control groundwater seepage at other sites. These two technologies are discussed below as they relate to potential applicability to the Crandon Project.

3.2.1 Slurry Wall

A slurry wall is a low permeability, typically horizontal, barrier to groundwater flow. Slurry walls are typically constructed by excavating a trench and backfilling with a low permeability bentonite grout based slurry. These engineered structures have been commonly employed to minimize groundwater inflow to areas that are being dewatered or to isolate waste containment areas from regional groundwater movement. Since a trench is required for the construction of the slurry wall, it is limited to controlling groundwater movement in unconsolidated soils.

Moreover, the depth of the slurry wall is limited by the depth to which it can be constructed. Both of these limitations preclude the application of a slurry wall as a groundwater inflow control for the Crandon Project, since it could not be constructed to the necessary depth and could not be constructed in the bedrock.

3.2.2 Artificial Ground Freezing

Artificial ground freezing (AGF) is similar in concept to other flow barrier technologies. The objective is to create a frozen barrier within the saturated geologic units to restrict the movement of groundwater. As it would be applied to the Crandon Project, a frozen wall would be constructed around the perimeter of the mine to restrict the horizontal flow of groundwater to the mine. The barrier would be constructed by drilling closely spaced boreholes into the weathered bedrock for the construction of sealed freeze tubes. The freeze tubes would be connected near the surface to a piping distribution system that would circulate chilled brine through the tubes. The brine would be chilled at the surface by a refrigeration plant to temperatures as low as approximately -20°C to -25°C. The brine would be pumped from the refrigeration plant and circulated in a closed circuit through the piping distribution and freeze tubes system to freeze the ground. Over the course of several months, the ground around the freeze tubes would freeze, coalescing into a frozen wall around the perimeter of the mine down to the depth of the freeze tubes.

Although freeze wall technology has been used frequently on civil engineering projects, it has typically been used as a more localized engineering control. This technology will be used for the Crandon Project during shaft construction to cut off groundwater seepage to the shafts. However, artificial ground freezing has not been typically employed at the scale required to control groundwater inflow into the entire mine. Furthermore, implementation of this technology would result in additional surface disturbance and require additional energy resources.

3.3 Selected Approach

Based on a review of the above alternatives, NMC proposes to limit groundwater inflow through grouting, as currently proposed, and to develop a WMCP for higher mine inflows up to 1,500 gpm by expanding the existing WWTF and the SAS as needed. Additional treatment trains similar to ones currently designed for the 600 gpm mine inflow, as discussed in Section 4.1.2, will be added proportionately in the event that inflow values indicate that the originally installed system will not be able to handle future expected flows (see Section 5 for contingency plan trigger mechanisms and implementation schedule). Disposal of the contingency flow wastewater volume will be through the SAS at Area H, utilizing design enhancements.

Section 4 provides greater detail on the engineering modifications necessary to handle the wastewater treatment and disposal needs associated with the contingency mine inflow.

4 Engineering Modifications

4.1 Wastewater Treatment Facilities Upgrade

The wastewater treatment facilities were evaluated to determine modifications which would be required to increase the treatment capacity to meet contingency plan flow conditions. The following facilities were evaluated:

- Wastewater Storage Basin Nos. 6 and 7
- Wastewater Treatment Facilities (WWTF)
- Discharge Lagoons
- Effluent Discharge Pumping and Force Main

A description of the proposed modifications to each of these facilities is provided below. To aid in understanding the proposed modifications, a revised water balance for the wastewater treatment facilities has been prepared to illustrate water flow conditions assuming a mine inflow value of 1,500 gpm, and is presented in Figure 3-1.

4.1.1 Wastewater Storage Basin Nos. 6 and 7

Modification of wastewater storage basin Nos. 6 and 7 will not be required under contingency plan flow conditions. The basis of design for these basins, from the PER (Foth & Van Dyke, 1995/1998b), was to provide sufficient capacity to store a 100 yr/24 hr storm in addition to storing a minimum of a 7 days of WWTF influent flow. This same criteria can be met, using the contingency plan flow rate, without increasing the basin capacity. The 7-day storage criteria was based on removing the evaporators from service for a 7-day period each year for cleaning and maintenance. It was assumed that, under worst case conditions, both units would be removed from service. Under contingency plan conditions, up to five equally sized evaporators would be in operation, and it is expected that a maximum of two of these evaporators could be off-line for maintenance at any one time. Consequently, sufficient storage capacity would need to be provided to store the WWTF inflow corresponding to the treatment capacity represented by two evaporators. Since all of the treatment trains will be equally sized, the WWTF inflow storage capacity requirement is the same as under PER design flow conditions. Therefore, additional storage capacity is not required.

4.1.2 Wastewater Treatment Facilities

The wastewater treatment facilities includes the following components:

- influent pumping
- lime feed/makeup/delivery system
- lime precipitation tanks
- solids contact clarifiers
- sulfide reaction tanks

- gravity filters
- reverse osmosis systems
- evaporator systems
- air stripper systems
- pH adjustment systems
- effluent pumping

The initial WWTF will consist of two treatment trains, each sized for an inflow capacity of 370 gpm. Details of the WWTF design are available in the PER (Foth & Van Dyke, 1995/1998b). If system evaluations during mine development indicate the probability of mine inflows exceeding the current design, and if secondary/tertiary grouting does not control inflow, then additional treatment trains will be constructed. Each additional treatment train will be designed with a capacity of 370 gpm, which matches the design capacity of the treatment trains to be constructed initially. Based on an assumed maximum mine inflow of 1,500 gpm, up to three additional treatment trains (each sized to treat up to 370 gpm) would be added. Depending on the number of additional trains to be constructed, and the location of these trains, a common use design approach or a stand-alone design approach may be used. Common use facilities would include systems such as chemical storage, influent pumping, and effluent pumping. If possible, common facilities would be used to serve all of the treatment trains. Alternatively, if required, the new treatment trains could be designed as stand-alone systems. Evaluation of design details of this nature would need to be made at the time that revised mine inflow projections are available and the number of required additional treatment trains is determined.

To accommodate expansion of treatment facilities that might be required as a result of implementing the WMCP, some changes to the existing plant site layout, as shown in Figure 3-2 of the PER (Foth & Van Dyke, 1995/1998b), would be made. These could include: (a) relocating the process water and gland water tanks to the area directly south of the reagent preparation and distribution building and east of the grinding building; (b) locating the existing evaporator/condenser units directly south of the existing clarifiers; and (c) locating the soft water mitigation tanks east of the acid storage tank.

By making these changes, the area adjacent to the south side of the existing WWTF would be available for construction of a building extension that would house the treatment equipment required for treating the increased inflow. New evaporator units could be installed to the west of the building extension. Additional clarifier capacity could be installed south of the acid storage tank. Details of the layout of the existing WWTF and the potential for expanding the facilities, will be addressed as part of a contingency section in the Final Engineering Report.

4.1.3 WWTF Solids, Filter Backwash, and Evaporator Brine Management

As described in the PER (Foth & Van Dyke, 1995/1998b), and as illustrated in Figure 3-1 of the WMCP, lime solids and filter backwash from the WWTF will be discharged to the tailings management area (TMA). Under the assumed contingency plan mine inflow of 1,500 gpm, the increased solids/backwash flow to the TMA will result in discharge of excess water from the mill circuit back to the WWTF. As indicated in the PER (Foth & Van Dyke, 1995/1998b), brine from the WWTF evaporators will be used in the mine paste backfill operations. This would continue to be the case under contingency plan flow conditions.

4.1.4 Discharge Lagoons

Based on the PER design criteria, two discharge lagoons will be provided, with each lagoon having a two day storage capacity based on a maximum flow of 636 gpm. The lagoons will be operated in an alternating one day fill/one day discharge cycle. Under the maximum PER design flow, each lagoon has one day of "contingency" capacity. Due to a reduction in the lagoon "contingency" capacity for effluent flows exceeding the PER design basis, additional discharge lagoon storage capacity would need to be provided under contingency plan flow conditions.

The design approach to be used under contingency plan mine inflows is as follows:

a) Additional discharge facilities will be constructed if flow projections during mine development indicate mine inflows in excess of the PER design value of 600 gpm.

b) A minimum of three storage structures will be required, with each structure providing one day of effluent storage. Two of the structures will be used in the normal operation fill/discharge alternation cycle. The third structure will provide "contingency" storage capacity.

c) For effluent flows up to 1,271 gpm, the "existing" lagoons will account for two of the storage structures, and a third structure will be added. At an effluent flow rate of 1,271 gpm, each of the existing lagoons would provide one day of effluent storage, and the volume requirement for the third structure would be approximately 1.8 million gallons in order to provide an additional one day storage capacity.

d) For flow conditions between 1,271 gpm and the maximum assumed contingency plan effluent flow of 1,532 gpm, the two "existing" lagoons would be modified through a hydraulic connection to serve as a single storage structure, and two additional storage structures would be constructed. Each of the additional storage structures would be sized for a storage capacity equal to one day of effluent flow. At the peak contingency plan effluent flow of 1,532 gpm, each of the additional storage structures would require a storage volume of approximately 2.2 million gallons.

Additional storage capacity could be provided through the use of either new lagoons or new tanks. Based on area requirements and minimization of potential dust contamination of the effluent, new lagoons would be located in the upland area north of the existing discharge lagoon area. This activity would expand the limits of plant site disturbance shown on Figure 2-1 of the MPA (Foth & Van Dyke, 1995/1998c). An alternative to the additional lagoon storage option could be to provide covered concrete or steel tanks for the additional effluent storage need. These facilities could be located in the construction laydown area south of wastewater basin

No. 5. The sizing of additional effluent storage facilities, if required, would be determined based on flow projections developed during mine development.

4.1.5 Effluent Discharge Pumping Facilities and Force Main

WWTF effluent will be pumped from the discharge lagoons to the SAS. Modification to the effluent discharge system would be required if the effluent flow increases significantly above the PER design rate of 636 gpm. Discharge flows up to about 1,000 gpm could be accommodated in the 10 inch force main proposed in the PER (Foth & Van Dyke, 1995/1998b) by replacing the pumps with pumps of a higher head capacity. If flow projections during mine development indicate the probability of effluent flows above 1,000 gpm, a second force main would be constructed.

4.2 Area H Soil Absorption System

A technical memorandum discussing the soil absorption system alternatives for meeting contingency flow conditions has been prepared and is provided in Appendix A of this report. The key elements of the analysis are summarized in this section.

The thickness of the outwash material in the area is an average of 70 feet. Numerical analysis presented in the SAS PER (Foth & Van Dyke, 1998b) has shown that the aquifer thickness that will transmit water from the SAS will be about 70 feet. Using a conservative aquifer thickness of 45 feet and allowing the groundwater elevation of 2 feet beneath the SAS distribution piping, the hydraulic capacity of the Area H SAS is increased to approximately 1,814 gpm. To accommodate a flow equivalent to the 1,814 gpm hydraulic capacity of the SAS, the hydraulic loading rate of each cell can be increased by reducing the hole spacing in the lateral distribution piping from 10 feet to 5 feet, and by specifying ¾ inch washed gravel as the distribution pipe bedding material. Appendix A includes a detailed discussion of the proposed design modifications. Figure 4-1 shows the layout of the Area H SAS, along with the flow distribution at each cell necessary to accommodate a mine inflow of 1,500 gpm. Note that at a mine inflow of 1,500 gpm, the discharge to the SAS (1,532 gpm) is less than the hydraulic capacity of the site (1,814 gpm).

The Area H SAS would be initially constructed according to the design presented herein so that modifications would not be necessary if the contingency flow scenario were to occur.

Impacts to wetland surface water flow in the vicinity of the Area H SAS as a result of an assumed contingency flow were evaluated using the same method of evaluation as presented in the SAS PER (Foth & Van Dyke, 1998b, Appendix K). This evaluation is included as Appendix B of this report. The assumed contingency flow rate was 3 times that used in Appendix K of the SAS PER (Foth & Van Dyke, 1998b). The calculations in Appendix B indicate that increased flow has no significant impact on the flood plain of Swamp Creek.

4.3 Other Discharge Options

In the very unlikely event that additional discharge capacity is required, continuous mitigation or a SAS site at Area A could be implemented. As described above, Area A is a potential SAS site that was evaluated as part of the SAS PER (Foth & Van Dyke, 1998b), and could likely accommodate modest amounts of water. A more likely scenario is that continuous mitigation would be used to provide additional discharge capacity in the unlikely event that it were needed. The mitigation water used for discharge to soft water lakes would be pumped from the storage system at the wastewater treatment facility to the appropriate water bodies. The rate of discharge would be designed to match the volume of water lost from the soft water lakes due to the mine dewatering. The distribution system would not be impacted by the discharge of mitigation water on a continuous basis. Therefore, there would be no need to provide greater capacity in the mitigation water distribution system to address the alternative for continuous discharge.

4.4 Summary of Contingency Plan Engineering Modifications

The wastewater treatment facilities include wastewater storage basin Nos. 6 and 7, wastewater treatment facilities equipment, discharge lagoons, effluent discharge pumps, and force main to the SAS.

Wastewater storage basins No. 6 and No. 7 would not have to be modified for the contingency flow condition. The WWTF equipment could be upgraded by adding additional treatment trains adjacent to the existing equipment. The discharge lagoon capacity would have to be increased if the amount of treated water exceeds the maximum PER design flow of 636 gpm. This would require one or two additional lagoons (or tanks) depending on the anticipated flow (one additional lagoon or tanks for flows up to 1,271 gpm; two additional lagoons or tanks if flows exceed 1,271 gpm).

The Area H SAS will be initially constructed with enhancements to increase its hydraulic capacity to approximately 1,814 gpm. Therefore, upgrades to the SAS to handle the contingency flow would not be necessary. A schedule for implementation of the WMCP is included in Section 5 of this report.

5 Contingency Plan Implementation

Prior sections of this report have described the underground mine development process and the three main phases of mine development as they relate to groundwater inflow. Alternatives for managing mine inflows that exceed the design capacity of the water treatment system have also been evaluated and the preferred alternatives have been selected. This section of the report addresses the implementation of the contingency plan. Specifically, this section addresses the decision process that will be followed to determine if the contingency plan needs to be implemented. Also addressed in this section of the report is the implementation schedule for upgrading the water management facilities once a decision has been made to implement the contingency plan. Note that an inherent assumption in the implementation process is to implement the plan in advance of mine inflow actually exceeding the capacity of the treatment system. It will be necessary for NMC to implement the contingency plan prior to inflow exceeding the capacity of the treatment system in order to facilitate mine development. The decision criteria and implementation schedule described in this section have been developed with this goal in mind.

5.1 Decision Process for Implementation

The key question with regard to implementing the contingency plan is: how and when will it be known whether or not the contingency plan will be required. This question will be addressed during each key stage of the mining process, i.e., during the period of drift development, during ore delineation and draining, and during actual mining. Figure 5-1 displays a decision flow chart for implementation of the contingency plan during the mine life. This figure summarizes the decision process that is discussed in this section.

Wastewater Treatment Implementation Condition 1: During the pre-production period prior to draining of the ore body, analysis of inflow during the initial development of hanging wall workings will provide the primary initial information that will be used to determine the need for the contingency plan. During this period, sustained inflows will likely be so low that the contingency plan is clearly not needed. However, though unlikely, flows may be high enough to clearly warrant the plan. Sustained inflows during this period of time in excess of 450 gpm (the BEJ prediction) would clearly indicate that when larger areas of the mine are open during the ore extraction process, and the overall seepage area of the mine increases, sustained inflow greater than the design capacity of the wastewater treatment facilities would be likely, and the contingency plan would be implemented. This is the first condition, as specified in Figure 5-1, that would lead to the implementation of the contingency plan.

Wastewater Treatment Implementation Condition 2: Following the decision flow chart, inflow data gathered during pre-production development will be used to re-evaluate hydraulic parameters and revise predictions of mine inflow. The new analyses would benefit from additional drawdown information gathered by the environmental monitoring plan. A simple analysis to assess the groundwater response during the initial development period would be to use the measured head differences across the till/saprolite to analytically calculate the average leakance or vertical conductivity of that unit. The leakance of this combined unit appears to be the most critical hydraulic parameter for determining long-term, sustained mine inflow. The general relationship for this analysis is derived from Darcy's Law as: $K/b = Q/(\Delta HA)$, where K is the vertical conductivity, b is the saprolite/till thickness (K/b is the leakance), Q is the total inflow to the workings, ΔH is the areal-average head difference between the till/saprolite and upper bedrock, and A is the area over which the head difference is induced. The area "A" and the average head difference will be developed from simple potentiometric surface maps derived from monitoring well data.

An analysis based on Darcy's Law assumes that all of the inflow comes from the overburden, and that the till/saprolite provides the main resistance to that flow. This assumption is consistent with the extensive analysis of the hydrogeologic conditions at the site. This type of analysis is likely to yield a range of hydraulic conductivity values for the till/saprolite. These values would then be entered into the regional groundwater flow model to assess whether or not the steady-state zinc mine inflow will be above the design treatment capacity and, therefore, whether or not the contingency plan will be required. If the revised inflow predictions indicate that predicted inflows will exceed the wastewater treatment capacity, the contingency plan will be implemented. This represents the second condition in Figure 5-1 that would lead to contingency plan implementation. Note that a complete recalibration of the model would not be completed at this juncture. Rather, the revised estimates of till/saprolite hydraulic conductivity would be entered into NMC's existing regional groundwater flow model to reassess inflows. Recalibration of the model would be completed at a later point in mine development when underground conditions become less transient in nature.

Wastewater Treatment Implementation Condition 3: During the ore delineation and draining phase, NMC will remove as much water as possible from the area to be mined in order to reduce pressure in the area and lower inflows that would occur during initial opening of mining stopes. Again, NMC will be limited by the treatment capacity available. If NMC finds that it can quickly dewater the ore body, or cannot pump water from the ore body at a high rate for a long time period, this will be a good indication that the contingency plan will not be needed and no further analysis on the need to implement the plan would be required until prior to the commencement of mining. Conversely, it is theoretically possible, although unlikely, that the ore cannot be effectively dewatered at a rate that approaches the treatment capacity of the wastewater treatment facilities. If this condition were to occur, it would be a good indication that the contingency plan would need to be implemented. This represents the third condition in Figure 5-1 that would lead to contingency plan implementation.

Wastewater Treatment Implementation Condition 4: There is a possibility that, during the draining period, inflow information alone will not be conclusive as to whether or not implementation of the contingency plan will eventually be necessary. In this case, some analysis of the groundwater response during the ore drainage period will be useful. During this period of time, mine inflow and groundwater levels around the mine will be closely monitored in accordance with the project's monitoring plan. A key component will be the measurement of water levels in the glacial till/saprolite and upper bedrock. This will provide the information needed to refine current estimates of the leakance for the massive saprolite and Early Wisconsinan till.

Again, simple Darcy Law analytical calculations, as described above, will be used to provide a revised range of hydraulic conductivities for the till/saprolite layer. These values will be used in the regional groundwater flow model to assess the long term steady-state inflow to the mine. If this rather quick analysis indicates that inflow would clearly exceed the capacity of the treatment system, the contingency plan would be implemented. This represents the fourth condition that would lead to the implementation of the contingency plan.

Wastewater Treatment Implementation Condition 5: Groundwater model recalibration during ore drainage represents the final check on the need for contingency plan implementation prior to mining. Approximately six months prior to commencement of ore removal, NMC will recalibrate the regional groundwater flow model. This point in the drainage process will provide the equivalent of a long term stress on the system of sufficient magnitude to yield extensive drawdown information for model recalibration and verification. After completion of the recalibration process, revised predictions of mine inflow will be completed. If these predictions indicate that mine inflow could exceed the treatment capacity of the wastewater treatment facilities, the contingency plan will be implemented. This represents the fifth condition in Figure 5-1 that would lead to the implementation of the contingency plan.

Wastewater Treatment Implementation Condition 6: The next decision step regarding the need to implement the contingency plan will be during the removal of ore from the mine stopes. If at any time during active mining sustained groundwater inflow exceeds 550 gpm, the contingency plan will be implemented. Sustained groundwater inflow will be determined by a thirty day rolling average. This trigger criteria represents the sixth condition in Figure 5-1 that would lead to implementation of the contingency plan.

The decision process described above logically assumes that mine inflow will slowly increase during mine development. With proper monitoring and analysis, the need for implementation of the contingency plan will be determined in advance of the treatment capacity being exceeded. This will allow ample time for NMC to implement secondary grouting programs, and finally, if necessary, to upgrade the wastewater treatment and disposal facilities. It is highly unlikely that sustained mine inflow could rapidly increase to near the treatment capacity of the wastewater treatment facilities during the pre-production period. Although this is highly unlikely, and is analogous to a completely unpredictable act of nature, it is a scenario that must be addressed in the implementation decision process. Accordingly, if at any time during the pre-production period sustained mine inflow exceeds 550 gpm, as measured on a 30-day rolling average, the contingency plan will be implemented. Mine development will cease if there is an indication that continued mine development will lead to an exceedance of the treatment capacity and disposal capacity of the water management systems.

