Project 89-223-19
August 1992

Canonie Environmental

SOUTH STREET
2.4
287755

Design Calculations
South Street Site Removal Action

REMOVAL RECORD

Walpole, Massachusetts

Prepared For:

South Street Site Potentially Responsible Parties

SDMS DocID 287755

Volume 2 of 2

Design Calculations
South Street Site Removal Action

Canonie Environmental

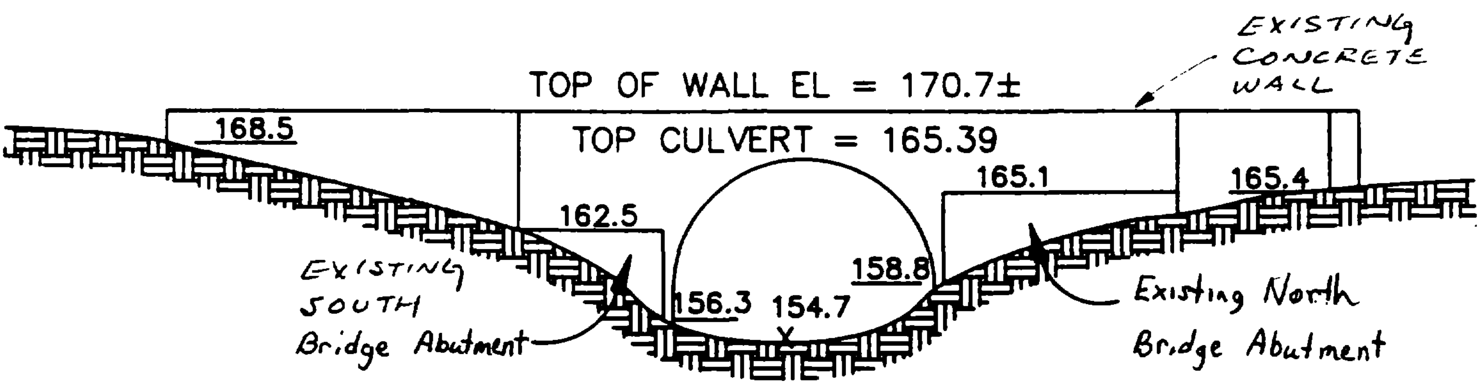
SECTION 3

3.0 RIVER DAM DESIGN

To ease the installation of the pipe arch culvert, the Neponset River will be dammed on the east side of the South Street bridge. This location was chosen because it was the site of a previous dam used to provide hydraulic power during the early part of the century. The abutments that remain from the original dam will be modified to provide the main support for the new dam. The dam will be a one-piece steel structure that will be lowered into place and rest against the abutments.

It is not imperative that the pipe arch culvert be installed in absolute dry conditions; therefore, some seepage under the dam can be tolerated. This is fortunate since the rocky conditions of the river bottom would make it difficult to embed the dam at any length that would provide an adequate water-tight seal. However, to provide a sufficient water seal, a portion of the riverbed between the dam location and the South Street bridge will be excavated. An impermeable liner will be installed in the excavated area prior to the dam being positioned at the abutments. After the dam has been installed, the impermeable liner will be covered with backfill to provide a resisting force to the uplifting pore pressure.

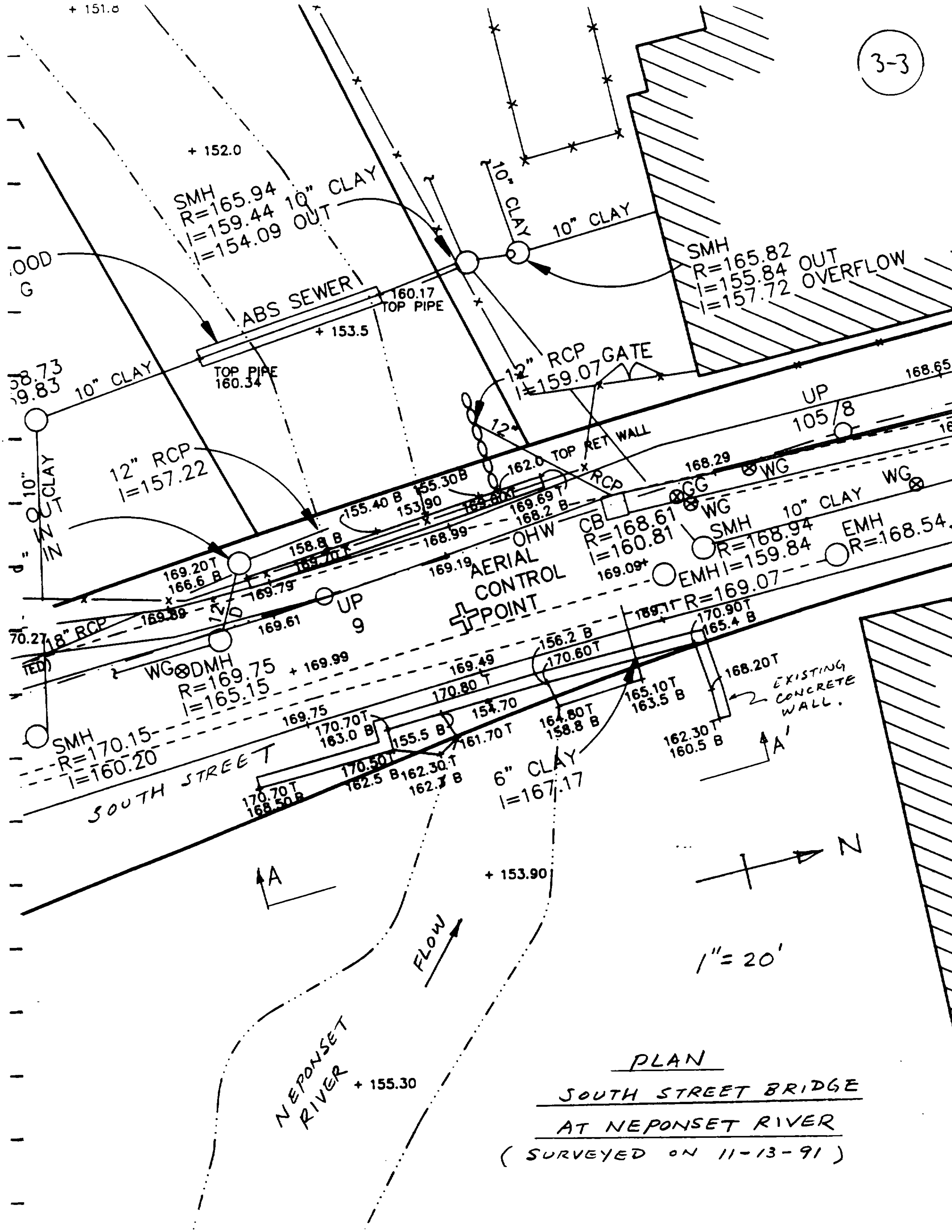
Section 3.1 Steel Frame Dam Design



EAST SIDE CULVERT ELEVATION A-A′

NOT TO SCALE

A DAM WILL BE PLACED AGAINST THE NORTH & SOUTH BRIDGE ABUTMENTS TO STOP THE RIVER FLOW FOR NEW ALUMINUM CULVERT CONSTRUCTION. THE DAM WILL BE REMOVED WHEN THE CULVERT CONSTRUCTION IS COMPLETED.