Discharge Implementation: A key component of the contingency plan is the ability of the SAS to handle the treated mine water. Analysis presented above indicates that the SAS will be able to handle up to approximately 1,814 gpm. It is possible, although unlikely, that the SAS may not be able to handle this capacity. As such, a parallel contingency plan (Figure 5-2) is required for the water disposal system that ties into the overall decision process for contingency plan implementation. Accordingly, NMC will, on a semi-annual basis, evaluate the capacity of the SAS. If at any point in time it is determined that mine inflow will exceed the SAS capacity, continuous mitigation would be implemented.

In the unlikely event that continuous mitigation were required due to excessive flows and the need for additional discharge capacity, some of the soft water lakes would be elevated to Level I water bodies. As such, continuous mitigation would afford substantial capacity to meet the demand that would not be available at the Area H SAS site. Accordingly, it is highly unlikely that a SAS site at Area A would ever be required.

Regulatory Interaction: A final component of the contingency plan is the administration of the plan as it relates to regulatory involvement. NMC will implement the plan in consultation with the WDNR. Key monitoring data such as groundwater drawdown, mine inflow, etc., will be provided to the WDNR on a routine basis. Results from the analysis of the data, such as revised estimates of mine inflow based on simple analytical calculations, revised model predictions, recalibration, and SAS capacity will also be provided to the WDNR on a routine basis. As a result, the decision on the need to implement the contingency plan will be completed with the full involvement of regulatory agencies.

5.2 Implementation Schedule

The schedule for implementing the various components of the contingency plan are described below. The discussion of time frames is based on a starting point of when the decision process for implementation described in Section 5.1 indicates a need to upgrade flow handling capability in the system. The schedule assumes that permitting of the upgrades will be covered under prior Crandon Project permit approvals, and, therefore, permitting will not impact the implementation of the upgrades.

5.2.1 Wastewater Treatment Facilities

The wastewater treatment facilities are the component of the contingency plan with the longest lead time for implementation. This is due to the need to fabricate equipment to meet the strict effluent limitations applied to the discharge of treated water to the SAS. The longest lead time items of the wastewater treatment facilities are the evaporators. Based on discussions with leading evaporator manufacturers, NMC can expect that from placement of the order, it will take 5 months for fabrication, 2-3 months for field construction, and another 1-2 months for startup. The design time for the system upgrade should be minimal due to the plan to have identical treatment trains in parallel arrangement. Assuming a 1-2 month design period prior to release for fabrication, the total time for implementation of the wastewater treatment facilities upgrade would be approximately 9-12 months.

In the event that flows exceed 1,000 gpm, it would also be necessary to construct an additional pipeline from the discharge lagoons to the Area H SAS. This could easily be accomplished within the 9-12 month time frame.

5.2.2 Area H Soil Absorption System

As discussed earlier, the Area H SAS system would be initially constructed to handle flows up to approximately 1,814 gpm, and therefore would not require any upgrades to meet contingency flow conditions.

5.2.3 Continuous Mitigation

The mitigation system will be constructed in the time period specified in the *Crandon Project Surface Water Mitigation Plan* (Foth & Van Dyke, 1998a). Providing continuous mitigation water discharge would not require any additional construction. The system would simply run on a continuous basis rather than start up and shut down to cover low water periods.

If additional capacity was required, additional evaporators would be needed for the wastewater treatment facilities. Evaporators are the critical component for the implementation schedule of the wastewater treatment facilities. As stated above, an evaporator would take approximately 9-12 months to become operational. During this time period, additional pumping equipment, storage structures, and/or piping for the continuous discharge of mitigation water could be constructed.

6 Conclusions

The WMCP provides a description of underground mine development measures that will be used to control mine inflow and facilitate dewatering. Data gathered from these procedures will provide early indications during the mine development process regarding the expected mine inflows as mining progresses. In the unlikely event that the expected mine inflows are projected to exceed the wastewater treatment facilities design basis of 600 gpm mine inflow, contingency plan measures outlined in this MWCP will be implemented. The design basis flow for this plan (1,500 gpm mine inflow) was selected based on the WDNR's review of bedrock pumping tests (USGS, 1999) which indicate that the WDNR's "worst case" prediction will likely exceed 1,200 gpm.

Implementation of the contingencies described herein could be accomplished within 12 months of notification that predicted inflows will exceed the project's design basis. Table 6-1 summarizes the implementation schedule for the various treatment/discharge facilities.

Table 6-1

Implementation Schedule

Facility	Design Capacity[1]	Capacity per MWCP	Time to Implement
SAS	~700 gpm	~1,800 gpm	-0-
WWTF	~630 gpm	~1,800 gpm[2]	9-12 months
Pipeline	~1,000 gpm	~2,000 gpm	<6 months
Treated Water Discharge Lagoons or Tanks	~630 gpm	~1,530 gpm[3]	<9 months

[1] Design capacity from PER (Foth & Van Dyke, 1995/1998b).
[2] Capacity could be increased by adding additional treatment trains.
[3] Capacity could be increased by increasing size of additional lagoons or tanks.

Prepared by: GR
Checked by: DMR

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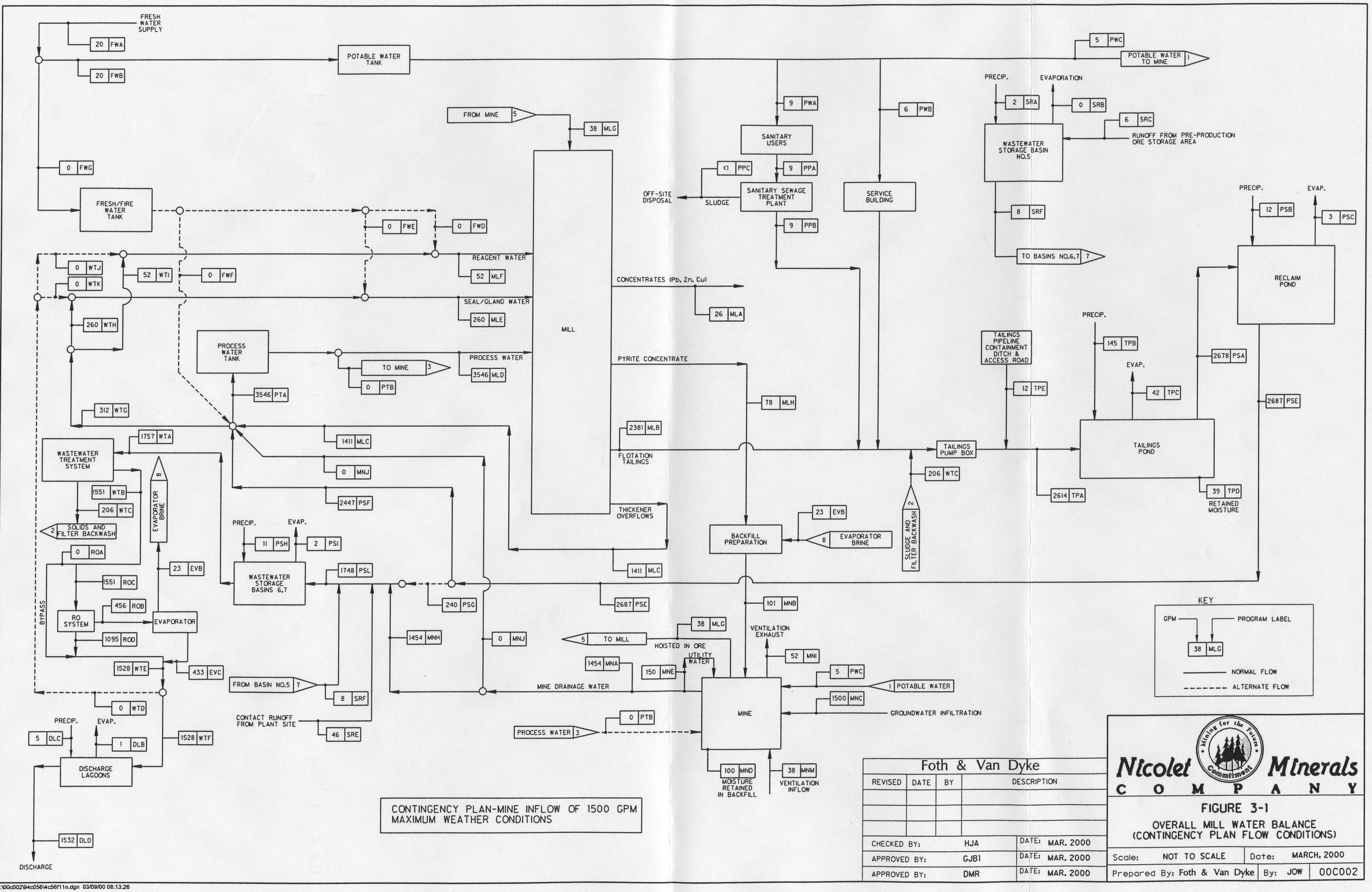
FIGURES FOR MINE WATER MANAGEMENT CONTINGENCY PLAN



Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION

CHECKED BY:	HJA	DATE:	MAR. 2000
APPROVED BY:	GJB1	DATE:	MAR. 2000
APPROVED BY:	DMR	DATE:	MAR. 2000

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FIGURE 3-1

OVERALL MILL WATER BALANCE
(CONTINGENCY PLAN FLOW CONDITIONS)

Scale:	NOT TO SCALE	Date:	MARCH, 2000
Prepared By: Foth & Van Dyke		By: JOW	00C002

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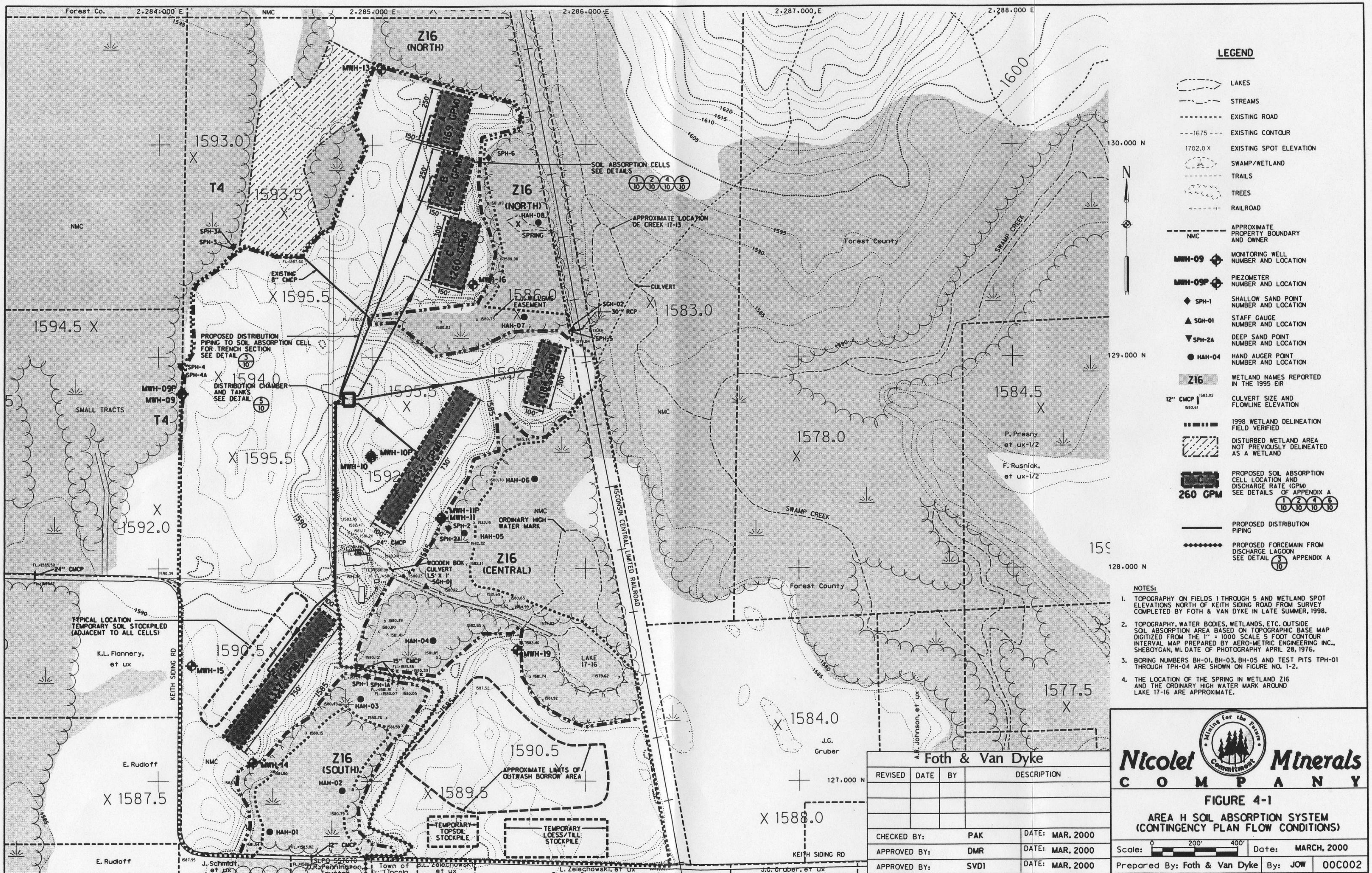

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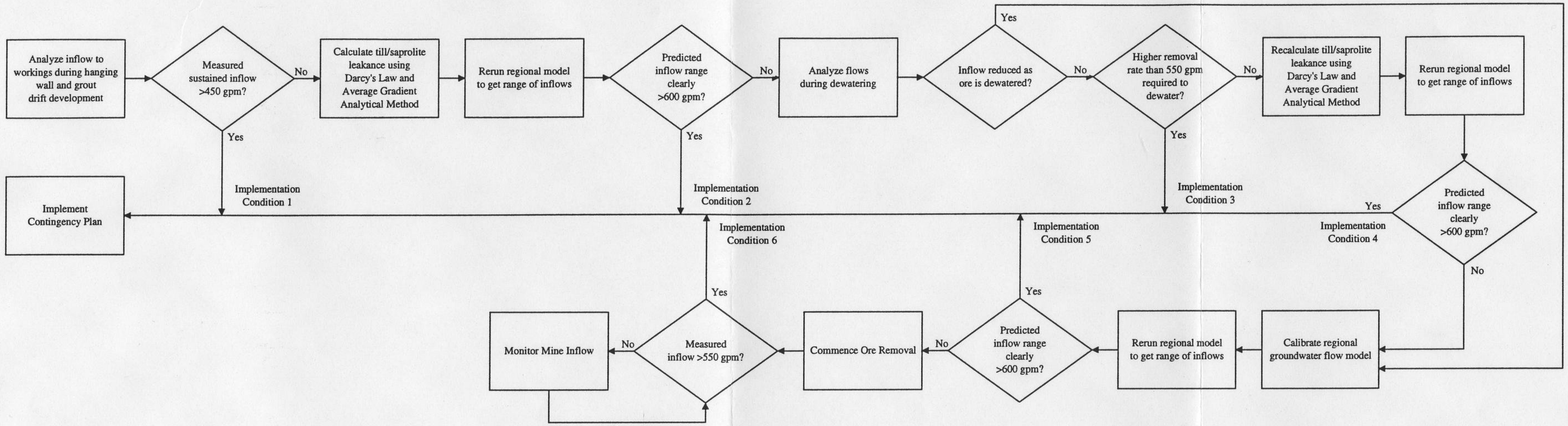

Note: If at any time sustained mine inflow (as measured on a 30-day rolling average) exceeds 550 gpm during the pre-production period, the contingency plan will be implemented.

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CHECKED BY:	MLD2	DATE:	MAR. 2000
APPROVED BY:	SVD1	DATE:	MAR. 2000
APPROVED BY:	DMR	DATE:	MAR. 2000

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FIGURE 5-1

DECISION FLOWCHART FOR IMPLEMENTATION OF WASTEWATER TREATMENT CONTINGENCIES

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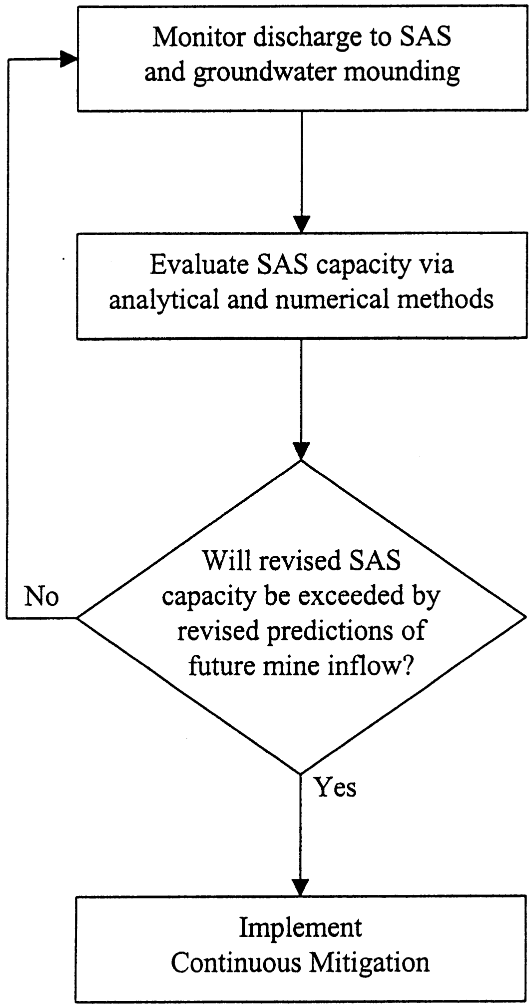

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CHECKED BY:	MLD2	DATE: MAR. 2000
APPROVED BY:	SVD1	DATE: MAR. 2000
APPROVED BY:	DMR	DATE: MAR. 2000

Nicolet Minerals COMPANY

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FIGURE 5-2

DECISION FLOWCHART FOR IMPLEMENTATION OF WATER DISCHARGE CONTINGENCIES

Scale: NOT TO SCALE		Date: MARCH, 2000	
Prepared By: Foth & Van Dyke	By: DAT		00C002

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Appendix A

Technical Memorandum - Contingency Flow Treated Water Discharge Evaluation

Foth & Van Dyke
Memorandum

March 10, 2000

TO: Steve Donohue, Foth & Van Dyke

CC: Gordon Reid, Nicolet Minerals Company
Jerry Sevick, Foth & Van Dyke
Denis Roznowski, Foth & Van Dyke
Master File

FR: Phil Korth, Foth & Van Dyke PAK

RE: Crandon Project - Soil Absorption System Evaluation for Mine Inflow Contingency Plan

Area H Soil Absorption System Contingency Flow Design Basis

Area H is in the northeast portion of the study area and is identified as a suitable location for the proposed soil absorption system (SAS). This site has excellent characteristics for a SAS and has adequate capacity to handle the treated wastewater flows predicted in the *Preliminary Engineering Report for Wastewater Treatment Facilities* (PER) (Foth & Van Dyke, 1995/1998). The original design analysis assumed operational constraints that are typically applied when the soil column is used for treatment of effluent. Since the soils at the SAS are not being relied upon for treatment, operational constraints that are focused purely on the hydraulic capacity will provide for a higher total hydraulic discharge capacity.

The first design value to be considered is the separation between the distribution piping and the maximum groundwater elevation beneath the SAS. The PER used a conservative value of 5 ft for separation. This is consistent with good design practice when the soil is an integral part of the wastewater treatment process. In this situation, the NMC wastewater treatment facility will be treating water to a level that will be cleaner than the groundwater it is discharging to. The soil will play no part in the wastewater treatment process. Therefore, the 5 ft of soil separation is not required, and, based on discussions with the WDNR, a 2 ft separation is appropriate for this application. This change increases the maximum elevation head of the groundwater under the system, and thus increases the flow rate that can be discharged.

Area H has a deep layer of outwash material that is the primary groundwater aquifer to the nearby wetlands and Swamp Creek. The average thickness of the outwash is about 70 ft. Numerical analyses in the PER have shown that the aquifer thickness that will transmit the infiltrated water is approximately 70 ft. By using an aquifer thickness of 45 ft in the hydraulic capacity calculation, and 2 ft of separation between the bed and groundwater mound, the hydraulic capacity of the system exceeds 1,500 gpm.

Specifically, an allowable groundwater elevation of 2 ft beneath the SAS distribution piping results in a hydraulic capacity up to 1,308 gpm. The combination of a 2 ft allowable groundwater elevation beneath the SAS and the use of an aquifer thickness of 45 ft results in a hydraulic capacity of approximately 1,814 gpm (see calculations in Attachment A). This value is greater than the maximum predicted contingency value of 1,532 gpm. These calculations confirm that adequate hydraulic capacity is available at Area H to accept all the treated water identified in the contingency plan.