PLAN

SOUTH STREET BRIDGE

AT NEPONSET RIVER

(SURVEYED ON 11-13-91)

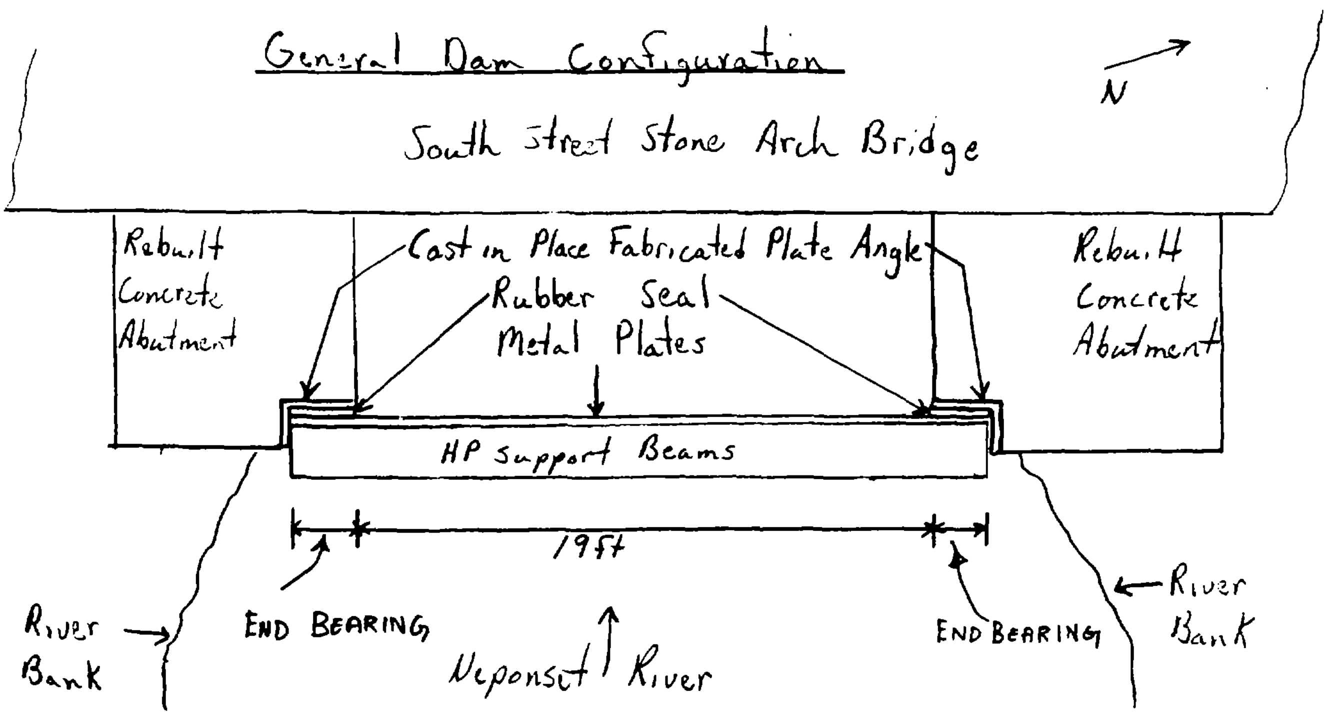
Canonie

By KAP Date 11/20/91 Subject South St Walpole Mass Sheet No. 1 of 10
Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 89-223-05
1/4" X 1/4"

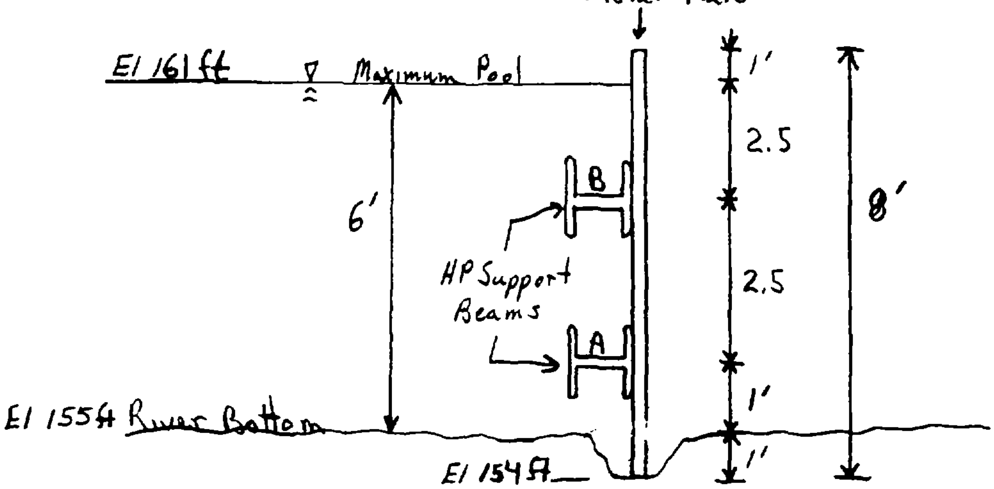
The Neponset River will be dammed on the east side of the South Street Stone arch bridge. The existing concrete abutments that are remnant of a previous dam at the same location will be rebuilt and used as primary supports.

The water level upstream of the dam will be controlled by four 15,000 gpm pumps that will be used to allow a maximum pool of six feet.

General Dam Configuration



Cross Section of Dam



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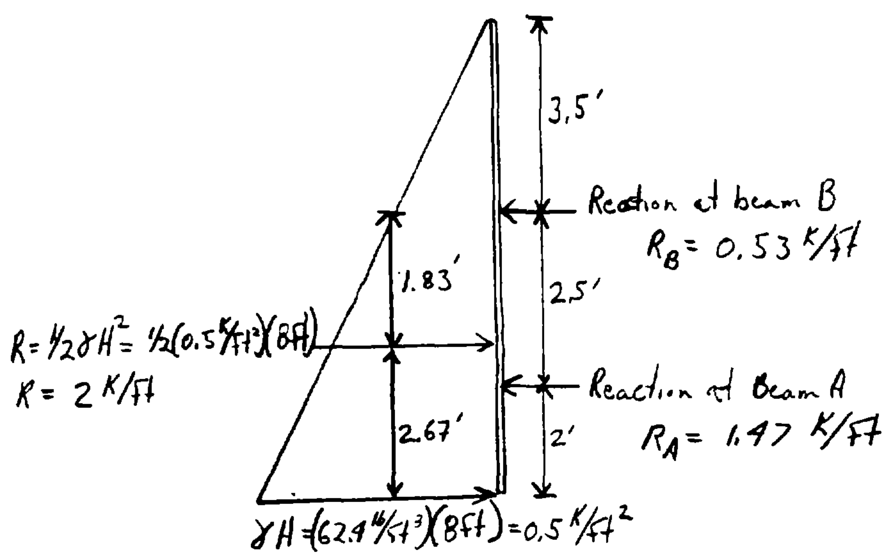
By KAP Date 11/20/91 Subject South Street, Walpole, Mass Sheet No. 2 of 10
Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 37-223-05
1/4" X 1/4"

For design purposes the entire hight of the metal plates will be assumed to be loaded by hydrostatic Forces. Therefore, the hydrostatic load will be assumed equal to an 8ft hight of water.

It will also be assumed that there is no support provided by the one foot of embedment

Loading Condition



$\circlearrowright + \Sigma M_B = 0$

$2.5' R_A = R(1.83')$

$R_A = \dfrac{(2\ ^K/_{ft})(1.83')}{2.5'}$

$R_A = 1.47\ ^K/_{ft}$

$\xrightarrow{+} \Sigma F_x = 0$

$R_A + R_B = R$

$R_B = R - R_A = 2\ ^K/_{ft} - 1.47\ ^K/_{ft}$

$R_B = 0.53\ ^K/_{ft}$

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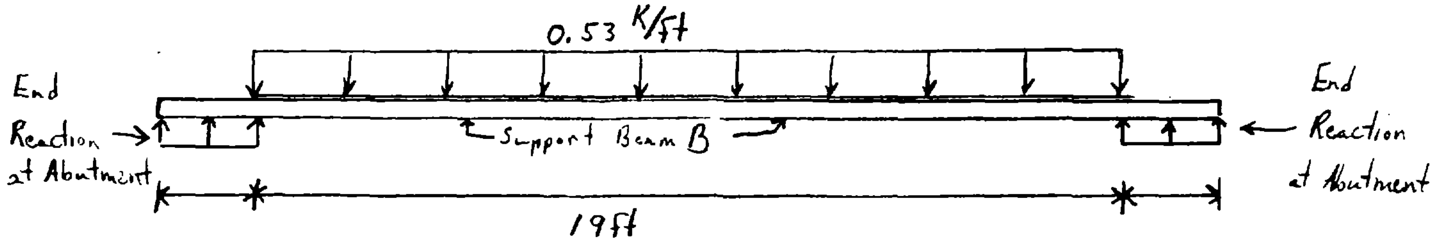
By KAP Date 11/20/91 Subject South St Walpole Mass Sheet No. 3 of 10
Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 89-223-05
1/4" X 1/4"

Loading on Beam B



Compact Beam $\Rightarrow$ $F_b = 0.66 F_y = 0.66(36 Ksi) = 24 Ksi$ Ref 3-1 p 5-20

Max moment $M = \frac{wL^2}{8} = \frac{(0.53 K/ft)(19 ft)^2}{8} = 23.92 K\text{-}ft$ Ref 3-1 p 2-114

Required Section Modulus
$$S_x = \frac{M}{F_b} = \frac{(23.92 K\text{-}ft)(12 in/ft)}{24 Ksi} = 11.96 in^3$$

Try HP 8×36 $S_x = 29.8 in^3$ $I_x = 119 in^4$ Ref 3-1 p 1-34

Actual Stress $f_b = \frac{M}{S_x} = \frac{(23.92 K\text{-}ft)(12 in/ft)}{29.8 in^3} = 9.63 \ll F_b$ ok

Checking Deflection $\Delta_{max} = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384}\frac{(0.53 K/ft)(1 ft/12 in)(19 ft \cdot 12 in/ft)^4}{(30,000 Ksi)(119 in^3)}$ Ref 3-1 p 2-11[illegible]

$\Delta_{max} = 0.435 in$

$\frac{\Delta_{max}}{L} = \frac{0.435 in}{(19 ft)(12 in/ft)} = 0.002 < \frac{L}{360}$ ok Ref 3-1 p 2-23

Checking Shear Stress

actual $f_v = \frac{1/2\, wL}{A_w} = \frac{1/2(0.53 K/ft)(19 ft)}{(0.445 in)(8.0 in)} = 1.41 Ksi$

allowable $F_v = 0.4 F_y = 0.4(36 Ksi) = 14.4 Ksi > f_v$ ok Ref 3-1 p 5-18

∴ HP 8×36 is addequate for the upper support beam

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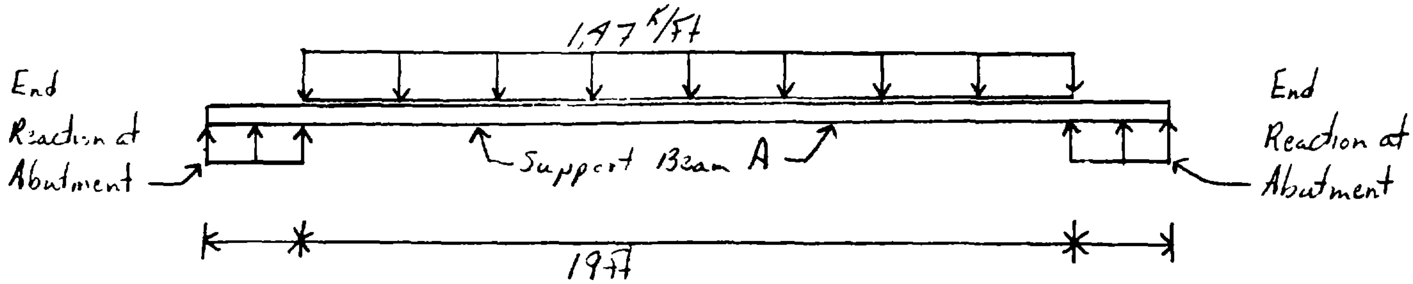
By KAP Date 11/20/91 Subject South St Walpole, Mass Sheet No. 4 of 10
Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 89-223-05
1/4" X 1/4"

Loading on Beam A



Compact Beam $\Rightarrow F_b = 24$ Ksi [Ref 3-1 p 5-20]

Max Moment $M = \frac{wL^2}{8} = \frac{(1.47\ K/ft)(19 ft)^2}{8} = 66.33$ K-ft [Ref 3-1 p 2-119]

Required Section Modulus
$$S_x = \frac{M}{F_b} = \frac{(66.33\ K\text{-}ft)(12\ in/ft)}{24\ Ksi} = 33.2\ in^3$$

Try HP 12×53 $S_x = 66.8\ in^3$ $I_x = 393\ in^4$ [Ref 3-1 p 1-34]

Actual Stress $f_b = \frac{M}{S_x} = \frac{(66.33\ K\text{-}ft)(12\ in/ft)}{66.8\ in^3} = 11.9 \ll F_b$ ok

Checking Deflection $\Delta max = \frac{5}{384}\frac{wL^4}{EI} = \frac{5}{384}\frac{(1.47\ K/ft \cdot 1 ft/12 in)(19 ft \cdot 12 in/ft)^4}{(30{,}000\ Ksi)(393\ in^3)}$

$\Delta max = 0.366$ in

$\frac{\Delta max}{L} = \frac{0.366\ in}{19 ft (12\ in/ft)} = 0.002 < \frac{1}{360}$ ok

Checking Shear Stress

actual $f_v = \frac{1/2\, wL}{A_w} = \frac{1/2(1.47\ K/ft)(19 ft)}{(11.78 in)(0.435 in)} = 2.73$ Ksi

allowable $F_v = 0.4 F_y = 0.4(36\ Ksi) = 14.4$ Ksi $> f_v$ ok [Ref 3-1 p 5-18]

∴ HP 12×53 is adequate for the lower support beam

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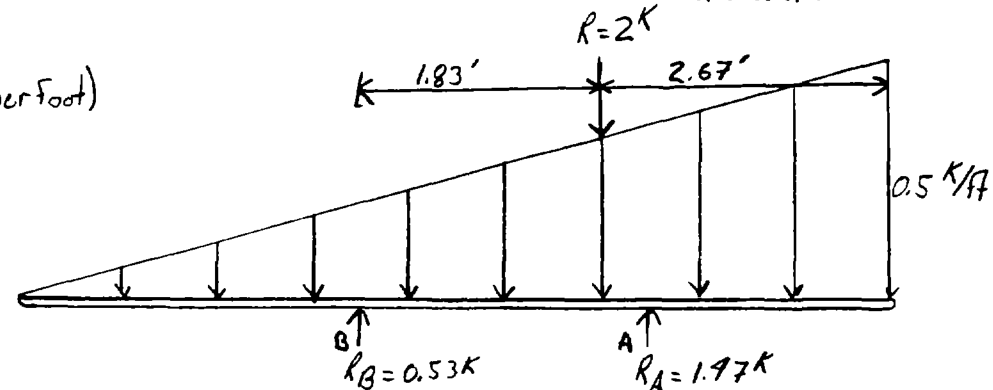
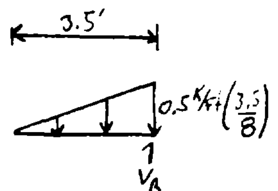
By KAP Date 11/20/91 Subject South St Walpole Mass Sheet No. 5 of 10

Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 89-223-05

1/4" X 1/4"

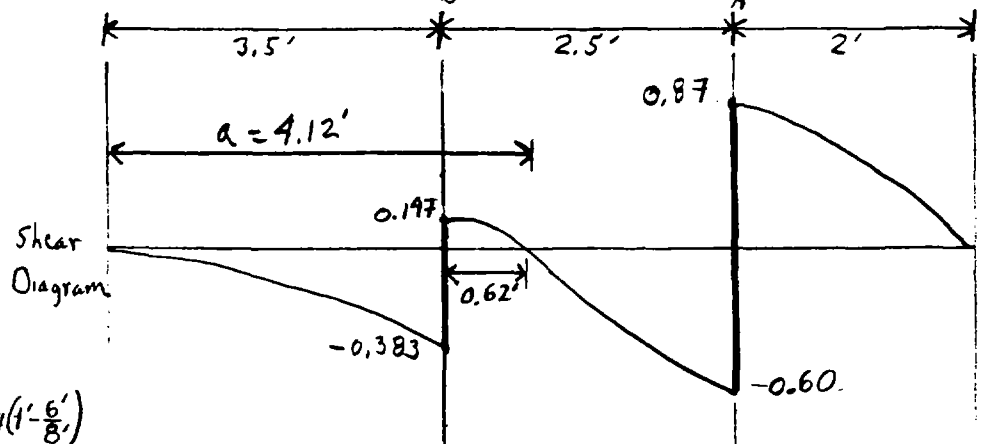
Loading on Steel Plate (per foot)