The increased flow to the SAS can be accomplished without changing the dimensions of the individual cells.

The vertical permeability of the soils at the SAS site, as measured in the field, was approximately 80 ft/d. With the contingency flow rate of approximately 1,500 gpm, the maximum hydraulic loading rate is only approximately 1.6 ft/d, well below field measured values

Area H Soil Absorption System Engineering Modifications

An evaluation of Area H shows the maximum hydraulic capacity of the site is capable of handling the contingency flow of 1,532 gpm. The maximum hydraulic capacity is a characteristic of the soil, and no engineering modifications are required to reach the maximum capacity.

As water application rates are increased, the importance of even water distribution over the SAS is also increased. If too much water is loaded at a single point, the system may fail due to inadequate capacity within a section of the soil absorption bed. The original design had distribution laterals with a spacing of 20 ft between laterals. Each lateral had orifices spaced at 10 ft apart. The piping system was designed to be installed over a 6 inch gravel layer intended to convey the water away from each orifice.

An additional evaluation was done on the water distribution system. Using the vertical permeability of 8 ft/day (10 times less than field measurements), about 21 sq ft of sand area is needed to allow the water to flow vertically into the groundwater. The gravel layer must transmit water at a rate of 0.58 ft/minute to dispose of the water discharge rate of approximately 1,500 gpm, as shown in the attached calculations. A clean aggregate, approximately ¾ inch in size, is capable of transmitting water at a rate of up to approximately 50 ft/min. Therefore, specifying a clean ¾ inch gravel coarse aggregate beneath the distribution system will allow passage of the contingency flow of 1,532 gpm, as well as the maximum hydraulic capacity flow of 1,814 gpm. It is recommended that a clean, coarse aggregate be used beneath the distribution system to allow a larger amount of water to be distributed over the SAS bed.

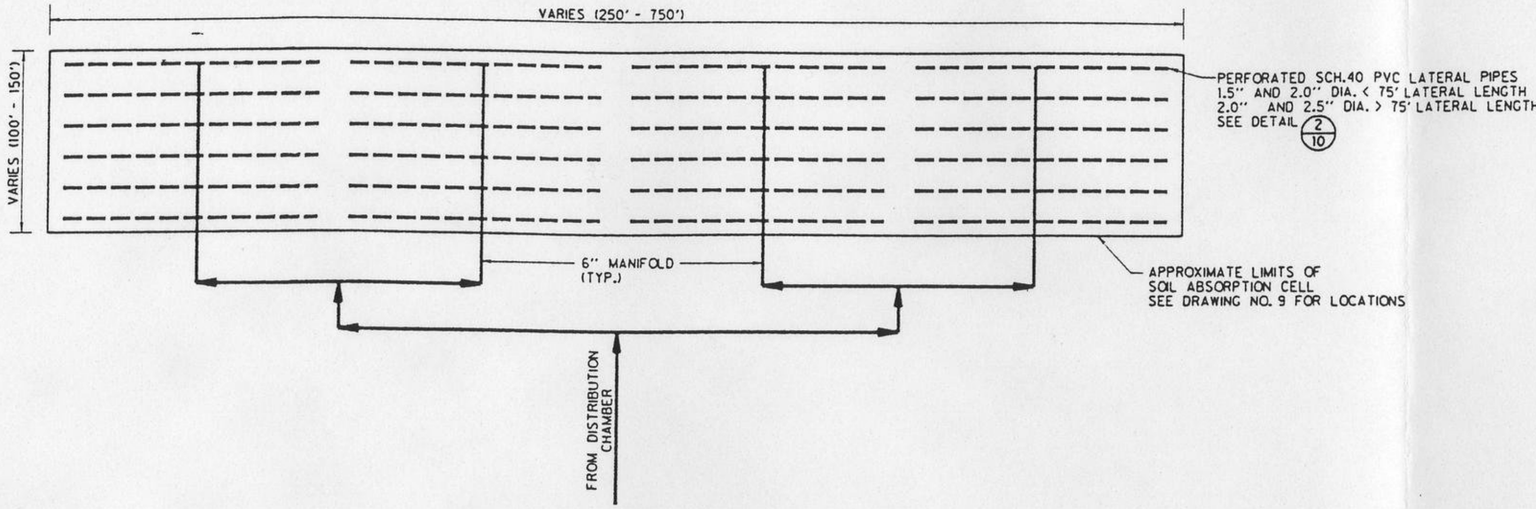
With the higher maximum flow rates, pipe capacity and the number of orifices could limit the water discharge rate. It is recommended the design be modified to distribute more flow by changing the orifice spacing to 5 ft. This change will also reduce the required sand area to 10.3 sq ft for each orifice (a circular area of 3.6 ft in diameter). Drawing No. 10 from the *Preliminary Engineering Report for the Crandon Project Soil Absorption System* (SAS PER) (Foth & Van Dyke, 1998) has been revised to show the new gravel layer specification and reduced orifice spacing, and is included as an attachment to this memo.

PAK:cer1
Attachments

References

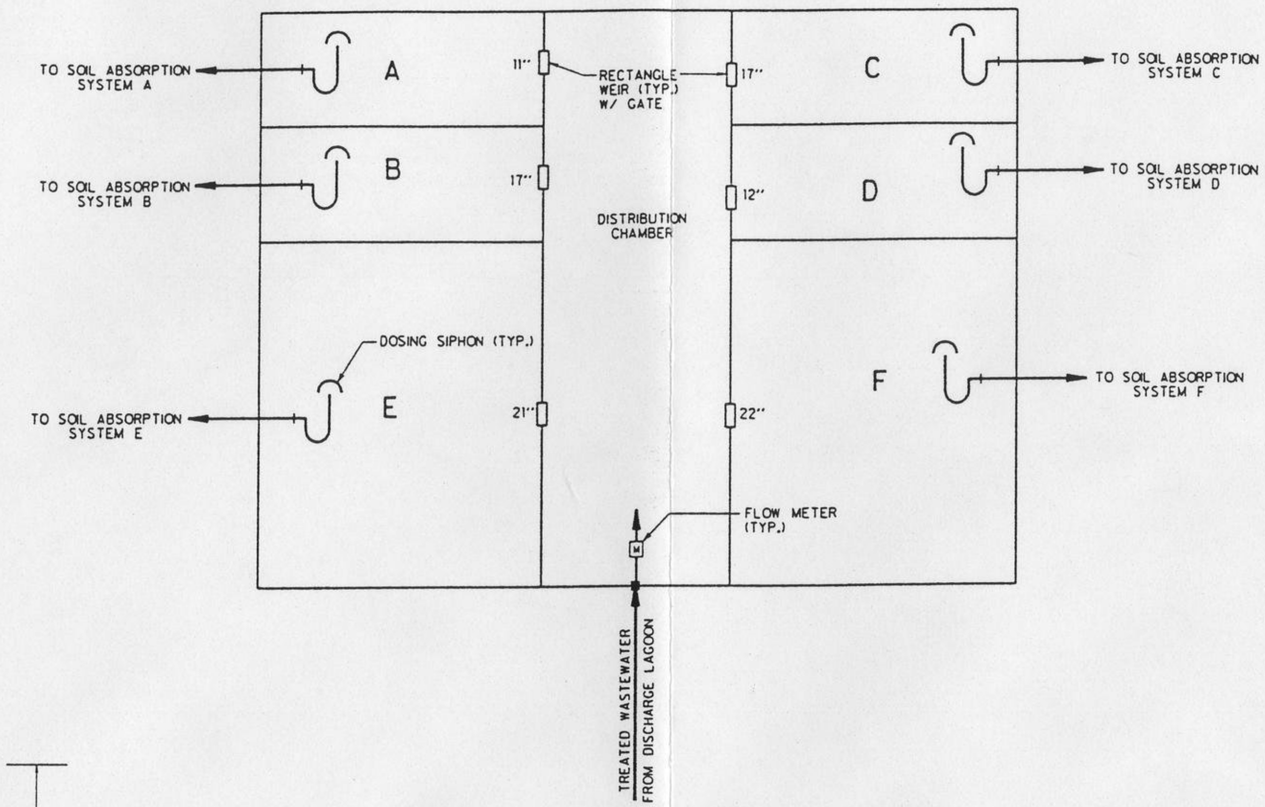
Foth & Van Dyke, 1995/1998. *Preliminary Engineering Report for Wastewater Treatment Facilities.* Originally Issued May 1995. Updated December 1998.

Foth & Van Dyke, 1998. *Preliminary Engineering Report for the Crandon Project Soil Absorption System.* November 1998.



1/10 SOIL ABSORPTION SYSTEM - PLAN SECTION

NOT TO SCALE



5/10 DISTRIBUTION CHAMBER AND TANKS

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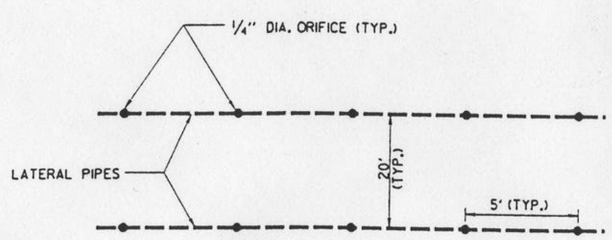
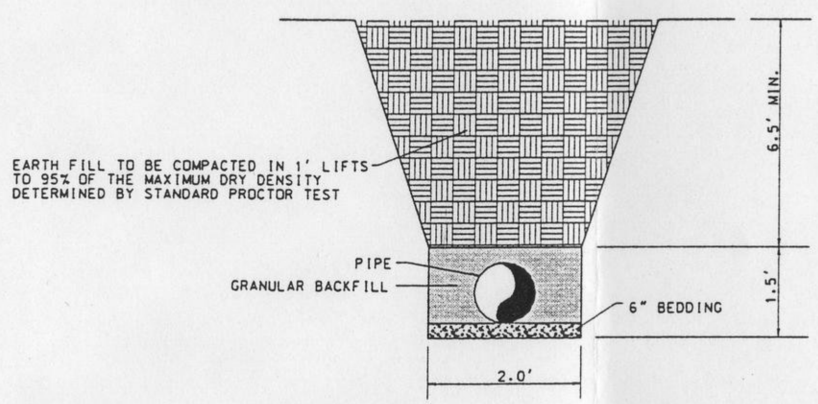


2/10 LATERAL DETAIL

NOT TO SCALE



3/10 TYPICAL TRENCH SECTION - BURIED PIPELINES

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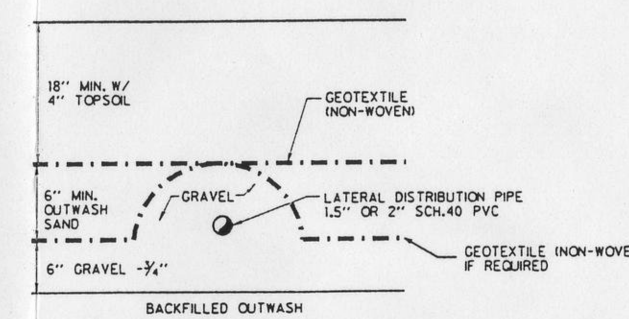

6/10 LATERAL DISTRIBUTION PIPING DETAIL

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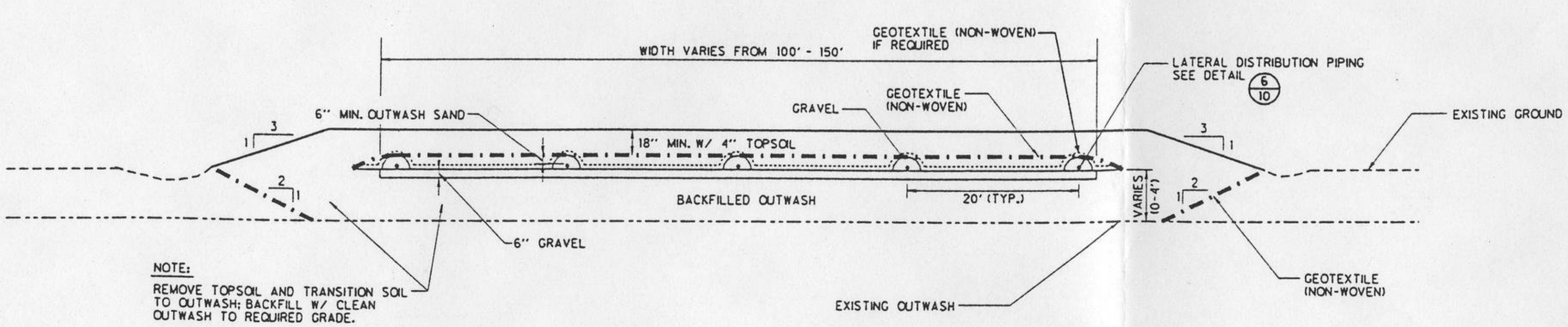

4/10 SOIL ABSORPTION SYSTEM - TYPICAL SECTION

NOT TO SCALE

Foth & Van Dyke

REVISED	DATE	BY	DESCRIPTION
⚠	3/2000	PAK	HOLE SPACING/GRAVEL SPECIFICATION

REUSE OF DOCUMENTS

THIS DOCUMENT HAS BEEN DEVELOPED FOR A SPECIFIC APPLICATION AND NOT FOR GENERAL USE. UNAPPROVED USE IS THE SOLE RESPONSIBILITY OF THE UNAUTHORIZED USER. THE DRAWING PRESENTS TYPICAL REPRESENTATION; REFINEMENTS MAY BE MADE DURING FINAL DESIGN.

Nicolet Minerals COMPANY

Mining for the Future · Commitment

TITLE

DETAILS

SCALE AS SHOWN	STATE WISCONSIN		COUNTY FOREST		
DRAWN BY JOW	DATE NOV.'98	CHECKED BY REM		DATE NOV. '98	
APPROVED BY SAD2	DATE NOV.'98	APPROVED BY GWS		DATE NOV. '98	
APPROVED BY	DATE NOV.'98	NICOLET MINING COMPANY		DATE NOV. '98	
DRAWING NO. 10			SHEET 10 OF 10	REVISION NO.	

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Attachment A

Soil Absorption System Design Calculations
Contingency Flow Conditions

Hydraulic Capacity Calculations (Pipe Elevation 2 ft Above Maximum Groundwater Mound and 30 ft Thick Aquifer)

CRANDON PROJECT
SOIL ABSORPTION SYSTEM DESIGN

RUDLOFF PROPERTY - AREA H

AQUIFER DEPTH (D) = 30 FEET

DESIGN STEP 1 - CALCULATE MAXIMUM CELL WIDTH

PIPE ELEVATION IS 2 FEET ABOVE MAXIMUM GW MOUND

W = ((K*D*H)/(d*L))
WHERE:

K = HYDRAULIC CONDUCTIVITY - FT/DAY
D = AQUIFER DEPTH, FT
PIPE ELEVATION = APPLICATION POINT FOR WASTEWATER DISPOSAL
DISCHARGE ELEVATION = MAXIMUM WATER LEVEL IN DISCHARGE WETLAND
MAXIMUM GROUNDWATER MOUND ELEV. = PIPE ELEVATION - **2 FEET**
H = VERTICAL GROUNDWATER ELEV.; MAX MOUND - DISCHARGE ELEV., FT
d = DISTANCE FROM CENTER OF APPLICATION SYSTEM TO DISCHARGE POINT, FT
L = HYDRAULIC APPLICATION RATE, FT/DAY
W = ABSORPTION POND MAX WIDTH, FT

CELL	A	B	C	D	E	F
K	75	75	75	75	75	75
D	30	30	30	30	30	30
PIPE ELEV.	1597	1597	1595	1591	1592	1592
DISCHARGE ELEV.	1581	1581	1581	1580.5	1580.5	1580.5
MAX MOUND ELEV.	1595	1595	1593	1589	1590	1590
H	14	14	12	8.5	9.5	9.5
d	300	200	200	200	250	250
L	0.70	1.05	0.90	0.96	0.86	0.86
W	150	150	150	100	100	100
LENGTH	250	250	300	300	730	750

Prepared by: PAK
Checked by: TWS

DESIGN STEP 2 - CALCULATE SIZE AND APPLICATION RATE TO MEET MAXIMUM MOUND RESTRICTIONS

GROUNDWATER MOUNDING FROM "EPA - LAND TREATMENT OF MUNICIPAL WASTEWATER"
MOUND IS DETERMINED GRAPHICALLY IN ATTACHED FIGURES

W/(4@T)^.5
W = BASIN WIDTH
T = LENGTH OF WASTEWATER APPLICATION, DAYS TO STEADY STATE
T = (d/((K*(H/d))/V))
@ = KD/V
K = HYDRAULIC CONDUCTIVITY
D = AQUIFER THICKNESS
V = SPECIFIC YIELD = 0.25 FOR FINE SAND

CELL	W	T	K	D	V	W/(4@T)^.5
A	150	21.43	75	30	0.25	0.17
B	150	9.52	75	30	0.25	0.26
C	150	11.11	75	30	0.25	0.24
D*	100	15.69	75	30	0.25	0.13
E	100	21.93	75	30	0.25	0.11
F	100	21.93	75	30	0.25	0.11

* CELL WIDTH REDUCED FROM MAXIMUM FOR ACTUAL DESIGN DUE TO PHYSICAL SPACE LIMITATIONS

RT
R = I/V
I = INFILTRATION RATE - FT/DAY
hO/RT = VALUE BASED ON GRAPHICAL SOLUTION ON ATTACHED SHEETS
hO = GROUNDWATER MOUND

CELL	T	I	V	RT	hO/RT	hO	MAX GW EL	MAX MOUND
A	21.43	0.70	0.25	60.00	0.08	4.80	1587	8
B	9.52	1.05	0.25	40.00	0.13	5.20	1586	9
C	11.11	0.90	0.25	40.00	0.14	5.60	1585	8
D	15.69	0.96	0.25	60.24	0.08	4.82	1582	7
E	21.93	**0.79**	0.25	69.30	0.10	6.93	1583	7
F	21.93	**0.79**	0.25	69.30	0.10	6.93	1583	7

CELL	GPD	GPM	LENGTH	WIDTH	AREA ACRES
A	196350	136	250	150	0.86
B	294525	205	250	150	0.86
C	302940	210	300	150	1.03
D	214583	149	300	100	0.69
E	431372	300	730	100	1.68
F	443190	308	750	100	1.72
TOTAL	1,882,959	1308			6.84

Note the hydraulic application rates (I) for cells E and F were reduced from Step 1 to 0.79 ft/day
This was done to keep the calculated mound height less than the maximum mound allowed.