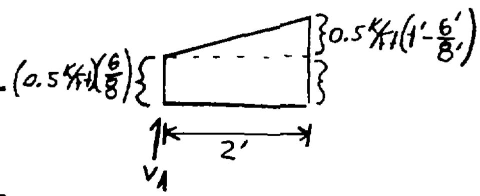




$V_B = \frac{1}{2}(0.5^{K/ft})(\frac{3.5}{8})(3.5')$

$V_B = 0.383^K$





$V_A = 2'(0.5^{K/ft})(\frac{6}{8}) + \frac{1}{2}(2')(0.5^{K/ft})(1' - \frac{6'}{8'})$

$V_A = 0.87^K$

Point of zero shear

$1/2(0.0624a)\,a = 0.53^K$

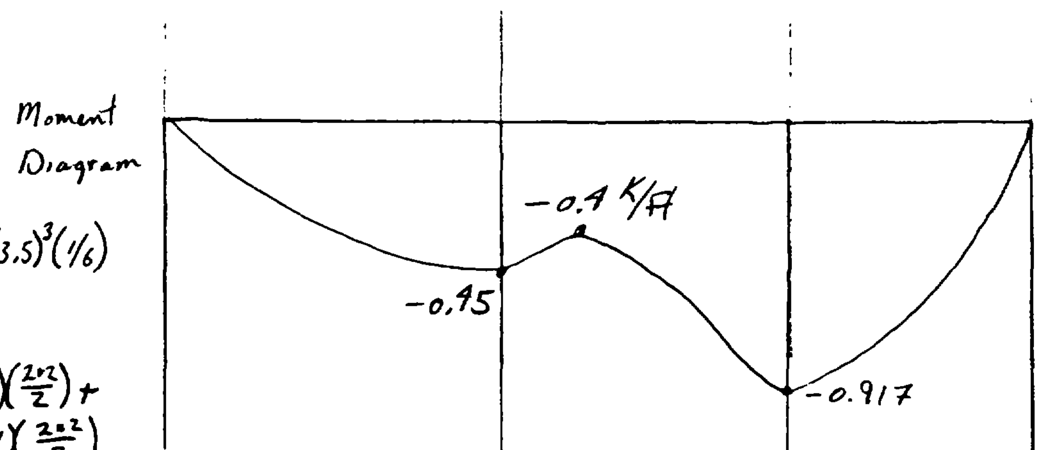
$0.0312a^2 = 0.53$

$a = 4.12'$ from the top of the steel plate

Moment at point of zero shear

$M = 0.53(4.12 - 3.5) - 0.0624(4.12)^3(1/6)$

$M = -0.4\ K\text{-}ft$



$M_B = (0.0624^{K/ft^3})(3.5)^3(1/6)$

$M_B = -0.45$

$M_A = (0.374^{K/ft^2})(\frac{2 \cdot 2}{2}) +$

$(0.5 - 0.374)(\frac{2 \cdot 2}{3})$

$= 0.917\ K\text{-}ft$

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By KAP Date 11/21/91 Subject South St Walpole Mass Sheet No. 6 of 10
Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 89-223-05
1/4" X 1/4"

Try 1/2" plate (Designed per foot)

$$S_x = \frac{1}{6} b h^2 = 1/6 (12'')(1/2'')^2 = 0.5 \text{ in}^3$$

$$\text{Required } S_x = \frac{M}{F_b} = \frac{(0.917 \text{ K-ft})(12''/\text{ft})}{24 \text{ Ksi}} = 0.459 \text{ in}^3 \approx S_x \quad \text{N.G.}$$

Try 5/8" plate

$$S_x = 1/6 b h^2 = 1/6 (12 \text{ in})(5/8'')^2 = 0.781 \text{ in}^3 > \text{Required } S_x \quad \text{oK}$$

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By KAP Date 11/21/91 Subject South St Walpole Mass Sheet No. 7 of 10
Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 39-223-05
1/4" X 1/4"

Beam to Plate Welds

The welds must be capable of transmitting the load on the plates due to the water pressure to the support beams.

The design load will be the 1.47 K/ft load that occurs at the lower support beam.

Determine the length of a 1/4 inch fillet weld (inches per foot) required using a shielded metal arc and E 70 electrodes ($F_u = 70$ Ksi)

Allowable shear:

effective throat of 1/4 fillet Weld $t_e = 0.707a = 0.707(.25") = 0.177"$

shear per inch $= R_w = t_e(0.3F_u) = 0.177"(0.3)(70\text{ Ksi}) = 3.7\text{ K/in}$

Required length of weld per foot of beam

$$L = \frac{(1.47\text{ K/ft})(1\text{ft})}{3.7\text{ K/in}} = 0.4\text{ in}$$

∴ Use 1/4 inch fillet weld 1 inch long ever six inches on both sides of the flange for both the upper and lower beam

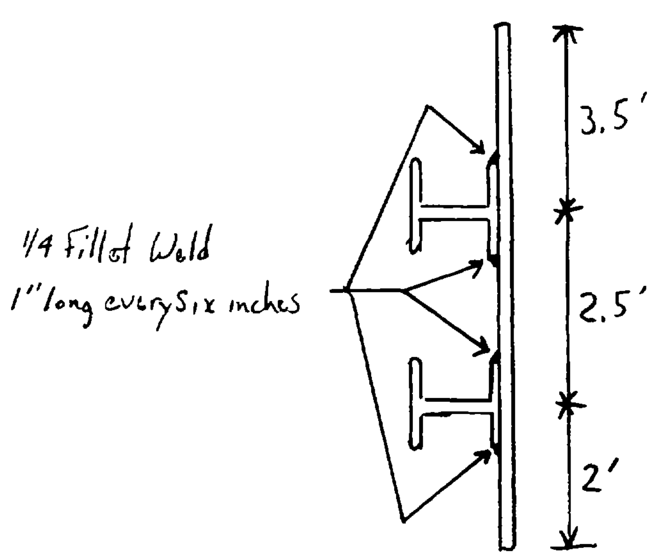


Plate to Plate Welds

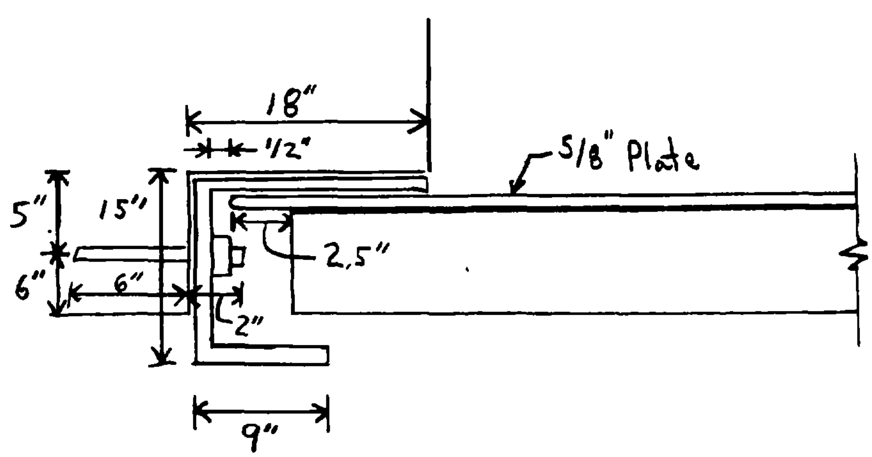
The sections of 5/8" metal plates will be welded together to provide a water tight seal.

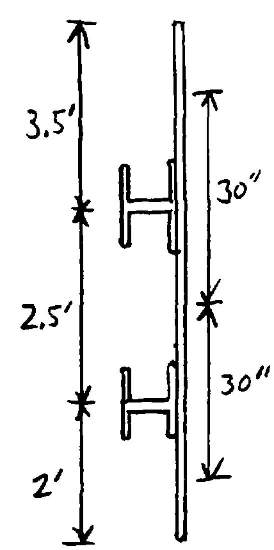
A complete penetration groove weld will be used at every plate to plate connection. The weld will run the entire 8 ft of the connection.

Concrete Abutment Bearing Plate

The concrete abutments will be rebuilt by the contractor to conform with the desired configuration

A fabricated 18" x 15" x 9" Channel 1/2" thick will be connected to the rebuilt abutments as shown by the contractor





Determine if the 18" bearing surface is adequate
assume a 30" effect bearing length to avoid a combined bearing effect

Effective Bearing Area $= (18''-2'')(30'') = 480\ in^2 = A_1$
Assume concrete area $A_2 = A_1$

Allowable Bearing $F_p = \phi(.85 f_c') A_1 \sqrt{A_2/A_1}$ Ref 3-2 Sec 10.15

$F_p = 0.7(0.85)(3\ Ksi)(480\ in^2)(1)$
$F_p = 857\ K$

Actual End load $= 1/2\ wL = 1/2(1.47\ K/ft)(19\ ft) = 13.97^K << F_p$ ok

Actual Bearing $f_p = \dfrac{R}{BN} = \dfrac{13.97^K}{(30'')(16'')} = 0.029\ Ksi$ Ref 3-1 p2-47

Minimum Plate thickness

$t_{min} = \sqrt{\dfrac{3 f_p n^2}{F_b}}$ $n = \dfrac{B}{2} - K = \dfrac{30''}{2} - 1\ 1/8'' = 13.875''$

$t_{min} = \sqrt{\dfrac{3(0.029\ Ksi)(13.875'')^2}{0.75(36\ Ksi)}} = 0.79'' \approx 1/2''$ (Angle Plate) + 5/8 (Dam Metal Plate)
ok

∴ The fabricated 18"×15"×9" channel 1/2" thick will act as an addequate bearing plate.

Total Weight

5/8" Plate Area $= (21.863')(8') = 174.7\ ft^2$ Wt = 26.6 lb/ft² Ref 3-1 1-99
HP 12×53 21.42' long
HP 8×36 21.42' long

Total Wt $= (174.7\ ft^2)(26.6\ lb/ft^2) + (53\ lb/ft + 36\ lb/ft)(21.42') = 6.5^K$

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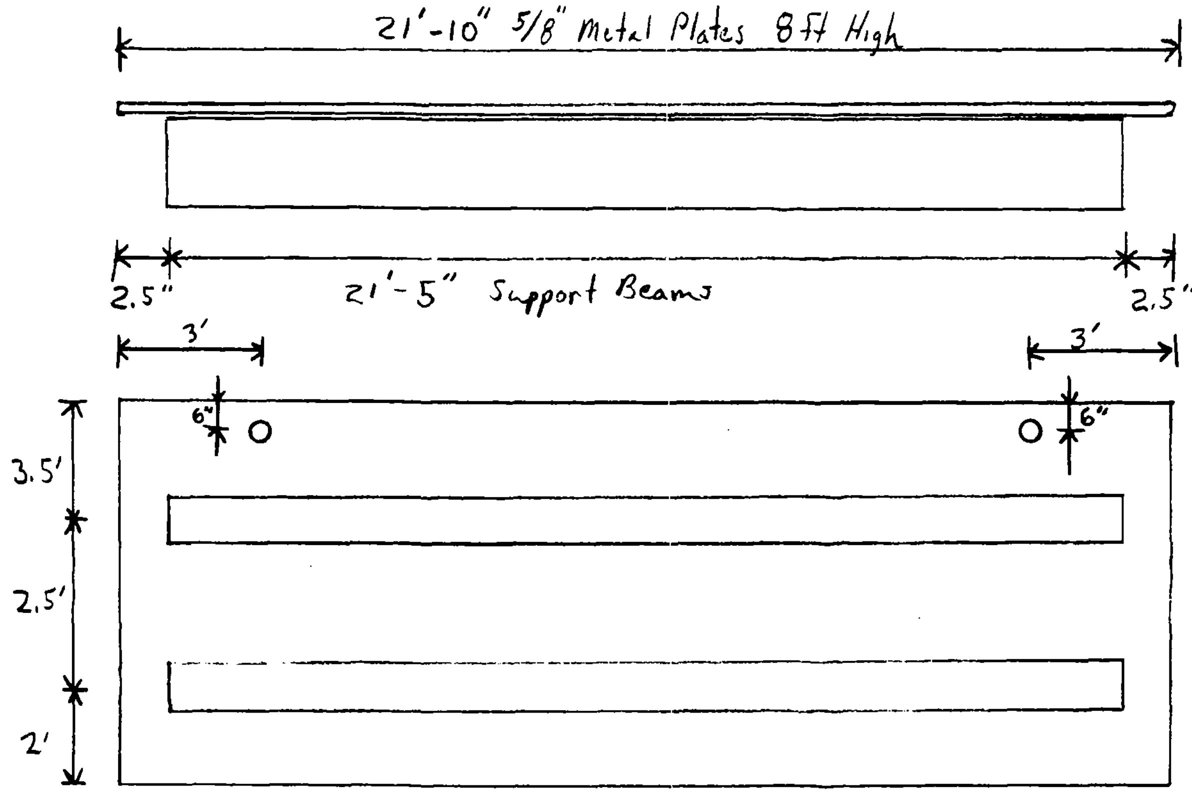
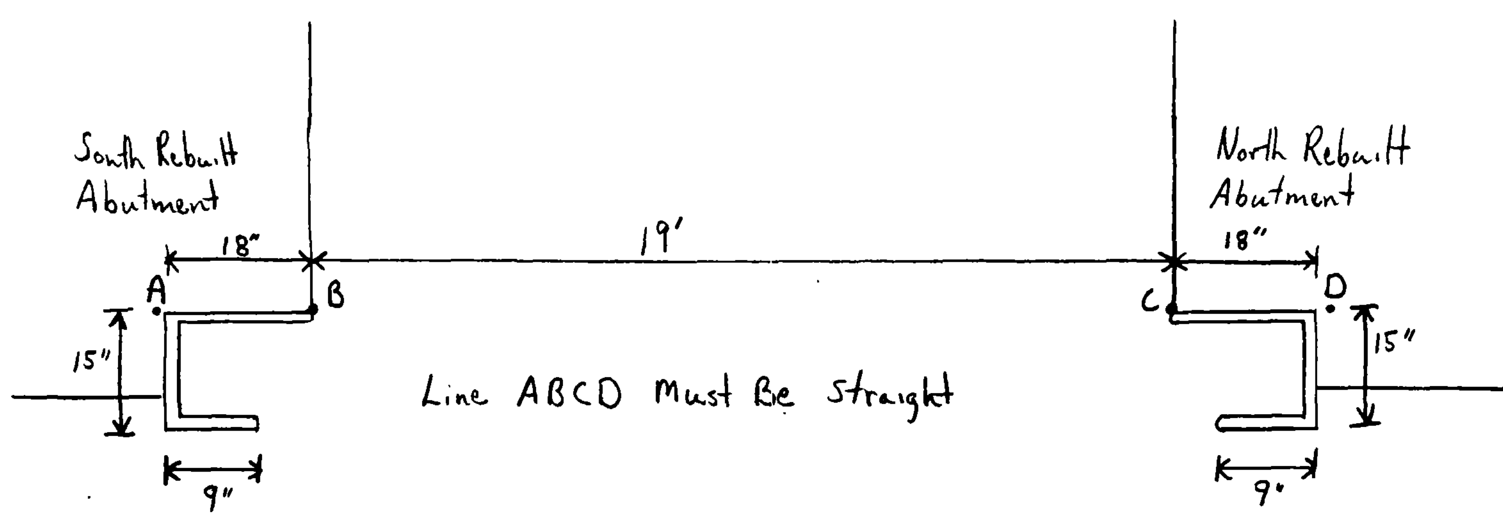
By KAP Date 11/21/91 Subject South St Walpole, Mass Sheet No. 10 of 10

Chkd. By CC Date 11-22-91 Dam Calculation Proj. No. 89-223-05

1/4" X 1/4"