Prepared by: PAK
Checked by: TWS

MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]

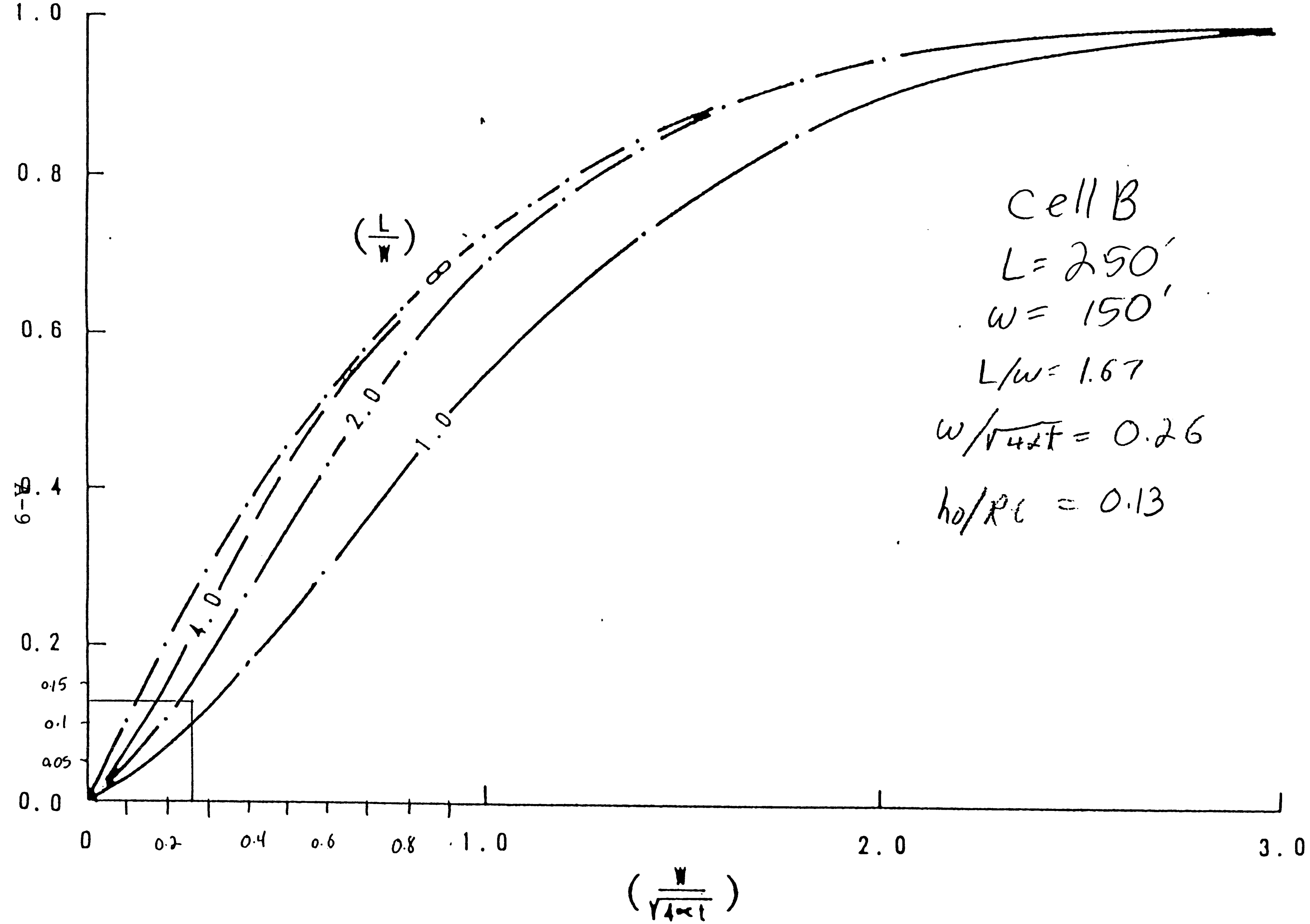

FIGURE 5-9

OUNDING CURVE FOR CENTER OF A RECTANGULAR RECHARGE AREA AT DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

MOUNDING CURVE FOR CENTER OF A SQUARE
RECHARGE AREA [24]

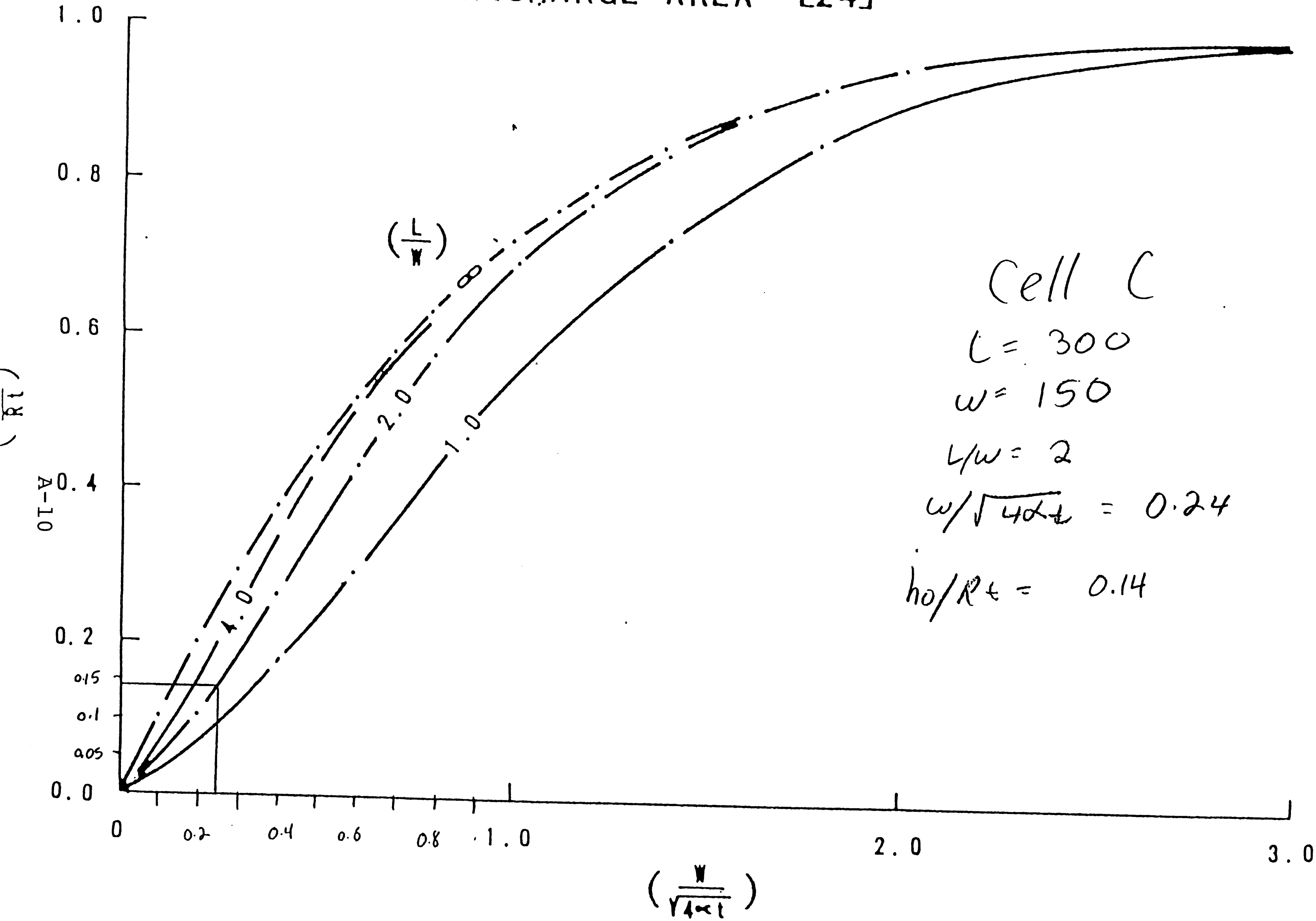

FIGURE 5-9
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DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]

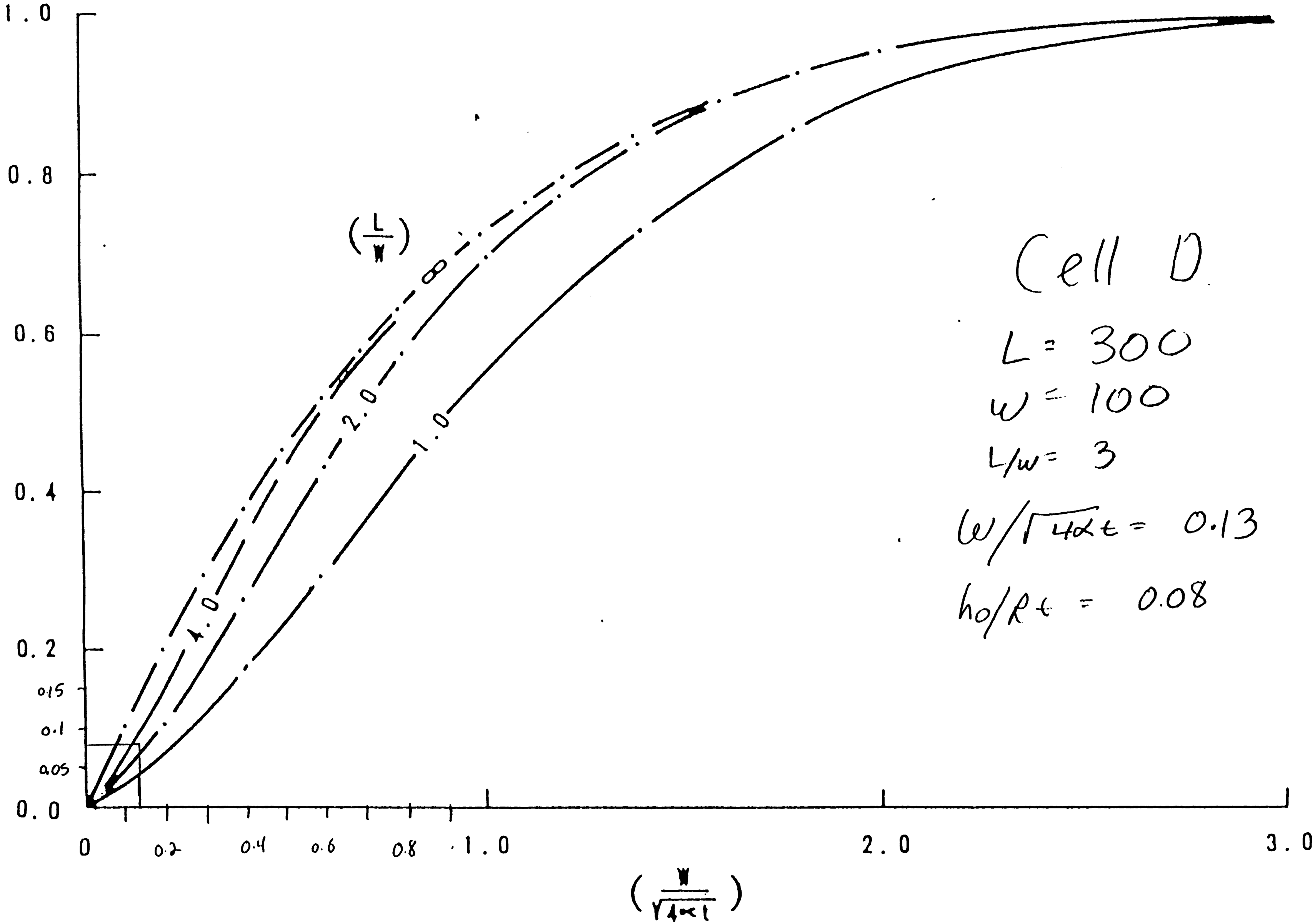

FIGURE 5-9

MOUNDING CURVE FOR CENTER OF A RECTANGULAR RECHARGE AREA AT DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

... FOR CENTER OF A SQUARE RECHARGE AREA [24]

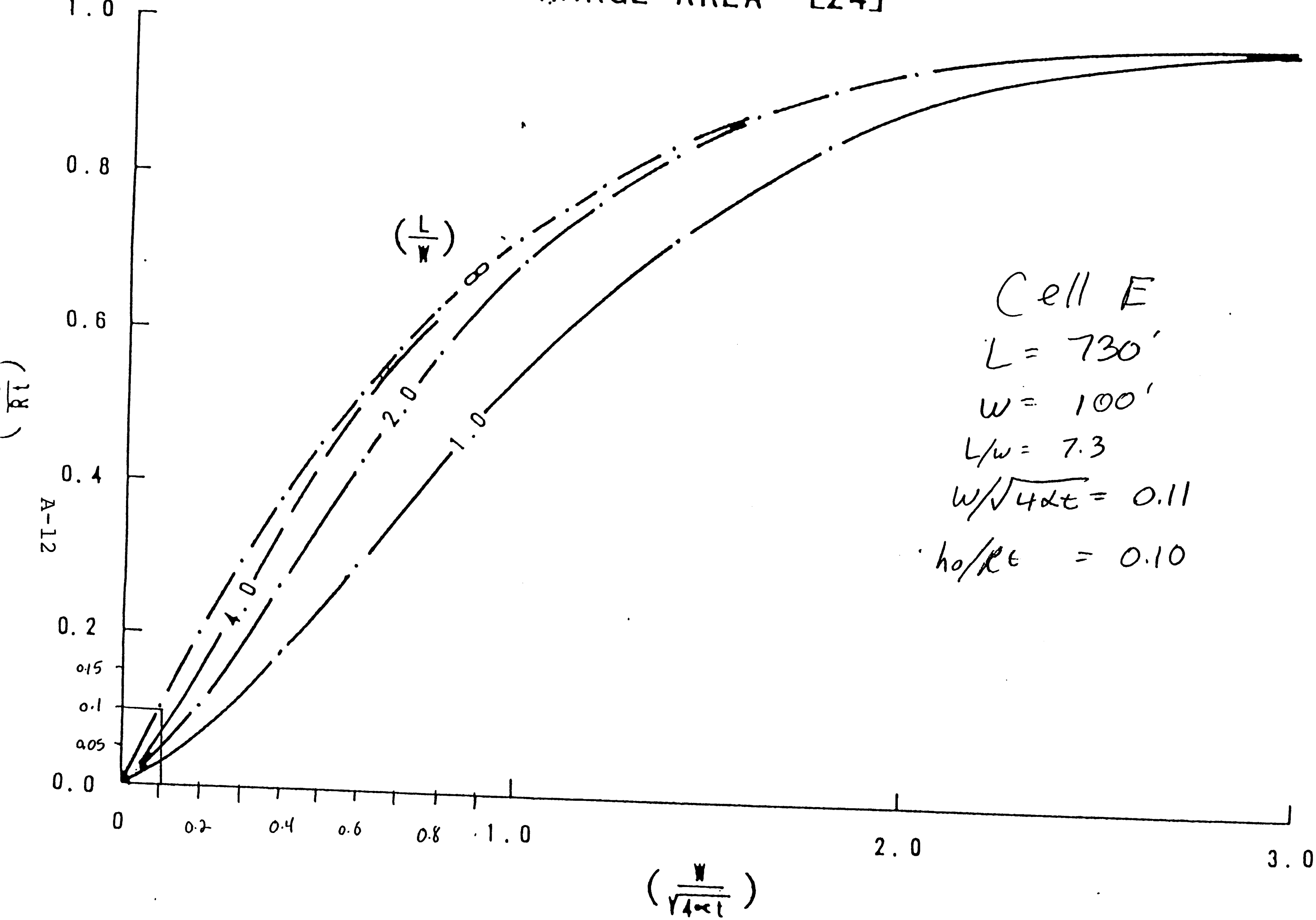

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FIGURE 5-9

MOUNDING CURVE FOR CENTER OF A RECTANGULAR RECHARGE AREA AT DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]

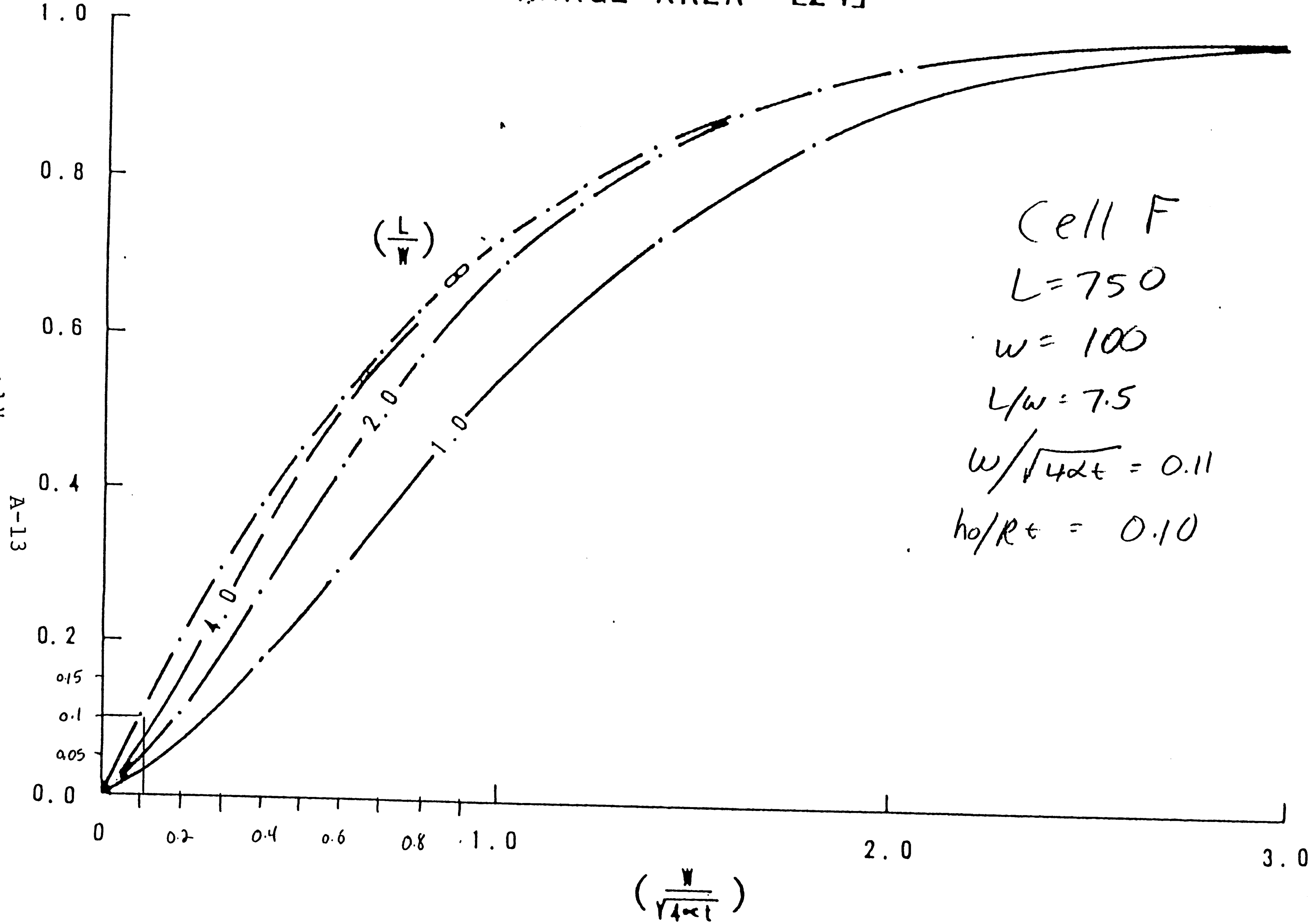

FIGURE 5-9

MOUNDING CURVE FOR CENTER OF A RECTANGULAR RECHARGE AREA AT DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

Hydraulic Capacity Calculations (Pipe Elevation 2 ft Above Maximum Groundwater Mound and 45 ft Thick Aquifer)

CRANDON PROJECT
SOIL ABSORPTION SYSTEM DESIGN

RUDLOFF PROPERTY - AREA H

AQUIFER DEPTH (D) = 45 FEET

DESIGN STEP 1 - CALCULATE MAXIMUM CELL WIDTH

PIPE ELEVATION IS 2 FEET ABOVE MAXIMUM GW MOUND; D = 45 FEET

W = ((K*D*H)/(d*L))
WHERE:

K = HYDRAULIC CONDUCTIVITY - FT/DAY
D = AQUIFER DEPTH, FT
PIPE ELEVATION = APPLICATION POINT FOR WASTEWATER DISPOSAL
DISCHARGE ELEVATION = MAXIMUM WATER LEVEL IN DISCHARGE WETLAND
MAXIMUM GROUNDWATER MOUND ELEV. = PIPE ELEVATION - 2 FEET
H = VERTICAL GROUNDWATER ELEV.; MAX MOUND - DISCHARGE ELEV., FT
d = DISTANCE FROM CENTER OF APPLICATION SYSTEM TO DISCHARGE POINT, FT
L = HYDRAULIC APPLICATION RATE, FT/DAY
W = ABSORPTION POND MAX WIDTH, FT

CELL	A	B	C	D	E	F
K	75	75	75	75	75	75
D	45	45	45	45	45	45
PIPE ELEV.	1597	1597	1595	1591	1592	1592
DISCHARGE ELEV.	1581	1581	1581	1580.5	1580.5	1580.5
MAX MOUND ELEV.	1595	1595	1593	1589	1590	1590
H	14	14	12	8.5	9.5	9.5
d	300	200	200	200	250	250
L	1.05	1.57	1.35	1.43	1.28	1.28
W	150	150	150	100	100	100
LENGTH	250	250	300	300	730	750

Prepared by: PAK
Checked by: TWS

DESIGN STEP 2 - CALCULATE SIZE AND APPLICATION RATE TO MEET MAXIMUM MOUND RESTRICTIONS

GROUNDWATER MOUNDING FROM "EPA - LAND TREATMENT OF MUNICIPAL WASTEWATER" MOUND IS DETERMINED GRAPHICALLY IN ATTACHED FIGURES

W/(4@T)^.5
W = BASIN WIDTH
T = LENGTH OF WASTEWATER APPLICATION, DAYS TO STEADY STATE
T = (d/((K*(H/d))/V))
@ = KD/V
K = HYDRAULIC CONDUCTIVITY
D = AQUIFER THICKNESS
V = SPECIFIC YIELD = 0.25 FOR FINE SAND

CELL	W	T	K	D	V	W/(4@T)^.5
A	150	21.43	75	45	0.25	0.14
B	150	9.52	75	45	0.25	0.21
C	150	11.11	75	45	0.25	0.19
D*	100	15.69	75	45	0.25	0.11
E	100	21.93	75	45	0.25	0.09
F	100	21.93	75	45	0.25	0.09

* CELL WIDTH REDUCED FROM MAXIMUM FOR ACTUAL DESIGN DUE TO PHYSICAL SPACE LIMITATIONS

RT
R = I/V
I = INFILTRATION RATE
hO/RT = VALUE BASED ON GRAPHICAL SOLUTION ON ATTACHED SHEETS
hO = GROUNDWATER MOUND

CELL	T	I	V	RT	hO/RT	hO	MAX GW EL	MAX MOUND
A	21.43	1.05	0.25	90.00	0.06	5.40	1587	8
B	9.52	1.57	0.25	59.81	0.10	5.98	1586	9
C	11.11	1.35	0.25	60.00	0.11	6.60	1585	8
D	15.69	1.43	0.25	89.73	0.07	6.28	1582	7
E	21.93	**0.99**	0.25	86.84	0.08	6.95	1583	7
F	21.93	**0.99**	0.25	86.84	0.08	6.95	1583	7

CELL	GPD	GPM	LENGTH	WIDTH	AREA ACRES
A	294525	205	250	150	0.86
B	441788	307	250	150	0.86
C	454410	316	300	150	1.03
D	321874	224	300	100	0.69
E	541635	376	730	100	1.68
F	556475	386	750	100	1.72
TOTAL	2,610,706	1814			6.84

Note the hydraulic application rates (I) for Cells E and F were reduced from Step 1 to 0.99 ft/day
This was done to keep the calculated mound height less than the maximum allowable mound.