Final Configuration

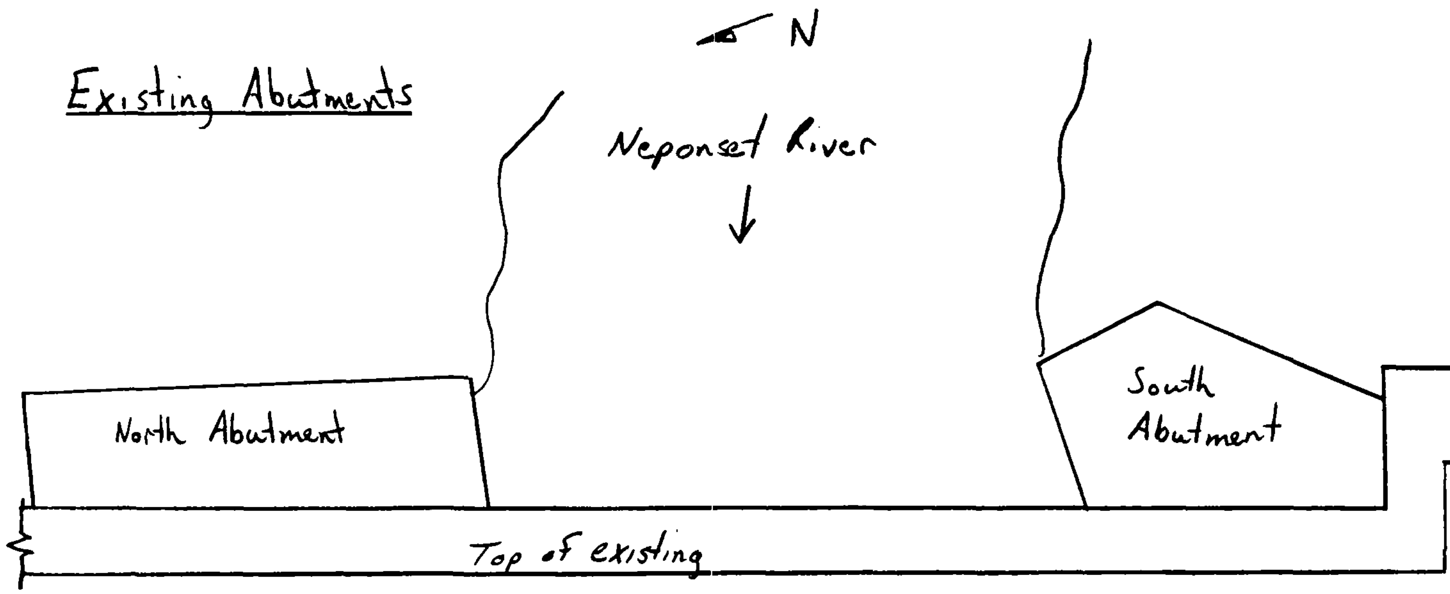




Section 3.2
The existing abutments are remnant of a previous dam at the South Street bridge. The abutments will be modified for the placement of the temporary dam to be installed during the culvert construction.

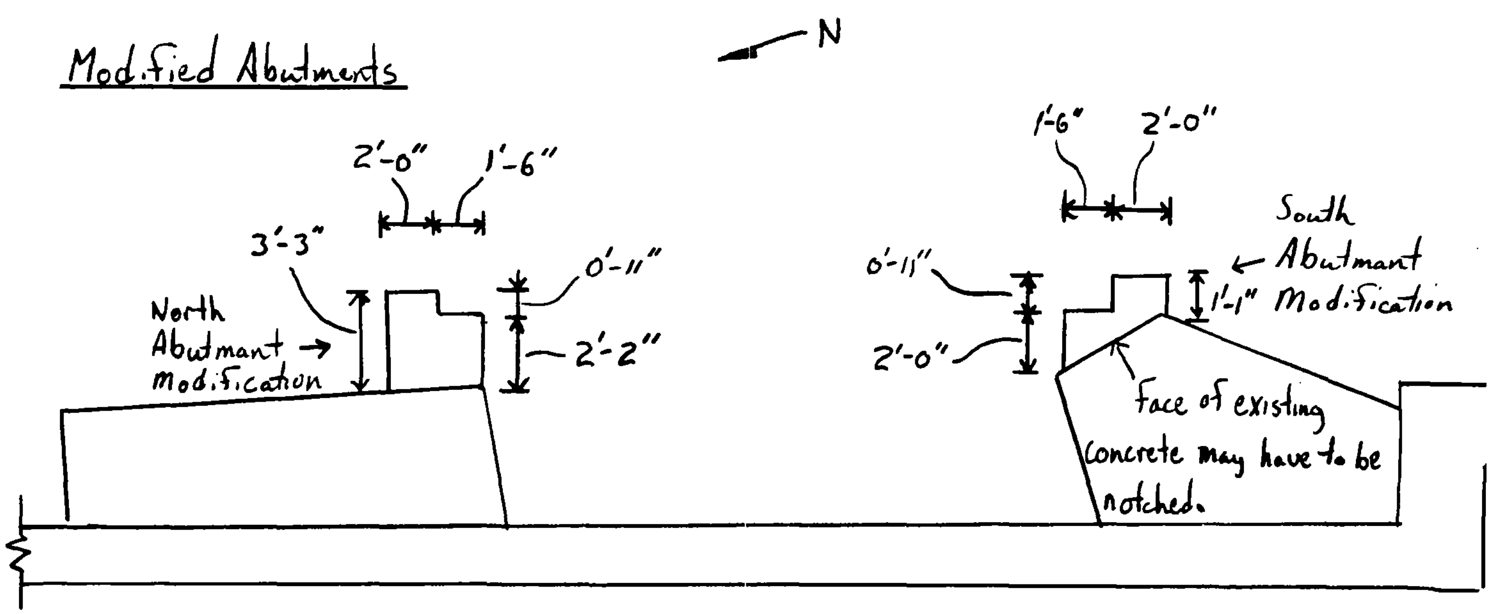
The dimensions and details provided below shall be field checked and adjusted if necessary by the concrete subcontractor.

Existing Abutments



1" = 6.7 ft

Modified Abutments



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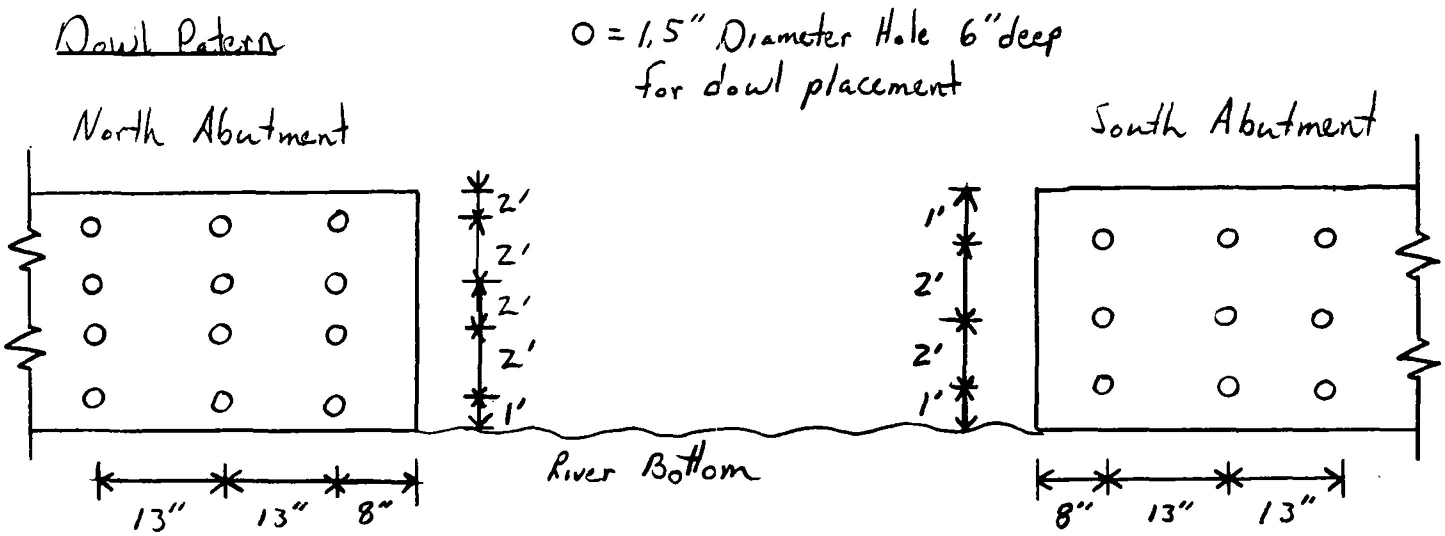
By KAP Date 6/2/92 Subject South Street Sheet No. 2 of 4
Chkd. By CC Date 6-2-92 Modified Abutment Details Proj. No. 89-223-17
1/4" X 1/4"

The new concrete will be conected to the existing concrete with 18" long 1" diameter dowles. The dowles will be used for each abutment modification. Each dowel will be grouted into a 1.5" diameter 8" deep hole in the existing concrete.