Prepared by: PAK
Checked by: TWS

MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]

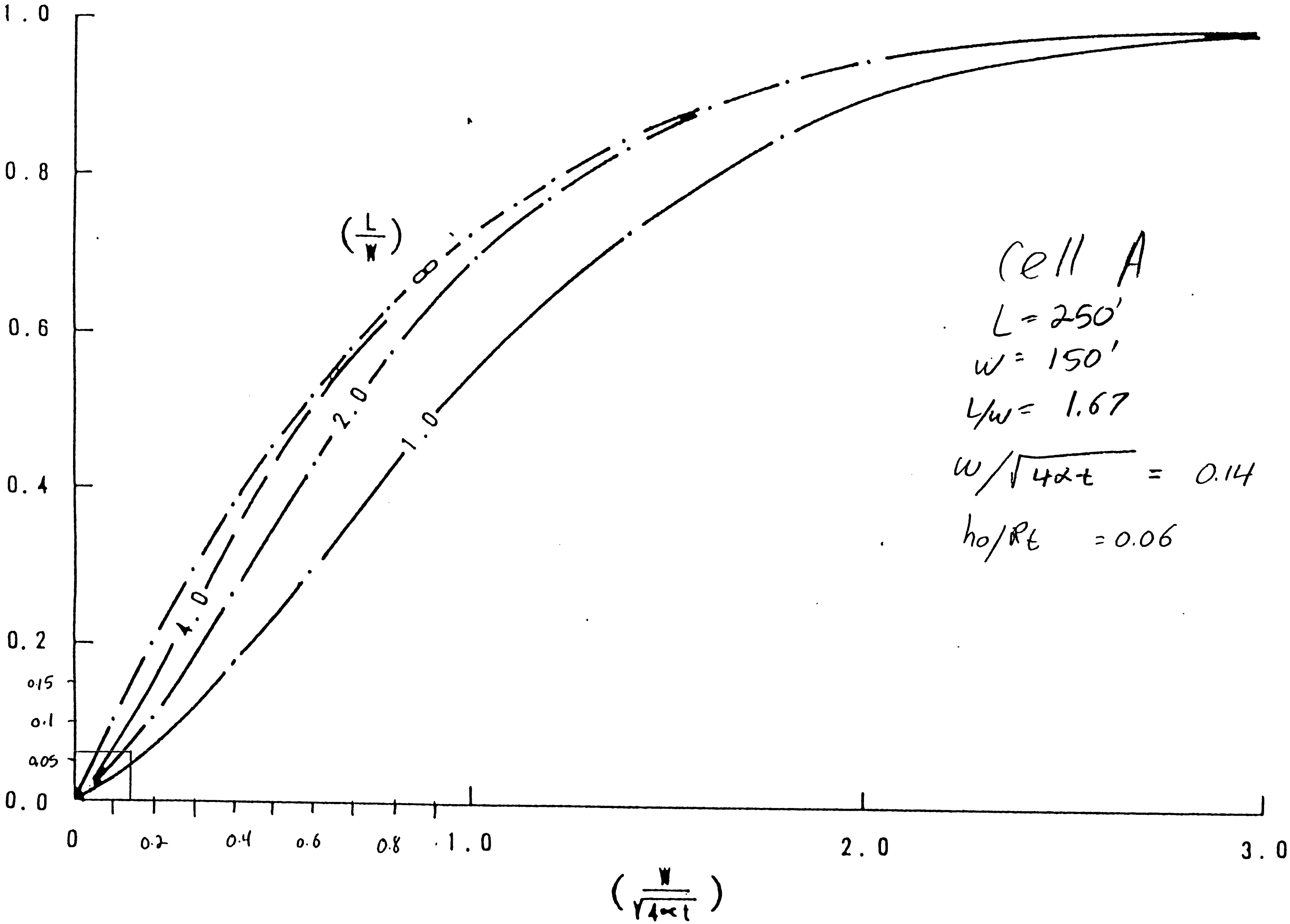

FIGURE 5-9

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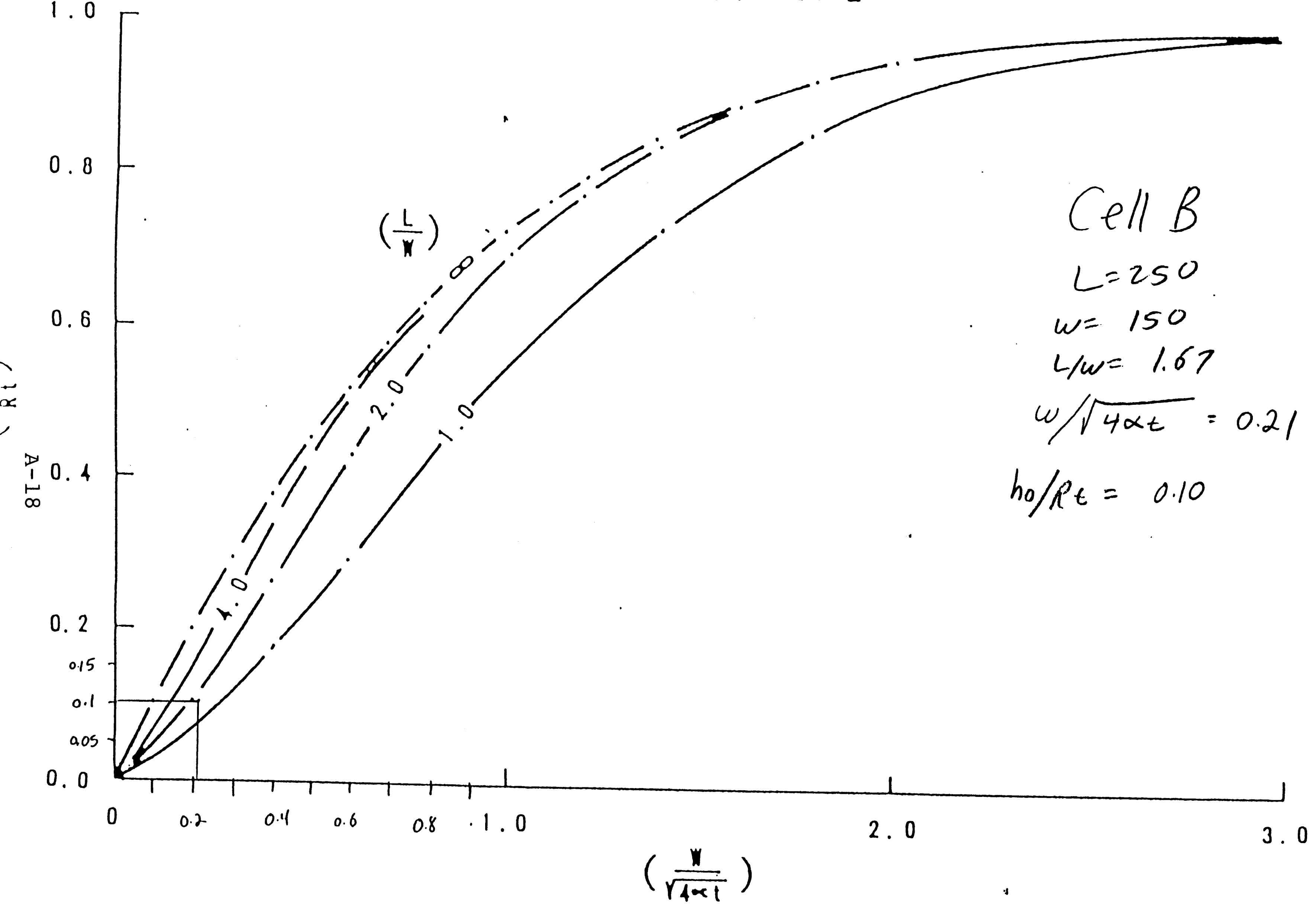
MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]



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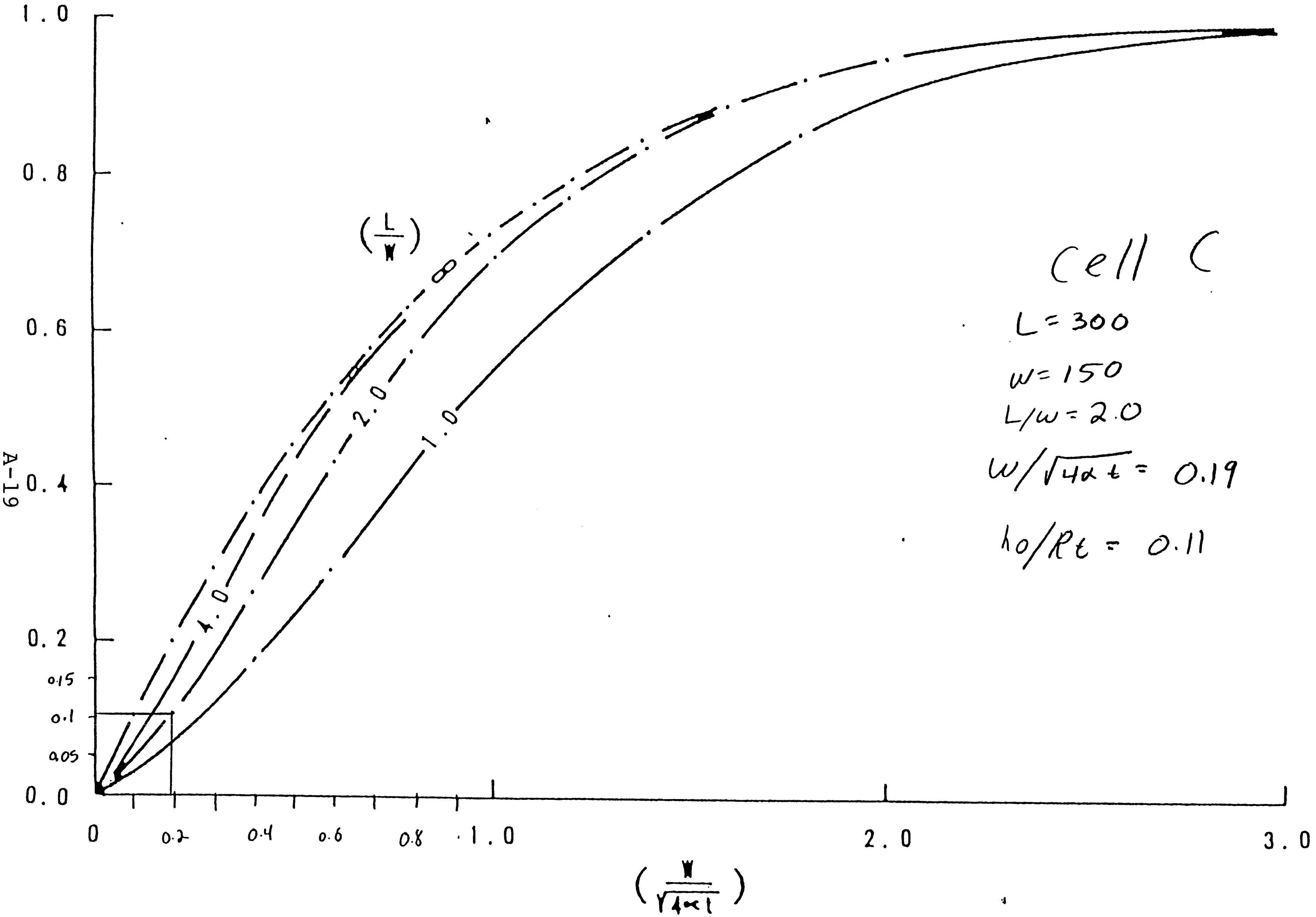
MOUNDING CURVE FOR CENTER OF A SQUARE RECHARGE AREA [24]



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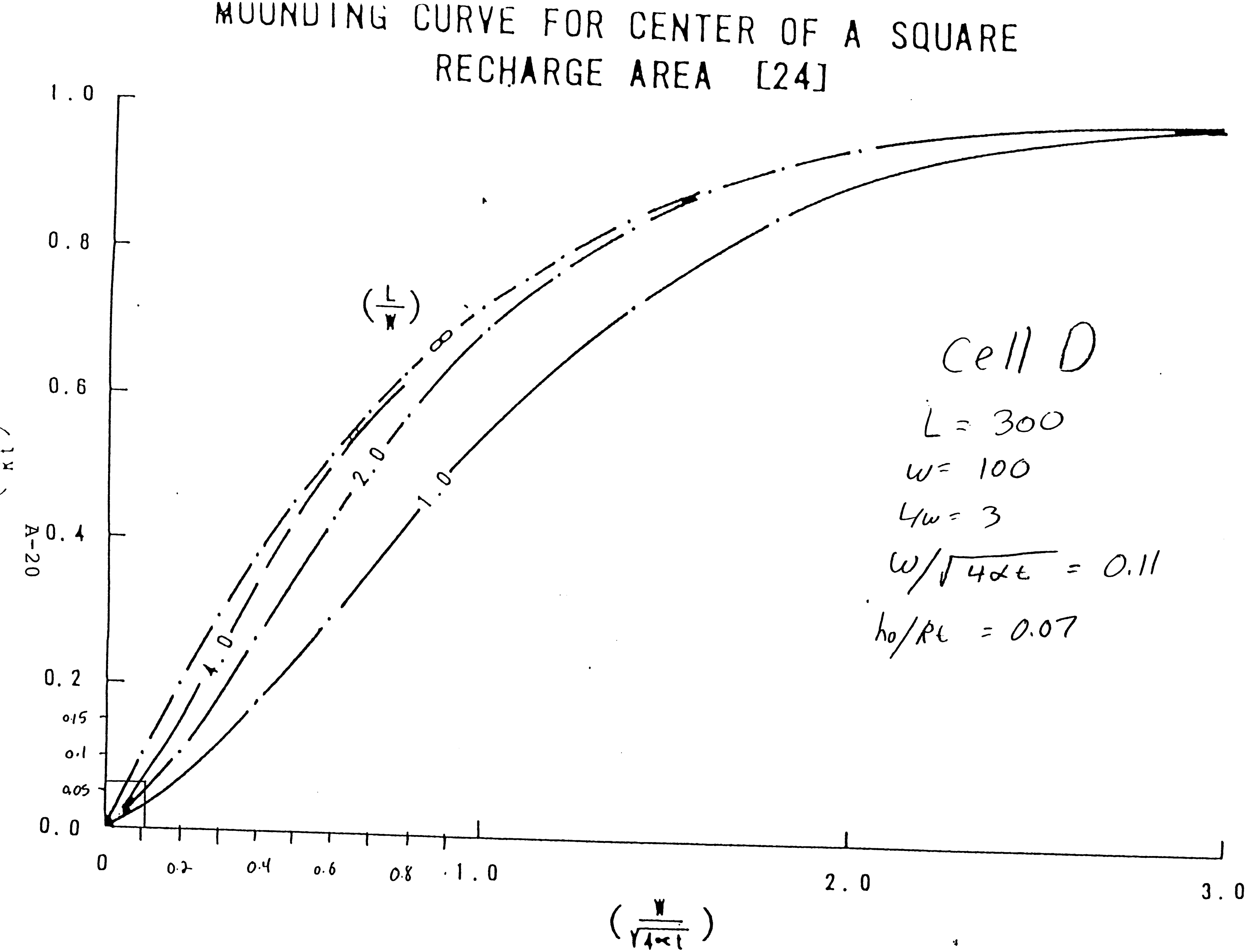

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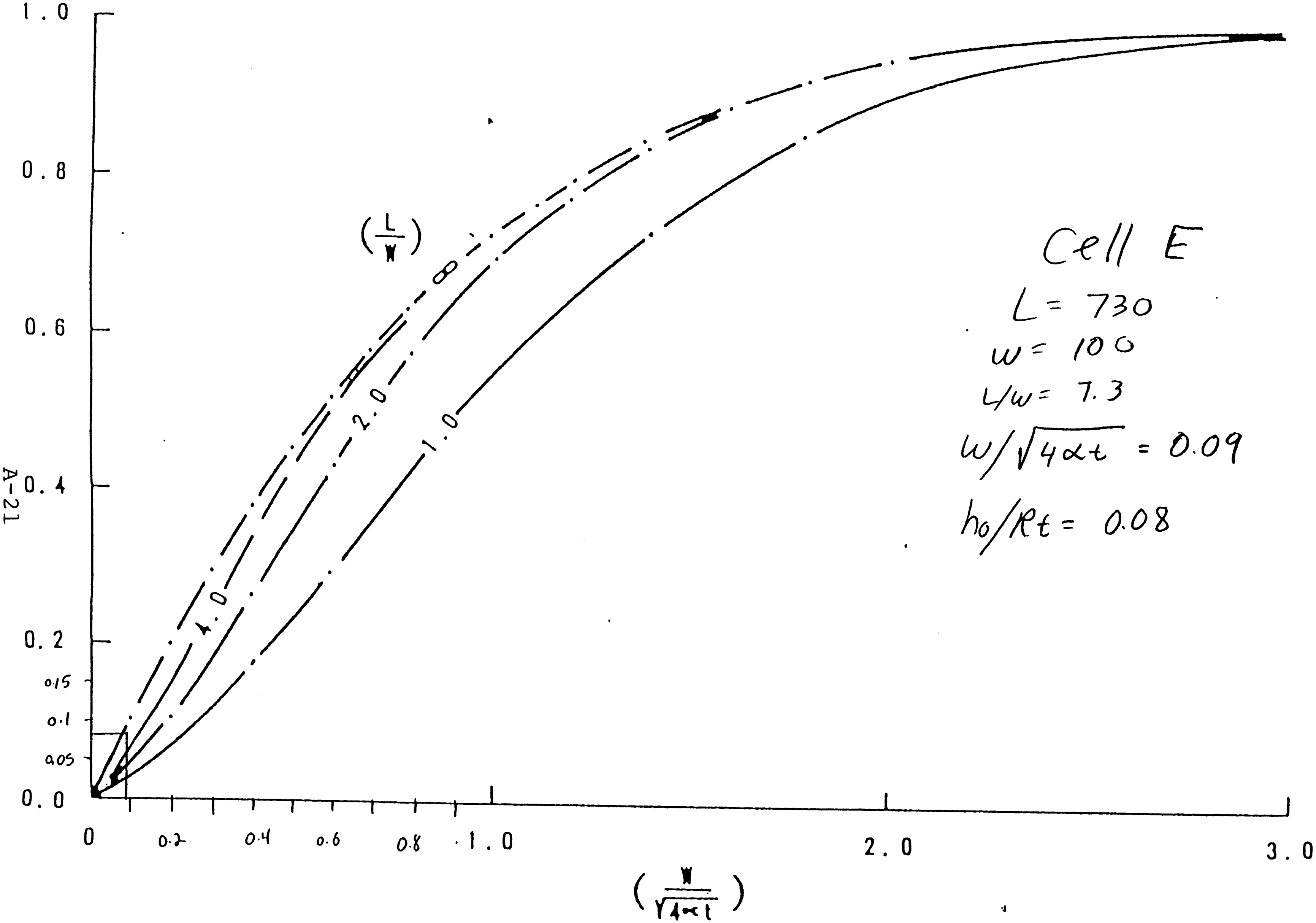

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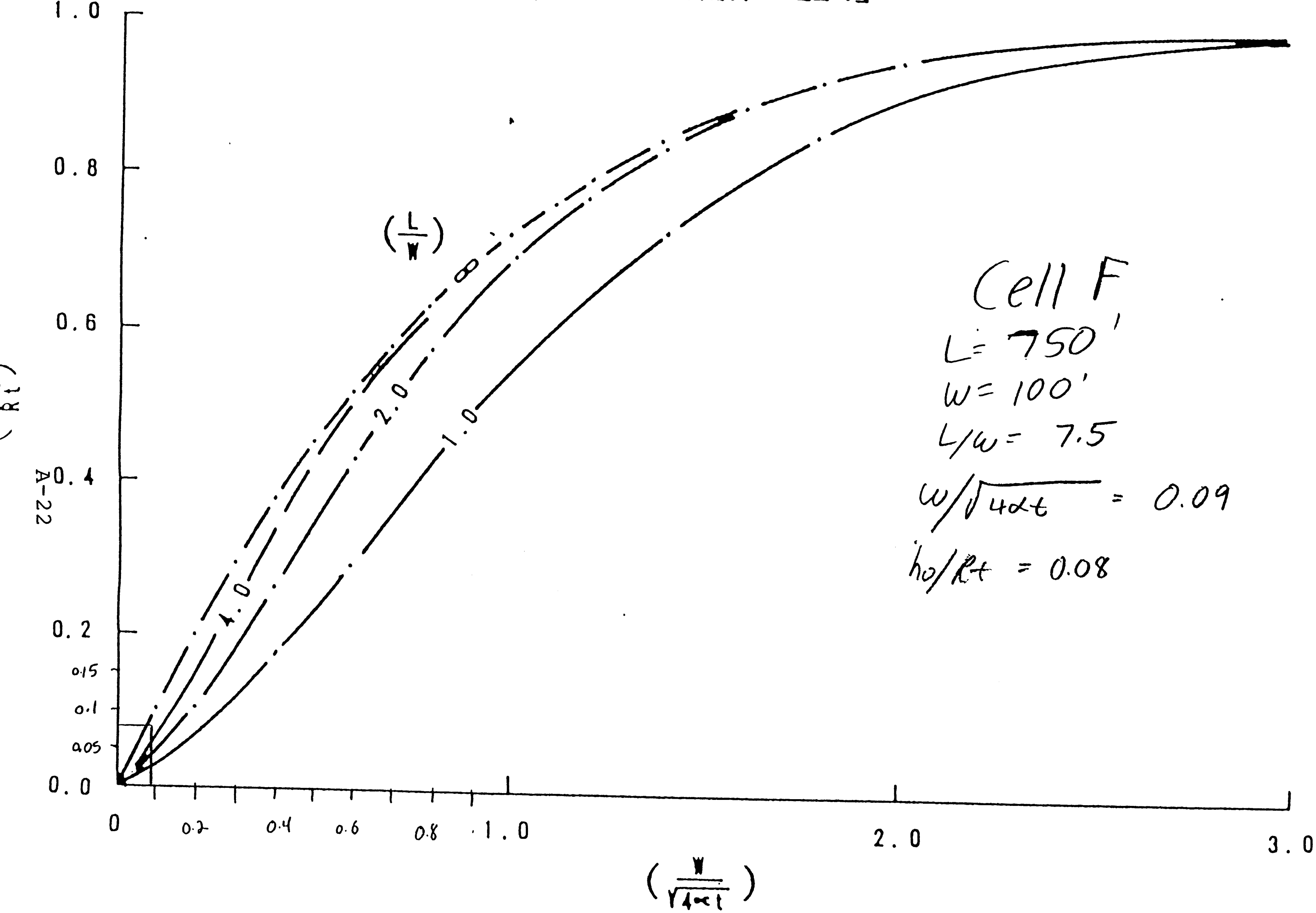


A-22

FIGURE 5-9

MOUNDING CURVE FOR CENTER OF A RECTANGULAR RECHARGE AREA AT DIFFERENT RATIOS OF LENGTH (L) TO WIDTH (W) [24]

Soil Absorption System Bed Design for Discharge Rate of 1,532 gpm

SOIL ABSORPTION BED DESIGN

ASSUME CELLS A - F OPERATE INDEPENDANTLY

ASSUME "EVEN DISTRIBUTION" OVER EACH CELL TO BE DONE WITH LATERALS SPACED 20 FEET APART AND WITH DISCHARGE ORIFICES SPACED 5 FEET APART.

FLOW RATE PER HOLE = 1.28 GPM BASED ON 1/4" ORIFICE AND A PRESSURE OF 3 FEET AT EACH ORIFICE

TO PROVIDE EVEN DISTRIBUTION, THE HEAD LOSS ACROSS THE LATERAL MUST BE <10% OF THE HEAD AT EACH ORIFICE

GIVEN THE ABOVE CONDITIONS, THE MAXIMUM LATERAL LENGTH IS

1.5 INCH PIPE = 1.59" ID	75 FEET (15 ORIFICES)
2 INCH PIPE = 2.047" ID	90 FEET (18 ORIFICES)
2.5 INCH PIPE = 2.469 ID	

CELL A

MAXIMUM AVERAGE FLOW RATE =	169	GPM
DIMENSIONS =	250	LONG
	150	WIDE
LATERAL LENGTH	30	FEET
LATERAL DIAMETER	1.59	INCHES
LATERAL LENGTH	32.5	FEET
LATERAL DIAMETER	2.047	INCHES
LATERALS PER ROW	4	
ROWS OF LATERALS	8	
TOTAL 1.5 INCH LATERAL LENGTH =	960	FEET
TOTAL 2 INCH LATERAL LENGTH =	1040	
NUMBER OF HOLES =	400	
FLOW RATE PER HOLE =	1.28	GPM
TOTAL FLOW RATE =	512	GPM
LATERAL VOLUME - 1.5 INCH =	0.01378166	CU FT/FT
LATERAL VOLUME - 2 INCH =	0.02284246	CU FT/FT
TOTAL LATERAL VOLUME =	277	GALLONS
DOSE VOLUME =	3320	GALLONS

CELL B

MAXIMUM AVERAGE FLOW RATE =	260	GPM
DIMENSIONS =	250	LONG
	150	WIDE
LATERAL LENGTH	30	FEET
LATERAL DIAMETER	1.59	INCHES
LATERAL LENGTH	32.5	FEET
LATERAL DIAMETER	2.047	INCHES
LATERALS PER ROW	4	
ROWS OF LATERALS	8	
TOTAL 1.5 INCH LATERAL LENGTH =	960	FEET
TOTAL 2 INCH LATERAL LENGTH =	1040	
NUMBER OF HOLES =	400	
FLOW RATE PER HOLE =	1.28	GPM
TOTAL FLOW RATE =	512	GPM
LATERAL VOLUME - 1.5 INCH =	0.01378166	CU FT/FT
LATERAL VOLUME - 2 INCH =	0.02284246	CU FT/FT
TOTAL LATERAL VOLUME =	277	GALLONS
DOSE VOLUME =	3320	GALLONS

Prepared by: PAK
Checked by: TWS

CELL C

MAXIMUM AVERAGE FLOW RATE =	260 GPM
DIMENSIONS =	300 LONG
	150 WIDE
LATERAL LENGTH	30 FEET
LATERAL DIAMETER	1.59 INCHES
LATERAL LENGTH	45 FEET
LATERAL DIAMETER	2.047 INCHES
LATERALS PER ROW	4
ROWS OF LATERALS	8
TOTAL 1.5 INCH LATERAL LENGTH =	960 FEET
TOTAL 2 INCH LATERAL LENGTH =	1440
NUMBER OF HOLES =	480
FLOW RATE PER HOLE =	1.28 GPM
TOTAL FLOW RATE =	614 GPM
LATERAL VOLUME - 1.5 INCH =	0.01378166 CU FT/FT
LATERAL VOLUME - 2 INCH =	0.02284246 CU FT/FT
TOTAL LATERAL VOLUME =	345 GALLONS
DOSE VOLUME =	4140 GALLONS

CELL D

MAXIMUM AVERAGE FLOW RATE =	184 GPM
DIMENSIONS =	300 LONG
	100 WIDE
LATERAL LENGTH	30 FEET
LATERAL DIAMETER	1.59 INCHES
LATERAL LENGTH	45 FEET
LATERAL DIAMETER	2.047 INCHES
LATERALS PER ROW	4
ROWS OF LATERALS	5
TOTAL 1.5 INCH LATERAL LENGTH =	600 FEET
TOTAL 2 INCH LATERAL LENGTH =	900
NUMBER OF HOLES =	300
FLOW RATE PER HOLE =	1.28 GPM
TOTAL FLOW RATE =	384 GPM
LATERAL VOLUME - 1.5 INCH =	0.01378166 CU FT/FT
LATERAL VOLUME - 2 INCH =	0.02284246 CU FT/FT
TOTAL LATERAL VOLUME =	216 GALLONS
DOSE VOLUME =	2588 GALLONS

CELL E

MAXIMUM AVERAGE FLOW RATE =	322 GPM
DIMENSIONS =	730 LONG
	100 WIDE
LATERAL LENGTH	70 FEET
LATERAL DIAMETER	2.047 INCHES
LATERAL LENGTH	20 FEET
LATERAL DIAMETER	2.469 INCHES
LATERALS PER ROW	8
ROWS OF LATERALS	5
TOTAL 1.5 INCH LATERAL LENGTH =	2800 FEET
TOTAL 2 INCH LATERAL LENGTH =	800 FEET
NUMBER OF HOLES =	720
FLOW RATE PER HOLE =	1.28 GPM
TOTAL FLOW RATE =	922 GPM
LATERAL VOLUME - 1.5 INCH =	0.02284246 CU FT/FT
LATERAL VOLUME - 2 INCH =	0.03323145 CU FT/FT
TOTAL LATERAL VOLUME =	677 GALLONS
DOSE VOLUME =	8127 GALLONS

Prepared by: PAK
Prepared by: TWS

CELL F

MAXIMUM AVERAGE FLOW RATE =	337 GPM
DIMENSIONS =	750 LONG
	100 WIDE
LATERAL LENGTH	70 FEET
LATERAL DIAMETER	2.047 INCHES
LATERAL LENGTH	20 FEET
LATERAL DIAMETER	2.469 INCHES
LATERALS PER ROW	8
ROWS OF LATERALS	5
TOTAL 1.5 INCH LATERAL LENGTH =	2800 FEET
TOTAL 2 INCH LATERAL LENGTH =	800 FEET
NUMBER OF HOLES =	720
FLOW RATE PER HOLE =	1.28 GPM
TOTAL FLOW RATE =	922 GPM
LATERAL VOLUME - 1.5 INCH =	0.02284246 CU FT/FT
LATERAL VOLUME - 2 INCH =	0.03323145 CU FT/FT
TOTAL LATERAL VOLUME =	677 GALLONS
DOSE VOLUME =	8127 GALLONS

CELL	FLOW - GPM	% TOT.	DOSE VOL GAL.	% TOT.
A	169	11%	3320	11%
B	260	17%	3320	11%
C	260	17%	4140	14%
D	184	12%	2588	9%
E	322	21%	8127	27%
F	337	22%	8127	27%
	1532	100%	29622	

Prepared by: PAK
Checked by: TWS

Hydraulic Capacity Calculations

Gravel Bedding Selection Analysis

Foth & Van Dyke
consultants · engineers · scientists

Client:	NMC	Scope I.D.:	00C002
Project:	SAS Contingency	Page:	1 of 2
Prepared By:	Phil Korth	Date:	2-4-00
Checked By:	REM	Date:	2-8-00

Calculate minimum permeability coefficient for gravel layer

Assume the following:

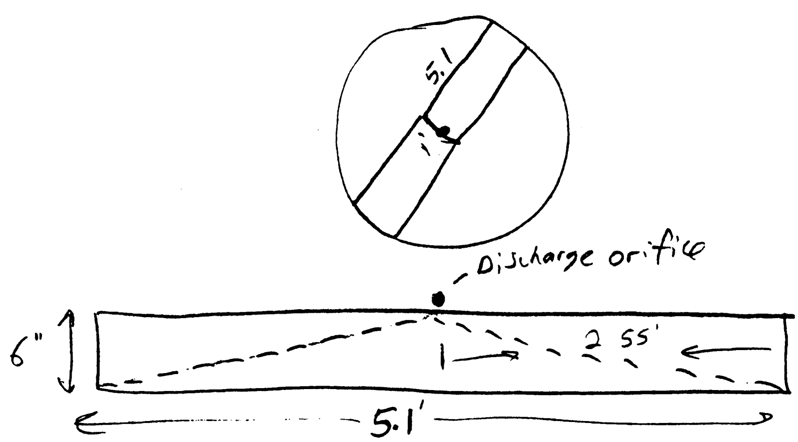
1. Outwash sand has vertical permeability of 8'/day
   = 8 ft³/ft²/day
   = 60 gallons/ft²/day

2. Maximum flow from 1- ¼" orifice = 1,228 gpd
   (1500 gpm - 460,500 gpd for cell F / 375 orifices = 1228 gpd/orifice) = .0019 cfs

3. Minimum area needed to drain 1228 gpd
   = 1228/60 = 20.5 ft²

4. Diameter of 20.5 ft² = 5.1 feet

5. Assume a block of gravel layer 1 foot wide × 5.1 feet long = 5.1 feet square



Slope = 0.5'/2.55' = 0.196

Flow = 5.1 ft²/20.5 ft² = .25   .0019 cfs × .25 = .00047 cfs
= Area of 25%

6. Equation: $Q = K i A$

where Q = flow cfs
K = coefficient of permeability, ft/sec
i = slope, ft/ft
A = Area of bed

we know Q = .00047
i = 0.196
A = 0.5 x 50% since bed is only half full of water.
= .25'

$$K = \frac{Q}{iA} = \frac{.00047}{(.196)(.25)} = .0096 \text{ ft/sec}$$
$$= 0.58 \text{ ft/min}$$

7. Gravel layer must have
$K > 0.58$ ft/min

From design Figure 1, D_{10} mm size should be greater than 1.0 mm

From Figure 6, drainage material curves 1, 2, 3, 6, 7, 8 all meet criteria

A 3/4" clean stone should be adequate

Reference- Equation in Step 6
Figure 1
Figure 6
All from "Soil Mechanics" Dept. of Navy 1982

2. DRAINAGE BLANKET. Figure 5 shows typical filter and drainage blanket installations.

a. Permeability. Figure 6 (Reference 5, Subsurface Drainage of Highways, by Barber) gives typical coefficients of permeability for clean, coarse-grained drainage material and the effect of various percentages of fines on permeability. Mixtures of about equal parts gravel with medium to coarse sand have a permeability of approximately 1 fpm. Single sized, clean gravel has a permeability exceeding 50 fpm. For approximate relationship of permeability versus effective grain size D_{10}, see Figure 1, Chapter 3.

b. Drainage Capacity. Estimate the quantity of water which can be transmitted by a drainage blanket as follows:

$$q = k \cdot i \cdot A$$

where q = quantity of flow, ft^3/sec

k = permeability coefficient, ft/sec

i = average gradient in flow direction, ft/ft

A = cross sectional area of blanket, ft^2

The gradient is limited by uplift pressures that may be tolerated at the point farthest from the outlet of the drainage blanket. Increase gradients and flow capacity of the blanket by providing closer spacing of drain pipes within the blanket.

(1) Pressure Relief. See bottom panel of Figure 7 (Reference 6, Seepage Requirements of Filters and Pervious Bases, by Cedergren) for combinations of drain pipe spacing, drainage course thickness, and permeability required for control of flow upward from an underlying aquifer under an average vertical gradient of 0.4.

(2) Rate of Drainage. See the top panel of Figure 7 (Reference 5) for time rate of drainage of water from a saturated base course beneath a pavement. Effective porosity is the volume of drainable water in a unit volume of soil. It ranges from 25 percent for a uniform material such as medium to coarse sand, to 15 percent for a broadly graded sand-gravel mixture.

c. Drainage Blanket Design. The following guidelines should be followed:

(1) Gradation. Design in accordance with Figure 4.

(2) Thickness. Beneath, structures require a minimum of 12 inches for each layer with a minimum thickness of 24 inches overall. If placed on wet, yielding, uneven excavation surface and subject to construction operation and traffic, minimum thickness shall be 36 inches overall.

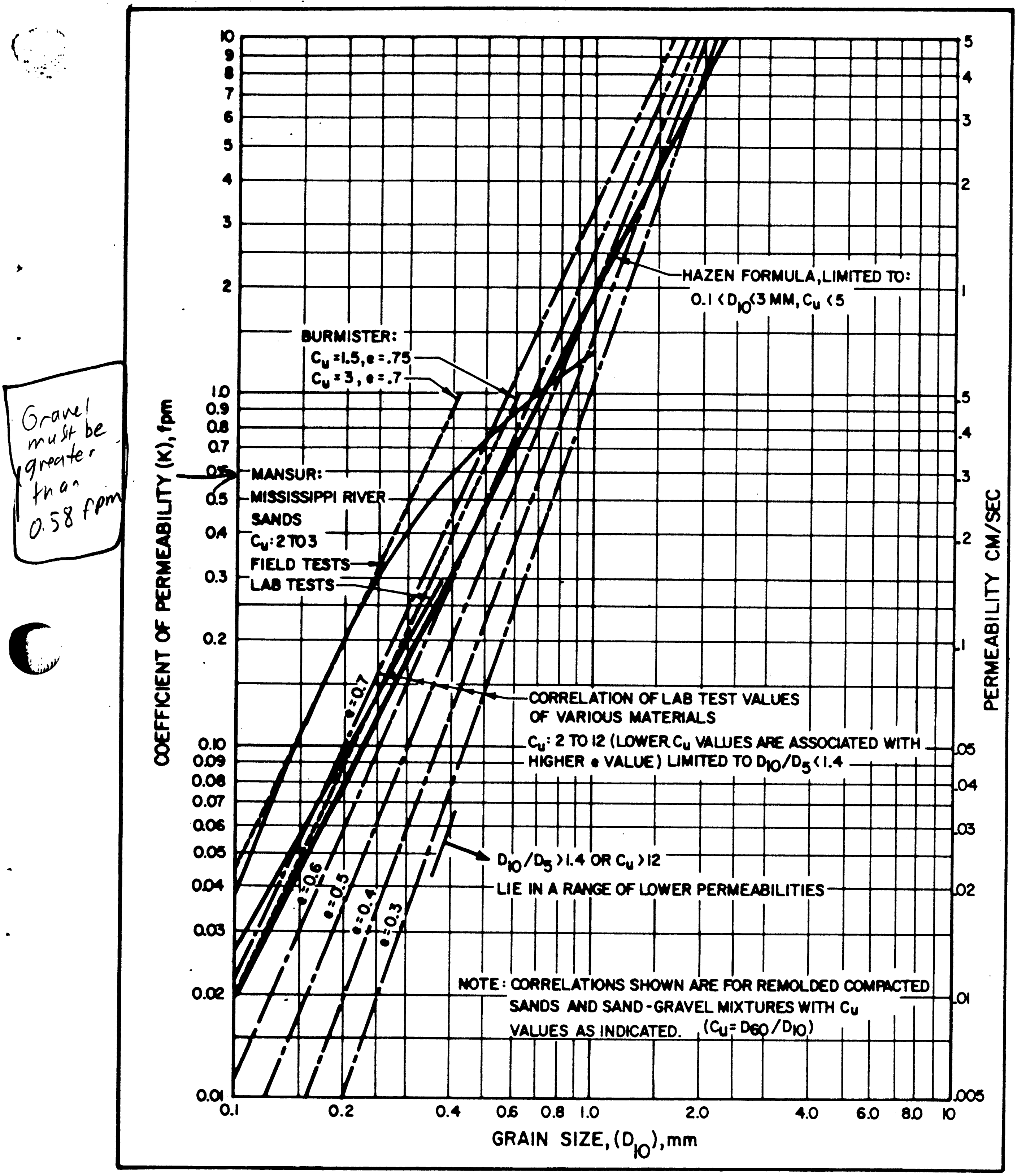
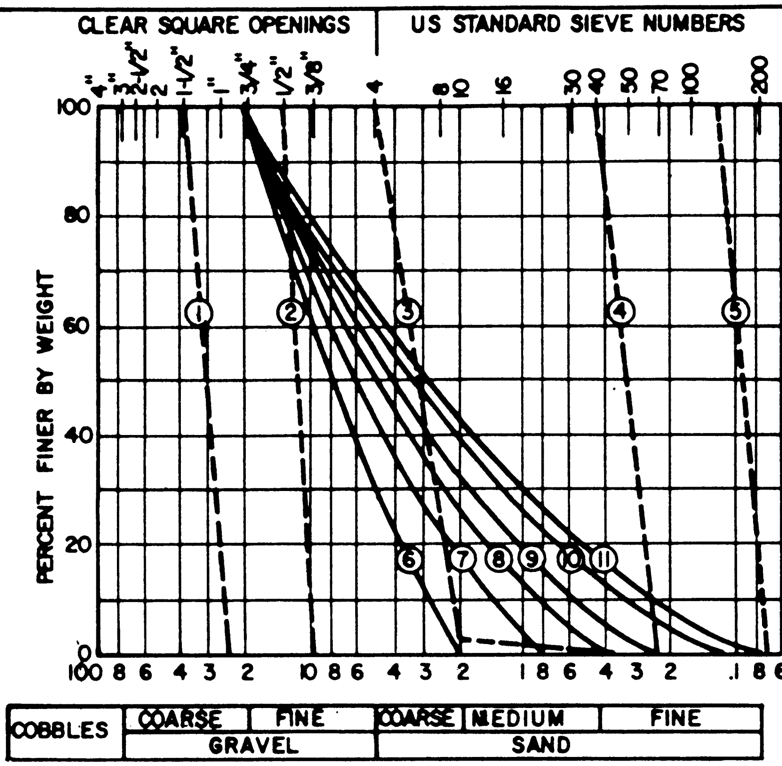
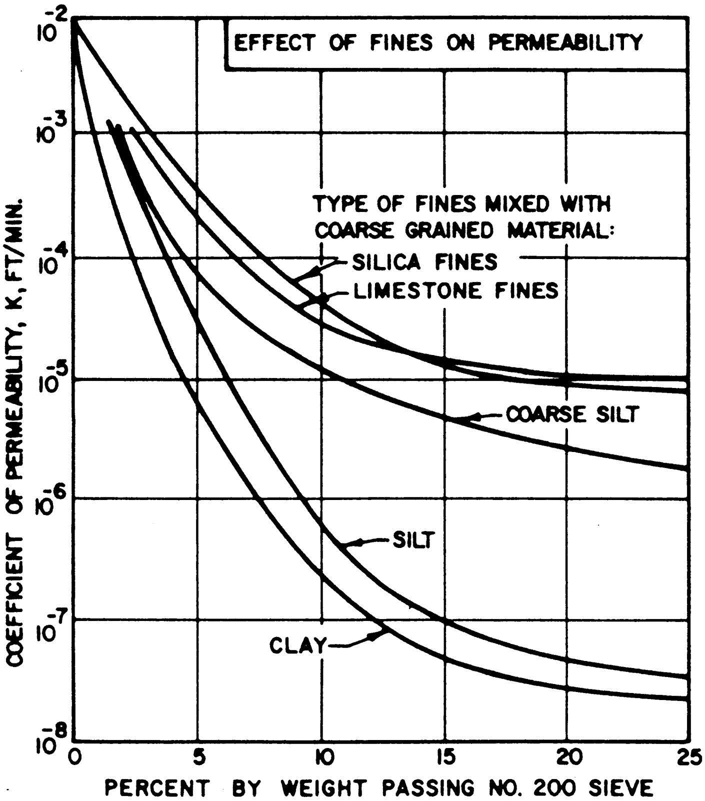