Dowl Patern



The new concrete will be reinforced with #5 ASTM standard reinforcing bars as shown in the following figures

Use Schmidt concrete test hammer to check the deterioration of the old concrete. The deteriorated surfaces shall be removed and cleaned. The dowels will be grouted into the existing concrete and the concrete face will be coated with Sika-dur (or equivalent) bonding agent prior to pouring of new concrete

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By NAS Date 6/8/92 Subject South Street Dowels Sheet No. 1 of 2
Chkd. By CC Date 6-8-92 Proj. No. 89-223-17
1/4" X 1/4"

DESIGN OF DOWELS
Ref. Yoder + Witczak "Principles of Pavement Design"

eq. 3.35 $\sigma = K y_o = \frac{H P_t}{4 \beta^3 E I} (2 + \beta z)$

eq. 3.29 $\beta = \sqrt[4]{\frac{K b}{4 E I}}$ in^{-1}

where:
E = modulus of elasticity of steel dowels = 29×10^6 psi
I = moment of inertia of steel dowels
b = width of dowel or diameter of dowel
K = modulus of dowel support concrete
= 1.5×10^6 psi
z = joint opening = 0
P_t = transferred load
σ = dowel bearing stress on concrete, 1000 psi maximum for existing deteriorated concrete.

Design dowels between existing deteriorated concrete and the poured fresh new concrete with gap = 0.

The purpose of the dowels is to connect the existing abutments to the modified abutments. The loads acting are as follows:

- force of water against the dam pushing the abutments together
- weight of the dam and HP support beams which will be resting on the ground

Therefore, no significant load transfer will occur at the dowels and analysis is unnecessary.

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By NAS Date 6/8/92 Subject South Street Dowels Sheet No. 2 of 2

Chkd. By CC Date 6-8-92 Proj. No. 89-223-17

1/4" X 1/4"

In case of settlement or other possible causes of lateral load, dowels will be provided. To bond the old and new sections of concrete abutments the Sikadur Hi-Mod, also called sikastix 370, will be used.

Characteristics of Sikadur Hi-Mod:

tensile strength = 3,500 psi
compressive strength = 8,400 psi min. at 28 days with 1:3¼ aggregate
compressive modulus = 600,000 psi min. at 28 days with 1:3¼ aggregate

If Sikadur Hi-Mod is not available, SONOBOND or an equivalent bonding agent can be used.

Reinforcement of Concrete Abutments

The force of water against the dam will cause a compressive force on the abutments. Since concrete is strong under compression and no tensile loads are anticipated, steel reinforcing bars are not necessary for the applied loads. The #5 reinforcing bars specified in the abutment details will serve to minimize cracking and tie the structure together thereby assuring that the abutment acts as intended in the design. Area of the reinforcement is based on minimum shrinkage and Temperature reinforcement required by ACI code 318-83 section 7.12.2.1

⇒ Required Steel area = 0.0014 Gross Concrete Area

The vertical reinforcement (6 #5 bars) use 4 #5 bars as ties. The vertical reinforcement is designed to withstand the shrinkage and temperature stresses.

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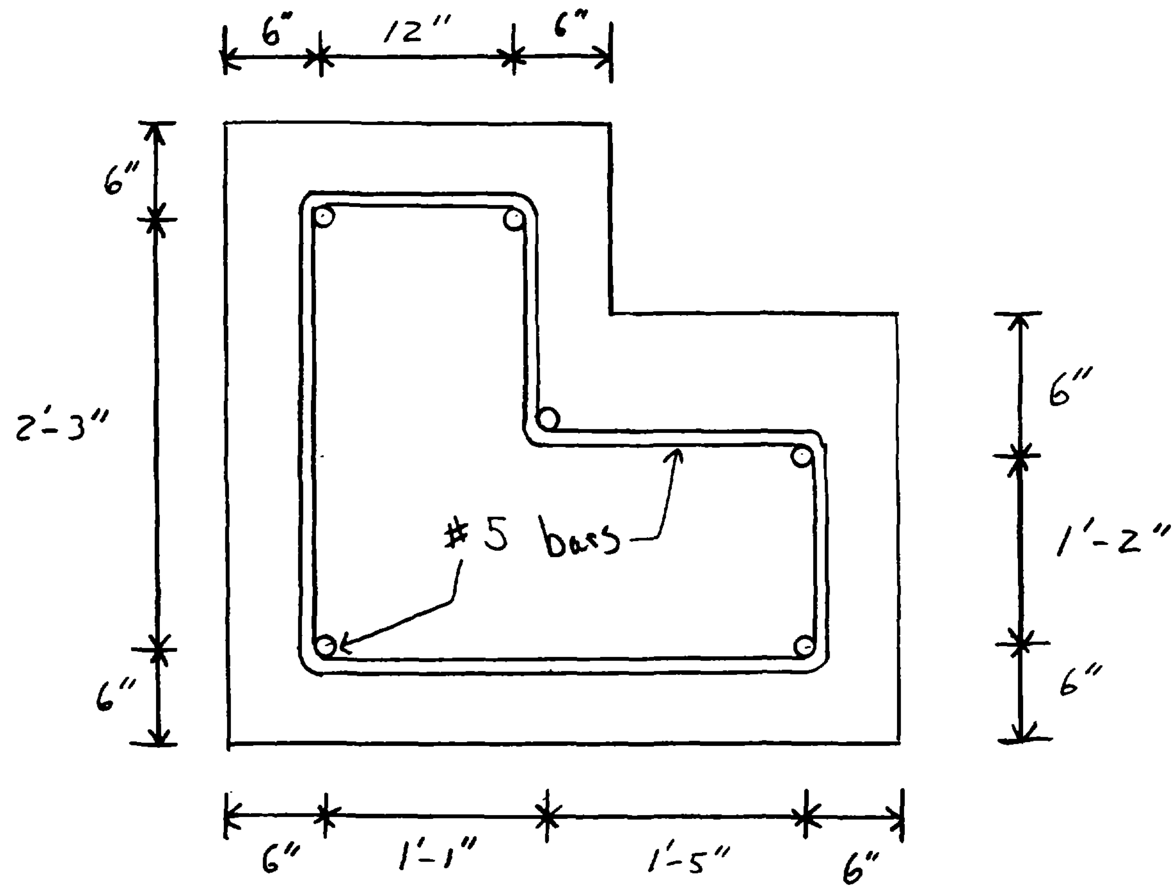
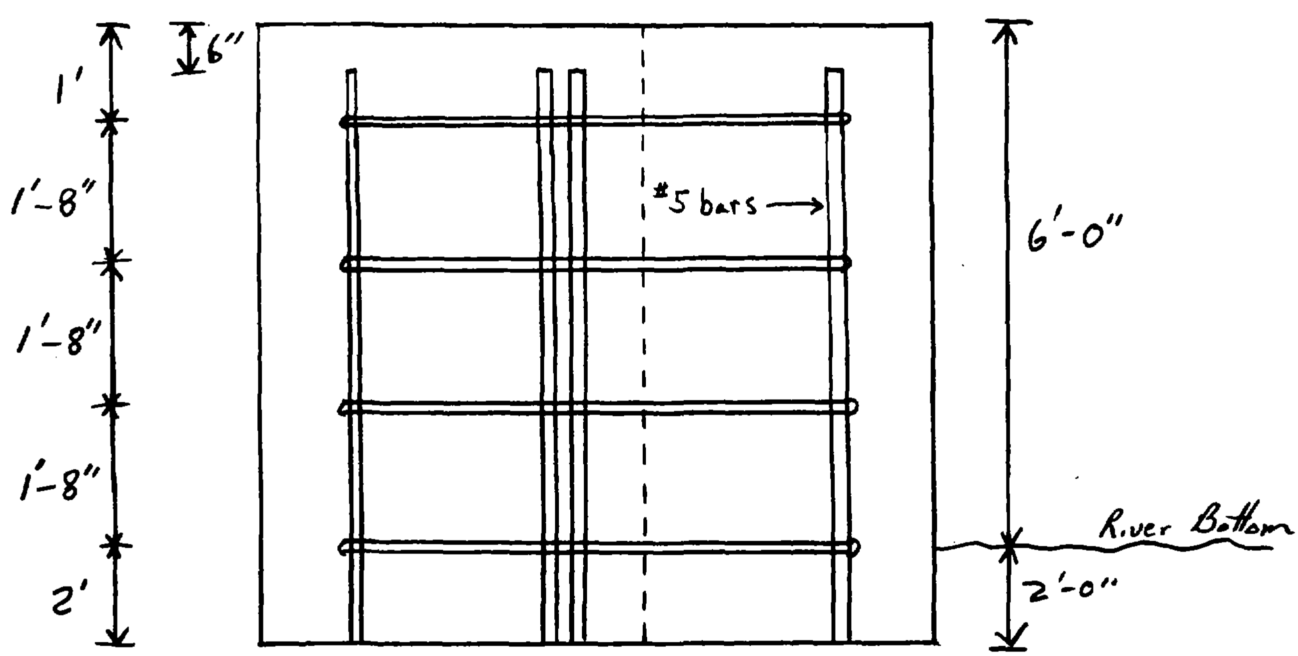
By KAP Date 6/2/92 Subject South Street Sheet No. 3 of 4