FIGURE 1
Permeability of Sands and Sand-Gravel Mixtures



COEFFICIENT OF PERMEABILITY FOR CLEAN COARSE-GRAINED DRAINAGE MATERIAL

CURVE	K, FT/MIN.	
1	73.7	3.7×10^1
2	56.9	2.8×10^1
3	5.41	
4	0.13	
5	0.01	
6	2.08	1×10^1
7	1.81	
8	0.70	
9	0.22	
10	0.08	
11	0.01	



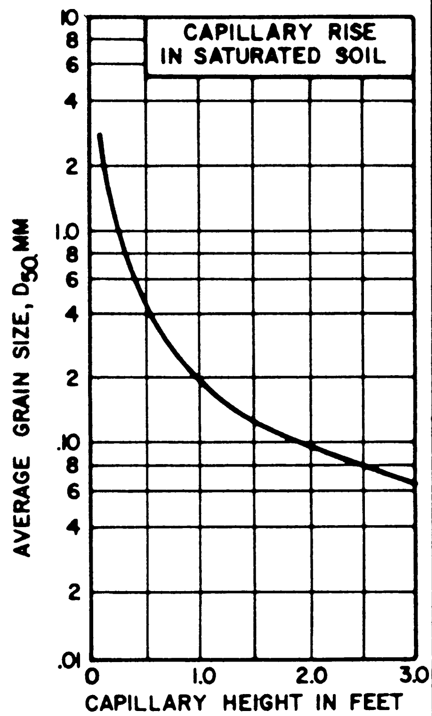


FIGURE 6
Permeability and Capillarity of Drainage Materials

Appendix B

Technical Memorandum - Contingency Flow Hydrologic Analysis

Foth & Van Dyke
Memorandum

March 9, 2000

TO: Steve Donohue, Foth & Van Dyke

CC: Gordon Reid, Nicolet Minerals Company
Jerry Sevick, Foth & Van Dyke
Denis Roznowski, Foth & Van Dyke
Master File

FR: Michael D. Liebman, P.E., Senior Water Resource Consultant MDL
Steve R. Birr, Water Resource Engineer SRB

RE: Crandon Project - Soil Absorption System Wetland Surface Water Flow Impacts
Inflow Contingency Plan - "Worst Case" Conditions

Introduction

The purpose of this memo is to evaluate changes to surface water hydrology or flood plain elevations in wetland Z16 and Swamp Creek that would result from discharge of additional treated wastewater to the project's Area H soil absorption system (SAS). The additional flow would result if mine inflow is greater than projected in the project's *Preliminary Engineering Report for the Crandon Project Soil Absorption System* (SAS PER) (Foth & Van Dyke, 1998).

With the discharge from the SAS ultimately entering nearby down-gradient wetlands (i.e., Z16 north, central, and south), the impact of the SAS on the hydrology and surface water flow of these wetlands was investigated. Also investigated was the potential impact of the SAS discharge on the floodplain of Swamp Creek.

This memorandum summarizes the results of the surface water flow evaluation. For conservativeness, discharge flows from the Area H SAS were assumed to be three times as much as the original flows (3 x 714 gpm = 2,142 gpm) used in the November 23, 1998 memorandum found in Appendix K of the SAS PER. This is a very conservative assumption, since the project's *Mine Water Contingency Plan* (MWCP) (Foth & Van Dyke, 2000) assumes a contingency flow to the SAS of only 1,532 gpm. For these analyses, it was assumed that positive surface water drainage is maintained across the dredged canal and snowmobile trail which bisect the wetland body.

Methods of Analyses

The method used to determine the impact of additional water discharges near wetlands was based on Haestad Methods "TR-55" hydrologic software, Mannings formula hydraulics across the wetland valley (Haestad Methods "Flowmaster" software), and culvert nomographs where culverts control hydraulics (adding weir flow where appropriate). An existing or base condition flow was established from various storm events for the drainage areas tributary to each of the Z16 north, central, and south wetlands. A depth of flow associated with each storm event peak flow was calculated using the appropriate hydraulics. Then, the rates of peak discharge from the SAS were added to the existing base flows calculated for each wetland, and new depths of flow were determined. Any increase in depth caused by the SAS discharge could then be determined. To be conservative, it was assumed that all of the additional water from each SAS cell discharges to the respective wetland. Also, to assure conservativeness, discharge from cell D was added to the flow analysis for both the central and north wetlands, as it is not certain to which wetland the flow will pass.

Possible effects of the SAS discharge on the Swamp Creek floodplain were evaluated by calculating the 100-year flood flow of Swamp Creek in this area, and determining the associated depth of flow in Swamp Creek. The flood flow was found using the NRC5 TR20 hydrological program. The associated depth of flow was calculated from a Mannings analysis ("Flowmaster") across a section of the Swamp Creek floodplain. By calculating this depth of flow with the SAS discharge added to the 100-year flow, an increase caused by the SAS could be determined.

Figure 1 illustrates the location of the SAS cells and their peak discharge rates, as used in this analysis, the location of the conveyance culverts, the location of the wetlands that were investigated, the location of the Swamp Creek cross-section used in the floodplain evaluation, and the location of the other valley sections used in the analysis.

Results

Table 1 illustrates the results of the hydraulic analysis. The north wetland (Area A) has a drainage area of 42.7 acres. To be conservative in terms of base flow comparison, adding SAS flow to a lower base flow would show a greater impact. The approximate 9-acre wetland area that may intermittently drain through the 8 inch agricultural drain tile was excluded from the area which drains to the Z16 north wetland. Using this assumption, Area A contributes from approximately 2 cfs to approximately 24 cfs of runoff for the 2 through 100-year recurrence interval storm events under existing conditions. Surface water flow for this wetland is controlled by the railroad culvert east of the north wetland. A base flow of 0.5 cfs was estimated from field observation of depth and velocity at this culvert. This brings total existing flow to a range from 0.5 cfs (no storm event) to 24.5 cfs (100-year storm event). The "worst case" peak discharge from the northern SAS cells (A, B, C, and D) is a combined 1,128 gal/min (2.51 cfs) of discharge, which would give a total flow rate of 5.01 cfs to 27.01 cfs of flow at the wetland for the 2 through 100-year storm events, respectively. Depth of increase of the wetland water surface upgradient from this culvert varied from 0.05 to 0.07 ft for these storm events if the SAS were operating, and showed 0.10 ft increase with SAS flow only.

Similarly, the central wetland (Area B1) has a drainage area of 50 acres. The range of flows for the 2-100 year storm events was found to be approximately 2 cfs to approximately 28 cfs. With the 735 gal/min (1.65 cfs) "worst case" discharge from SAS cells D and E, the depth increase based on the wetland valley hydraulics if the SAS were operating was from 0.01 to 0.02 ft during the storm events, and 0.05 ft with no storm flow.

The south wetland (south of Keith Siding Road - Area B2) has a drainage area of 105 acres, which includes the 50-acre central area (Area B1). This drainage area includes a dredge canal and pond, and contributes from approximately 4 cfs to approximately 53 cfs of runoff for the 2 through 100-year storm events under existing conditions. The proposed "worst case" peak discharge from the southern SAS cell (F) is 513 gal/min (1.14 cfs) of flow which, when combined with the central area flows (735 gal/min or 1.65 cfs), gives a total flow rate of 6.79 cfs to 55.79 cfs of flow in the wetland when the SAS is operating. For the south wetland, both valley section and culvert crossing hydraulics were evaluated. Depth increase of the wetland water surface at these locations varied from 0.00 ft to 0.03 ft if the SAS were operating during storm events, and 1.1 ft with only SAS flow. The capacity of the 12-inch CMCP that conveys runoff across Keith Siding Road is only 4 cfs. Therefore, weir flow overtopping of the road is predicted for larger storm events under base flow conditions without the SAS in operation. The weir length was estimated to be 680 ft, which is the width of the wetland at the road (refer to Figure 1). Attachment 1 documents modeling data used in obtaining results illustrated in Table 1.

For Swamp Creek, floodplain impacts were found to be non-existent because the increase in flow caused by the "worst case" SAS discharge changed the 100-year flow from 277.5 cfs to 280.01 cfs, which makes no detectable difference to the hydraulics of the floodplain.

Conclusion

The wetland surface water elevation increase as a direct result of the SAS discharge was shown to be minimal. Under predicted base flow conditions during operation of the SAS (i.e., with no surface water component and 100% discharge from the SAS), 0.10 ft is predicted to occur in the upstream vicinity of the culvert in the north wetland. At the south wetland, the post-SAS operation base flow predicts an increase in elevation of 1.1 ft in the upgradient vicinity of the culvert which crosses Keith Siding Road.

Referring to the valley section modeling, the biggest increase to onsite flow depths during storm events is 0.03 ft. Any increase at the northern wetland will occur only at the upstream side of the railroad culvert due to the presence of the railroad berm. Similarly, increases at the south area culvert will only extend upstream on NMC property due to the road grade of Keith Siding Road.

No changes to the Swamp Creek floodplain elevation occur as a result of operation of the SAS. As such, floodplain issues are not relevant.

Based upon the information contained on Table 1 and the documentation in Attachment 1, it has been shown that the amount of flow that will be discharged from the SAS is very small in comparison to the existing drainage basin runoff to each wetland and, as a result, surface water hydrology in the Z16 wetland and Swamp Creek will not be significantly affected by SAS operation.

References

Foth & Van Dyke, 1998. *Preliminary Engineering Report for the Crandon Project Soil Absorption System*. Appendix D to the *Preliminary Engineering Report for Wastewater Treatment Facilities for the Crandon Project*. November 1998.

Foth & Van Dyke, 2000. *Crandon Project - Addendum No. 1 to the Preliminary Engineering Report for Wastewater Treatment Facilities (Mine Water Management Contingency Plan)*. March 2000.

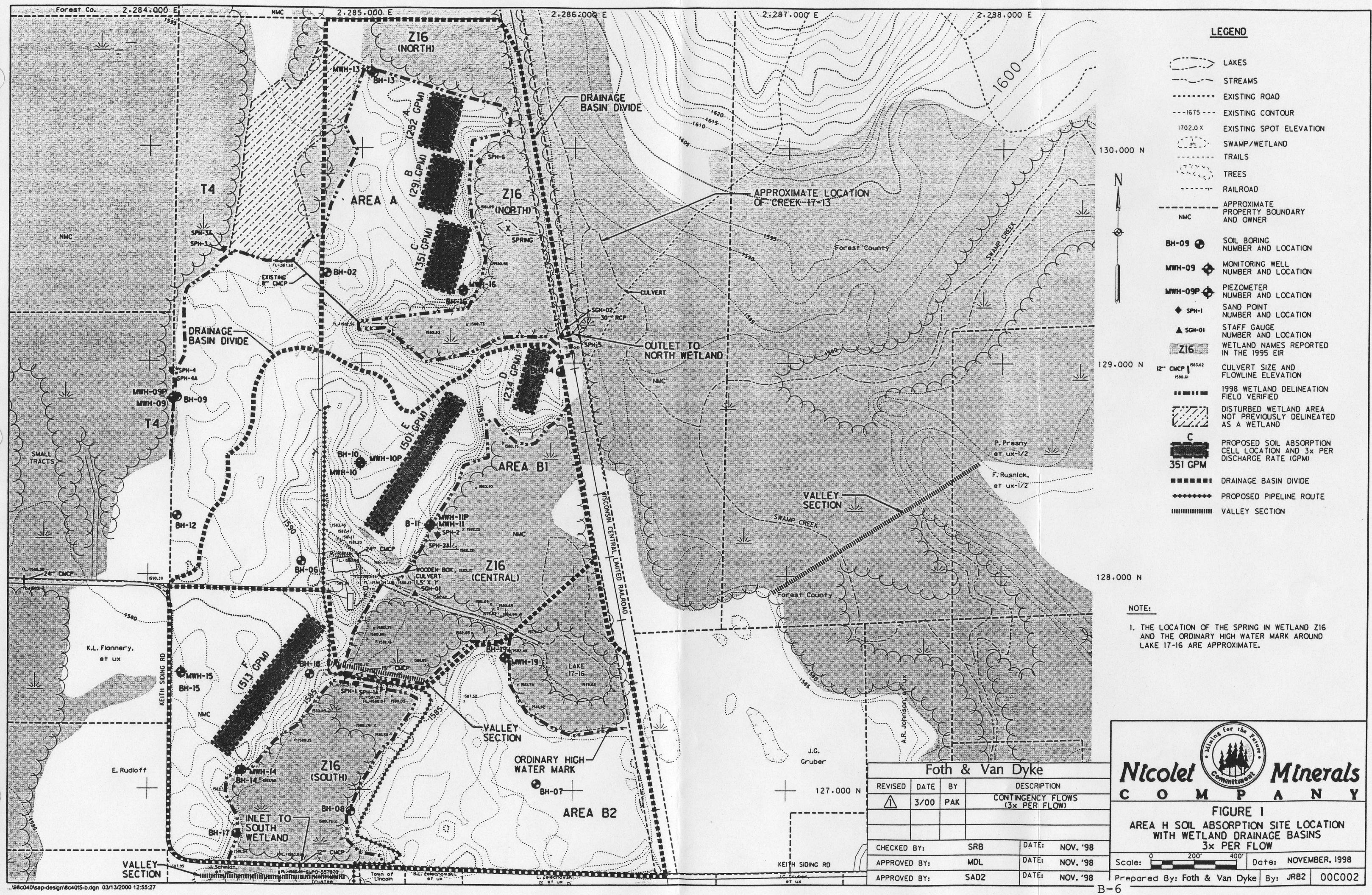
Table 1

Crandon Project - Soil Absorption Sites Surface Flow Analyses

		Storm Events					
North Area (42.7 Acres)		0 Year (Base)	2 Year (2.4")	5 Year (3.1")	10 Year (3.6")	25 Year (4.2")	100 Year (5.0")
Railroad Crossing	Existing Flow	0.5 cfs	2.5 cfs	4.5 cfs	11.5 cfs	16.5 cfs	24.5 cfs
	Existing Depth	1.15 ft	1.30 ft	1.40 ft	1.75 ft	2.10 ft	2.71 ft
	Existing + SAS Flow	3.01 cfs	5.01 cfs	7.01 cfs	14.01 cfs	19.01 cfs	27.01 cfs
	Existing + SAS Depth	1.25 ft	1.35 ft	1.45 ft	1.80 ft	2.17 ft	2.78 ft
Central Area (50 Acres)							
Valley Section[1]	Existing Flow	0 cfs	2 cfs	5 cfs	13 cfs	19 cfs	28 cfs
	Existing Depth	0 ft	0.06 ft	0.10 ft	0.18 ft	0.23 ft	0.29 ft
	Existing + SAS Flow	1.65 cfs	3.65 cfs	6.65 cfs	14.65 cfs	20.65 cfs	29.65 cfs
	Existing + SAS Depth	0.05 ft	0.08 ft	0.12 ft	0.19 ft	0.24 ft	0.30 ft
South Area (105 Acres)							
Valley Section[1]	Existing Flow	0 cfs	4 cfs	9 cfs	24 cfs	36 cfs	53 cfs
	Existing Depth	0 ft	0.06 ft	0.10 ft	0.18 ft	0.24 ft	0.30 ft
	Existing + SAS Flow	2.79 cfs	6.79 cfs	11.79 cfs	26.79 cfs	38.79 ft	55.79 cfs
	Existing + SAS Depth	0.05 ft	0.09 ft	0.12 ft	0.20 ft	0.25 ft	0.31 ft
Keith Siding Road	Existing Flow	0 cfs	4 cfs	9 cfs	24 cfs	36 cfs	53 cfs
	Existing Depth	0 ft	1.70 ft	1.72 ft	1.75 ft	1.76 ft	1.78 ft
	Existing + SAS Flow	2.79 cfs	6.79 cfs	11.79 cfs	26.79 cfs	38.79 cfs	55.79 cfs
	Existing + SAS Depth	1.10 ft	1.71 ft	1.72 ft	1.75 ft	1.77 ft	1.79 ft
Swamp Creek (15 Sq Mi)							
Floodplain Section	Existing Flow	3.5 cfs	66.5 cfs	114.5 cfs	153.5 cfs	204.5 cfs	277.5 cfs
	Existing Depth	0.05 ft	0.31 ft	0.43 ft	0.51 ft	0.61 ft	0.73 ft
	Existing + SAS Flow	6.01 cfs	69.01 cfs	117.01 cfs	156.01 cfs	207.01 cfs	280.01 cfs
	Existing + SAS Depth	0.07 ft	0.32 ft	0.43 ft	0.52 ft	0.61 ft	0.73 ft

[1] Section locations are shown on attached Figure 1.

Prepared by: SRB
Checked by: MDL


LEGEND
LAKES
STREAMS
EXISTING ROAD
---1675--- EXISTING CONTOUR
1702.0 X EXISTING SPOT ELEVATION
SWAMP/WETLAND
TRAILS
TREES
RAILROAD
NMC APPROXIMATE PROPERTY BOUNDARY AND OWNER
BH-09 SOIL BORING NUMBER AND LOCATION
MWH-09 MONITORING WELL NUMBER AND LOCATION
MWH-09P PIEZOMETER NUMBER AND LOCATION
SPH-1 SAND POINT NUMBER AND LOCATION
SGH-01 STAFF GAUGE NUMBER AND LOCATION
Z16 WETLAND NAMES REPORTED IN THE 1995 EIR
12" CMCP CULVERT SIZE AND FLOWLINE ELEVATION
1998 WETLAND DELINEATION FIELD VERIFIED
DISTURBED WETLAND AREA NOT PREVIOUSLY DELINEATED AS A WETLAND
C 351 GPM PROPOSED SOIL ABSORPTION CELL LOCATION AND 3x PER DISCHARGE RATE (GPM)
DRAINAGE BASIN DIVIDE
PROPOSED PIPELINE ROUTE
VALLEY SECTION
NOTE:
1. THE LOCATION OF THE SPRING IN WETLAND Z16 AND THE ORDINARY HIGH WATER MARK AROUND LAKE 17-16 ARE APPROXIMATE.
Z16 (NORTH)
Z16 (CENTRAL)
Z16 (SOUTH)
AREA A
AREA B1
AREA B2
T4
A (252 GPM)
B (291 GPM)
C (351 GPM)
D (234 GPM)
E (501 GPM)
F (513 GPM)
DRAINAGE BASIN DIVIDE
APPROXIMATE LOCATION OF CREEK 17-13
OUTLET TO NORTH WETLAND
INLET TO SOUTH WETLAND
VALLEY SECTION
ORDINARY HIGH WATER MARK
LAKE 17-16
SWAMP CREEK
WISCONSIN CENTRAL LIMITED RAILROAD
KEITH SIDING RD
Forest County
SMALL TRACTS
K.L. Flannery, et ux
E. Rudloff
J.G. Gruber
A.R. Johnson, et ux
P. Presny et ux-1/2
F. Rusniak, et ux-1/2
Town of Lincoln
WOODEN BOX CULVERT
SPRING
CULVERT
1600
2,284,000 E
2,285,000 E
2,286,000 E
2,287,000 E
2,288,000 E
130,000 N
129,000 N
128,000 N
127,000 N
Foth & Van Dyke
REVISED | DATE | BY | DESCRIPTION
1 | 3/00 | PAK | CONTINGENCY FLOWS (3x PER FLOW)
CHECKED BY: SRB DATE: NOV. '98
APPROVED BY: MDL DATE: NOV. '98
APPROVED BY: SAD2 DATE: NOV. '98
Nicolet Minerals COMPANY
Mining for the Future Commitment
FIGURE 1
AREA H SOIL ABSORPTION SITE LOCATION WITH WETLAND DRAINAGE BASINS 3x PER FLOW
Scale: 0 200' 400'
Date: NOVEMBER, 1998
Prepared By: Foth & Van Dyke
By: JRB2
00C002
...\98c040\sap-design\8c40f5-b.dgn 03/13/2000 12:55:27

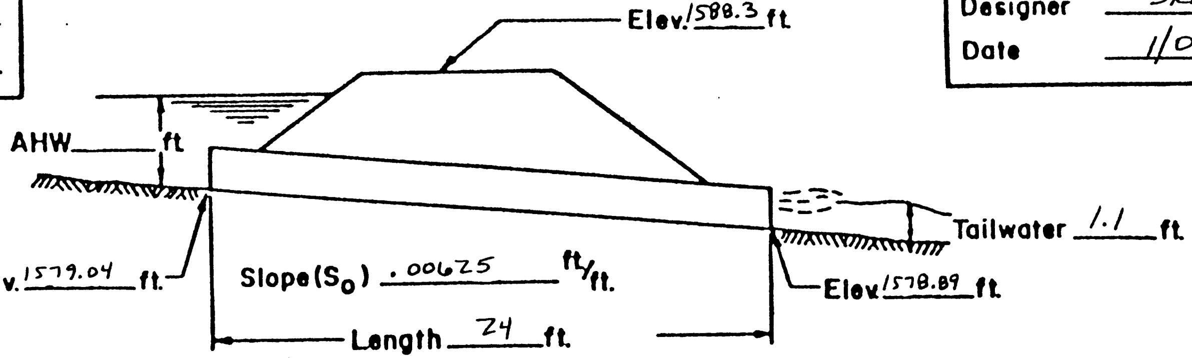
Attachment 1

SAS Hydrology Calculations

HYDROLOGIC AND CHANNEL INFORMATION

Project: NORTH AREA
Culvert Sta.: RAILROAD CULVERT

Designer: SRB
Date: 1/00



Hydrology:

(____ freq.) Q= ________ cfs.

(____ freq.) Q= ________ cfs.

$$\frac{L}{100 S_o} = \frac{24}{100(.00625)} = 38.04$$

Location comments:

Culvert			Q	Capacity Charts HW	Inlet Cont.		Outlet Control							Controling HW	Outlet Velocity	Comments
~~Entrance~~	Material	Size			$\frac{HW}{D}$	HW	K_e	d_c	$\frac{d_c+D}{2}$	h_o*	H	LS_o	**HW			
Base Flow	RCP	30"	.5 / 3.01				.9	.2 / .5	1.35 / 1.5	1.35 / 1.5	<.1	.15	1.15 / 1.25			0.10 INCREASE
2-YR			2.5 / 5.01				.9	.4 / .7	1.4 / 1.6	1.45 / 1.6	<.1	.15	1.3 / 1.35			0.05 INCREASE
5-YR			4.5 / 7.01				.9	.6 / .9	1.55 / 1.7	1.55 / 1.7	<.1	.15	1.4 / 1.45			0.05 INCREASE
10-YR			11.5 / 14.01				.9	1.1 / 1.2	1.8 / 1.85	1.8 / 1.85	.1 / .2	.15	1.75 / 1.80			0.05 INCREASE
25-YR			16.5 / 19.01				.9	1.3 / 1.5	1.9 / 2	1.9 / 2	.35 / .37	.15	2.1 / 2.17			0.07 INCREASE
100-YR			24.5 / 27.01				.9	1.7 / 1.8	2.1 / 2.15	2.1 / 2.15	.76 / .78	.15	2.71 / 2.7			0.07 INCREASE

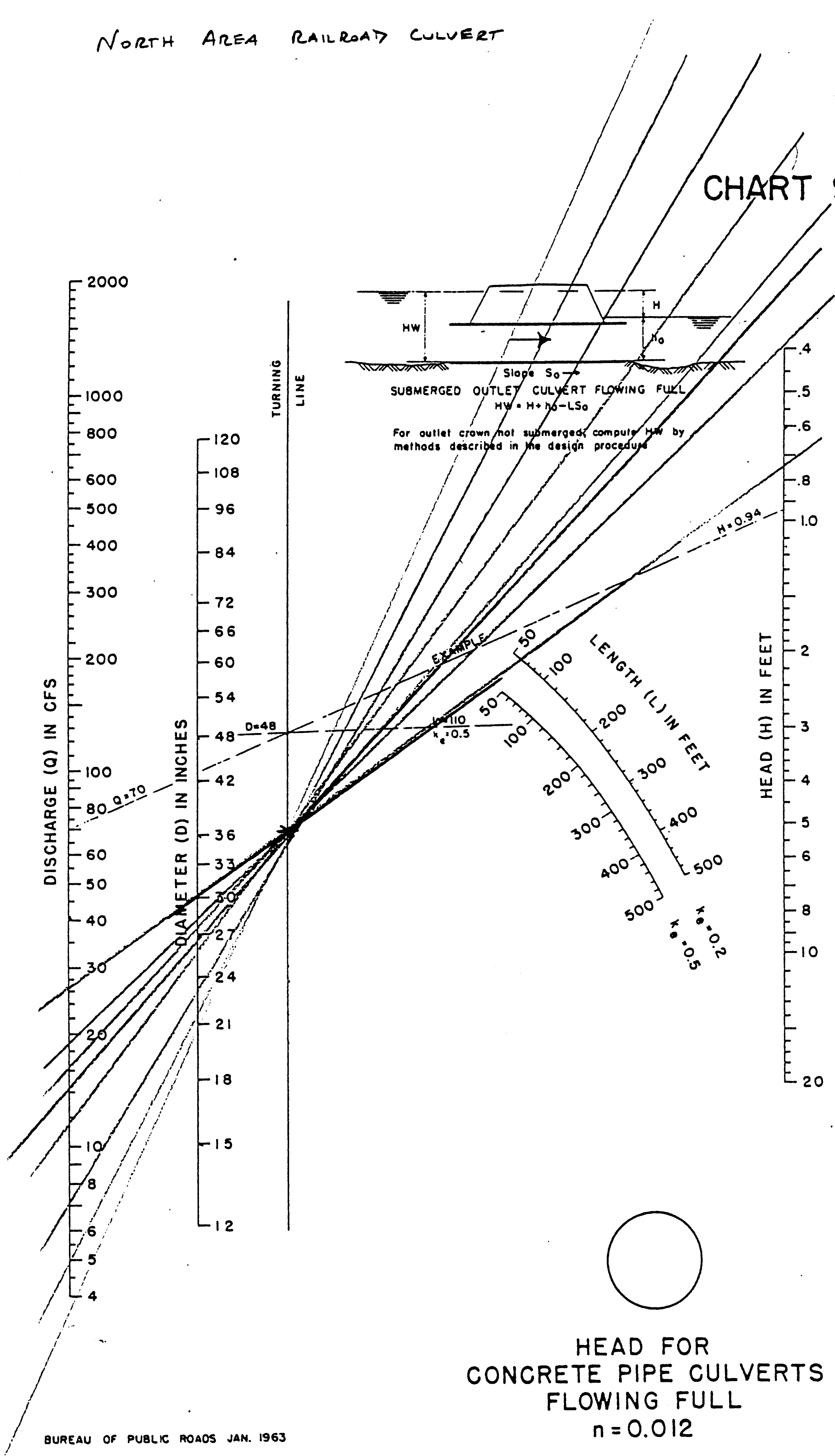
* h_o = The greater of $\frac{d_c+D}{2}$ or TW

** $HW = H + h_o - LS_o$

Summary & Recommendations:

North Area Railroad Culvert

CHART 9



HEAD FOR
CONCRETE PIPE CULVERTS
FLOWING FULL
n = 0.012

BUREAU OF PUBLIC ROADS JAN. 1963

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: CENTRAL VALLEY - BASE CONDITION 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	380.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	1.65 cfs

Computed Results:

Depth............	0.05 ft
Velocity.........	0.08 fps
Flow Area........	20.02 sf
Flow Top Width...	380.84 ft
Wetted Perimeter.	380.85 ft
Critical Depth...	0.01 ft
Critical Slope...	0.4598 ft/ft
Froude Number....	0.06 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: CENTRAL VALLEY - PROPOSED 2-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	380.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	3.65 cfs

Computed Results:

Depth............	0.08 ft
Velocity.........	0.11 fps
Flow Area........	32.25 sf
Flow Top Width...	381.36 ft
Wetted Perimeter.	381.37 ft
Critical Depth...	0.01 ft
Critical Slope...	0.3854 ft/ft
Froude Number....	0.07 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: CENTRAL VALLEY - PROPOSED 5-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	380.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	6.65 cfs

Computed Results:

Depth............	0.12 ft
Velocity.........	0.14 fps
Flow Area........	46.25 sf
Flow Top Width...	381.94 ft
Wetted Perimeter.	381.96 ft
Critical Depth...	0.02 ft
Critical Slope...	0.3373 ft/ft
Froude Number....	0.07 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: CENTRAL VALLEY - PROPOSED 10-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	380.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	14.65 cfs

Computed Results:

Depth............	0.19 ft
Velocity.........	0.20 fps
Flow Area........	74.39 sf
Flow Top Width...	383.12 ft
Wetted Perimeter.	383.14 ft
Critical Depth...	0.04 ft
Critical Slope...	0.2831 ft/ft
Froude Number....	0.08 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: CENTRAL VALLEY - PROPOSED 25-YR   1/00

Solve For Depth

Given Input Data:

Bottom Width.....	380.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	20.65 cfs

Computed Results:

Depth............	0.24 ft
Velocity.........	0.23 fps
Flow Area........	91.47 sf
Flow Top Width...	383.83 ft
Wetted Perimeter.	383.86 ft
Critical Depth...	0.05 ft
Critical Slope...	0.2623 ft/ft
Froude Number....	0.08 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: CENTRAL VALLEY - PROPOSED 100-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	380.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	29.65 cfs

Computed Results:

Depth............	0.30 ft
Velocity.........	0.26 fps
Flow Area........	113.75 sf
Flow Top Width...	384.76 ft
Wetted Perimeter.	384.80 ft
Critical Depth...	0.06 ft
Critical Slope...	0.2421 ft/ft
Froude Number....	0.08 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SOUTH AREA - BASE CONDITION 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	680.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	2.79 cfs

Computed Results:

Depth............	0.05 ft
Velocity.........	0.08 fps
Flow Area........	34.61 sf
Flow Top Width...	680.81 ft
Wetted Perimeter.	680.82 ft
Critical Depth...	0.01 ft
Critical Slope...	0.4656 ft/ft
Froude Number....	0.06 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SOUTH AREA - PROPOSED 2-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	680.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	6.79 cfs

Computed Results:

Depth............	0.09 ft
Velocity.........	0.12 fps
Flow Area........	59.04 sf
Flow Top Width...	681.39 ft
Wetted Perimeter.	681.40 ft
Critical Depth...	0.01 ft
Critical Slope...	0.3821 ft/ft
Froude Number....	0.07 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SOUTH AREA - PROPOSED 5-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	680.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	11.79 cfs

Computed Results:

Depth............	0.12 ft
Velocity.........	0.14 fps
Flow Area........	82.23 sf
Flow Top Width...	681.93 ft
Wetted Perimeter.	681.95 ft
Critical Depth...	0.02 ft
Critical Slope...	0.3380 ft/ft
Froude Number....	0.07 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SOUTH AREA - PROPOSED 10-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	680.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	26.79 cfs

Computed Results:

Depth............	0.20 ft
Velocity.........	0.20 fps
Flow Area........	134.66 sf
Flow Top Width...	683.16 ft
Wetted Perimeter.	683.19 ft
Critical Depth...	0.04 ft
Critical Slope...	0.2817 ft/ft
Froude Number....	0.08 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SOUTH AREA - PROPOSED 25-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	680.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	38.79 cfs

Computed Results:

Depth............	0.25 ft
Velocity.........	0.23 fps
Flow Area........	168.23 sf
Flow Top Width...	683.95 ft
Wetted Perimeter.	683.98 ft
Critical Depth...	0.05 ft
Critical Slope...	0.2594 ft/ft
Froude Number....	0.08 (flow is Subcritical)

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SOUTH AREA - PROPOSED 100-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	680.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	55.79 cfs

Computed Results:

Depth............	0.31 ft
Velocity.........	0.27 fps
Flow Area........	209.33 sf
Flow Top Width...	684.91 ft
Wetted Perimeter.	684.95 ft
Critical Depth...	0.06 ft
Critical Slope...	0.2393 ft/ft
Froude Number....	0.08 (flow is Subcritical)

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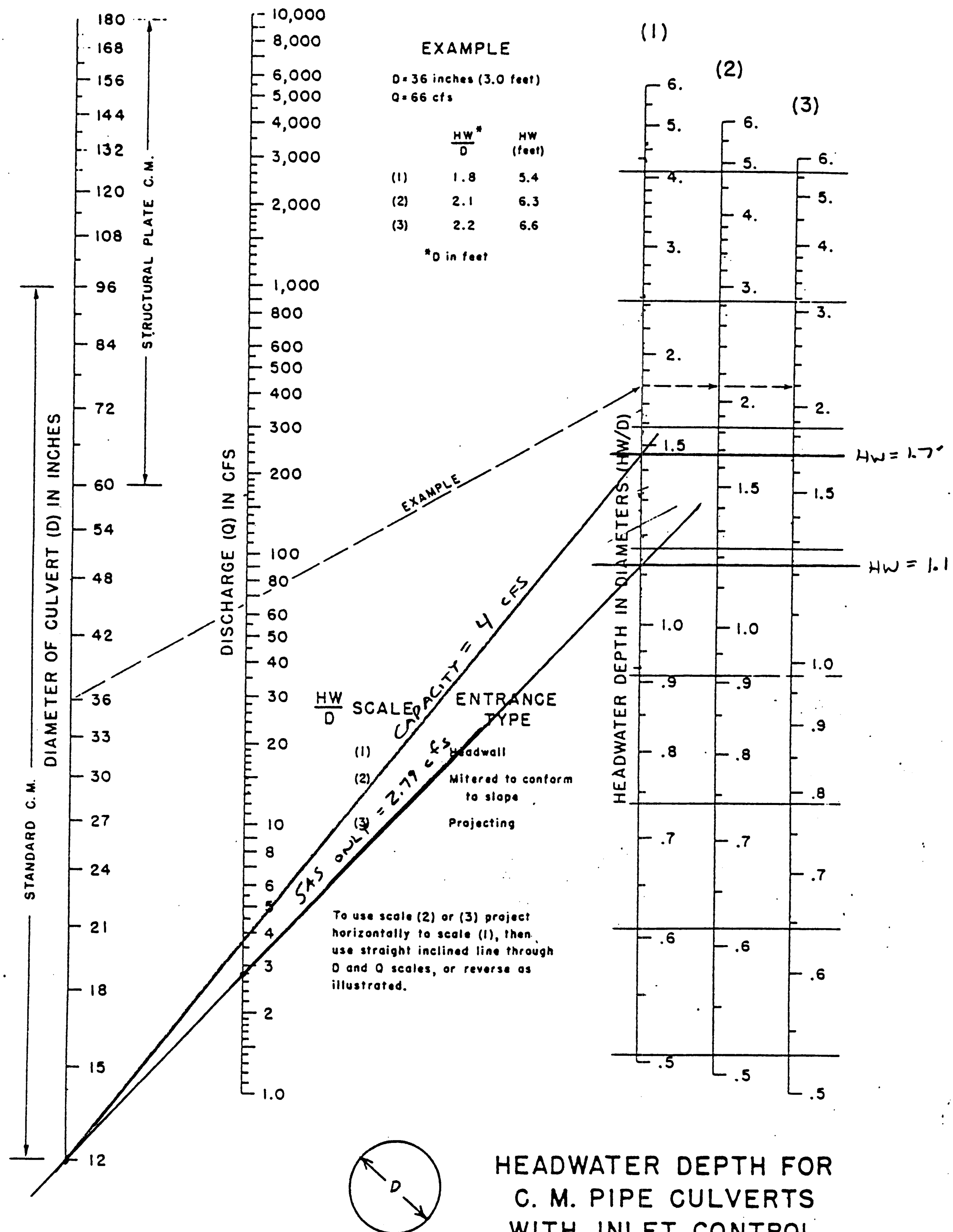
South Area 12" CMP

CHART 5



HEADWATER DEPTH FOR
C. M. PIPE CULVERTS
WITH INLET CONTROL

BUREAU OF PUBLIC ROADS JAN. 1963

5-23

Foth & Van Dyke
engineers · architects · scientists

Client: NMC-Keith Siding Road	Scope I.D.: ØØCØØZ
Project: SAS - Contingency Plan	Page: 1
Prepared By: SRB	Date: 1/31/00
Checked By: MDL	Date: 2/24/00

WITH NEW FLOW VALUES: SOUTH AREA

Depth comparison over road based on increase of flow over 4 cfs and 680 foot wide weir (width of wetland at road)

$$Q = 3LH^{1.5}, \text{ solve for } H: H = \left(\frac{Q}{3L}\right)^{2/3}$$

YEAR	EXISTING (Q)	PROPOSED (Q)	EXISTING H	PROPOSED H
2	4	6.79	1.70	1.71
5	9	11.79	1.72	1.72
10	24	26.79	1.75	1.75
25	36	38.79	1.76	1.77
100	53	55.79	1.78	1.79

Ex: $H = 1.7 + \left(\frac{2.79}{3(680)}\right)^{2/3} = 1.71$

Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SWAMP CREEK - PROPOSED BASE CONDITION 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	800.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	6.01 cfs

Computed Results:

Depth............	0.07 ft
Velocity.........	0.10 fps
Flow Area........	58.54 sf
Flow Top Width...	801.17 ft
Wetted Perimeter.	801.18 ft
Critical Depth...	0.01 ft
Critical Slope...	0.4070 ft/ft
Froude Number....	0.07 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SWAMP CREEK - PROPOSED 2-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	800.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	69.01 cfs

Computed Results:

Depth............	0.32 ft
Velocity.........	0.27 fps
Flow Area........	253.71 sf
Flow Top Width...	805.06 ft
Wetted Perimeter.	805.10 ft
Critical Depth...	0.06 ft
Critical Slope...	0.2367 ft/ft
Froude Number....	0.09 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SWAMP CREEK - PROPOSED 5-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	800.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	117.01 cfs

Computed Results:

Depth............	0.43 ft
Velocity.........	0.34 fps
Flow Area........	348.60 sf
Flow Top Width...	806.94 ft
Wetted Perimeter.	807.00 ft
Critical Depth...	0.09 ft
Critical Slope...	0.2105 ft/ft
Froude Number....	0.09 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SWAMP CREEK - PROPOSED 10-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	800.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	156.01 cfs

Computed Results:

Depth............	0.52 ft
Velocity.........	0.38 fps
Flow Area........	414.54 sf
Flow Top Width...	808.25 ft
Wetted Perimeter.	808.31 ft
Critical Depth...	0.11 ft
Critical Slope...	0.1975 ft/ft
Froude Number....	0.09 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SWAMP CREEK - PROPOSED 25-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	800.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	207.01 cfs

Computed Results:

Depth............	0.61 ft
Velocity.........	0.42 fps
Flow Area........	491.59 sf
Flow Top Width...	809.77 ft
Wetted Perimeter.	809.85 ft
Critical Depth...	0.13 ft
Critical Slope...	0.1855 ft/ft
Froude Number....	0.10 (flow is Subcritical)

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Trapezoidal Channel Analysis & Design
Open Channel - Uniform flow

Worksheet Name: NMC

Comment: SWAMP CREEK - PROPOSED 100-YR 1/00

Solve For Depth

Given Input Data:

Bottom Width.....	800.00 ft
Left Side Slope..	8.00:1 (H:V)
Right Side Slope.	8.00:1 (H:V)
Manning's n......	0.080
Channel Slope....	0.0010 ft/ft
Discharge........	280.01 cfs

Computed Results:

Depth............	0.73 ft
Velocity.........	0.47 fps
Flow Area........	589.83 sf
Flow Top Width...	811.71 ft
Wetted Perimeter.	811.80 ft
Critical Depth...	0.16 ft
Critical Slope...	0.1734 ft/ft
Froude Number....	0.10 (flow is Subcritical)

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