Chkd. By CC Date 6-2-92 Modified Abutment Details Proj. No. 89-223-17

1/4" X 1/4"

North Modified Abutment





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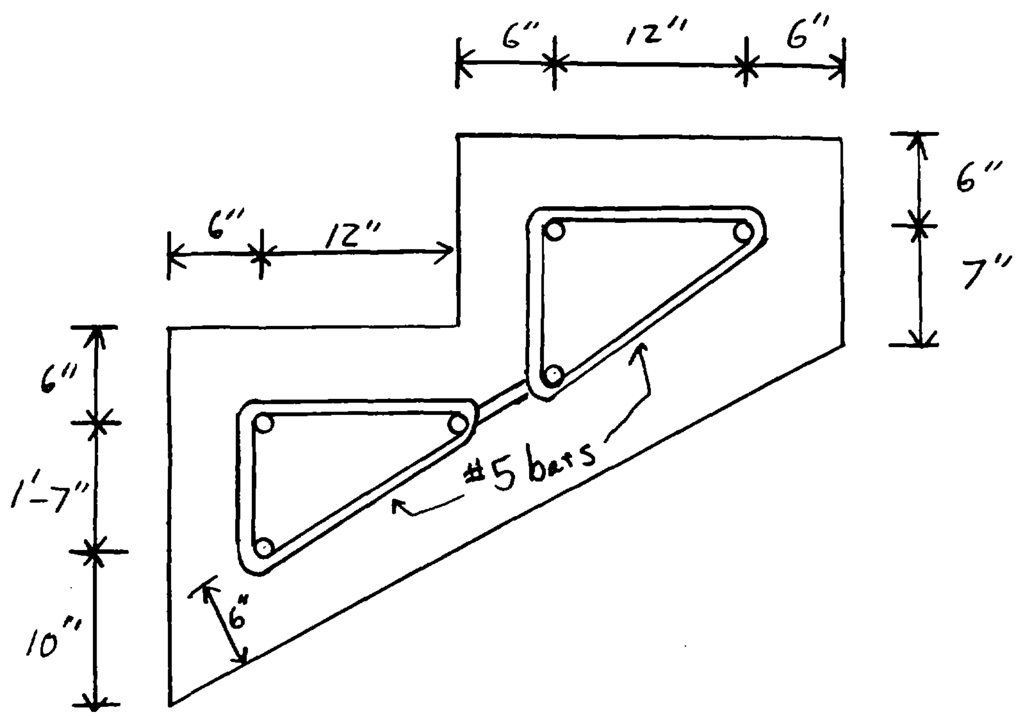
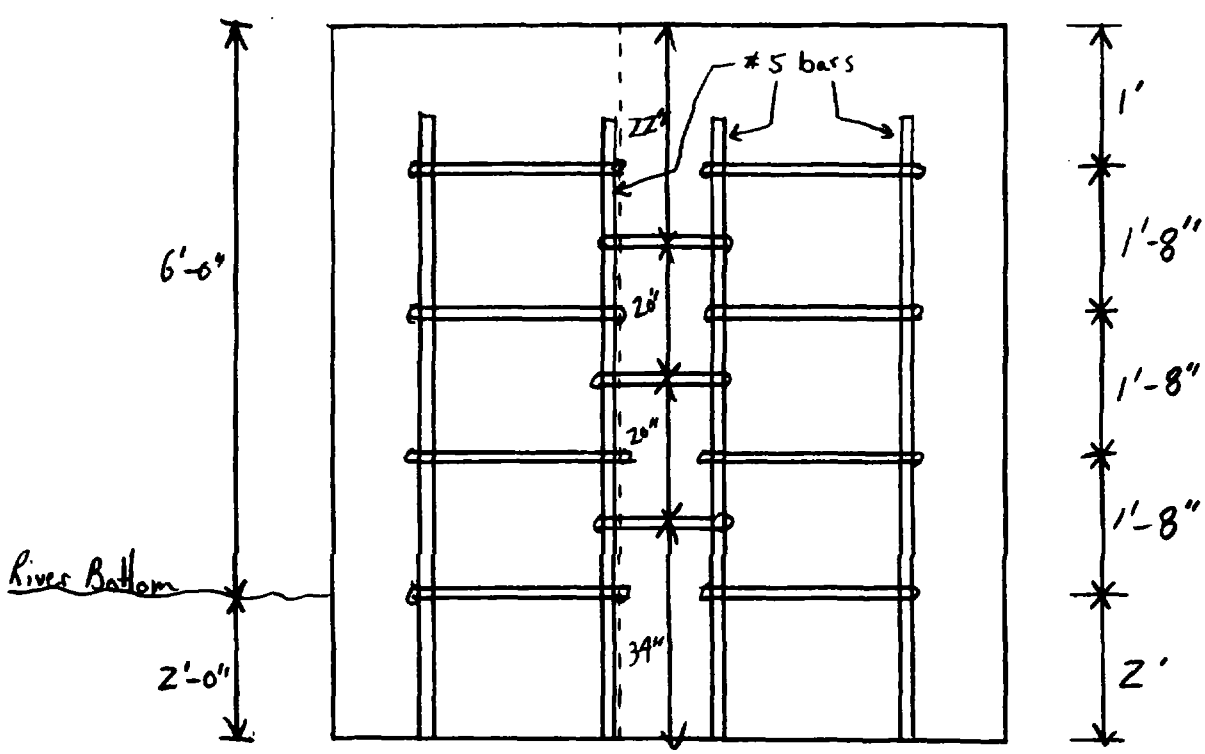
By KAP Date 6/2/92 Subject South Street Sheet No. 4 of 4
Chkd. By CC Date 6-2-92 Modified Abutment Details Proj. No. 89-223-17
1/4" X 1/4"

South Modified Abutment





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By MT Date 6/1/92 Subject Grace - South Street Sheet No. 1 of 12
Chkd. By RMC Date 6/1/92 Dam Water Balance Proj. No. 89-223-17

1/4" X 1/4"

Section 3.3 Temporary Dam Water Balance

Purpose: The purpose of this calc brief is to evaluate the capability of the pumping system above the temporary dam to pass the 2-, 3-, 4-, and 5-year flood events. If the 5-year flood event overtops the dam, the maximum water surface elevation (WSEL) that the water reaches at the dam will be estimated.

Methods: A spreadsheet was used to perform the water balance calculations. The inflow hydrograph was modelled after the hydrograph produced by the SCS's TR-55 tabular method using a NOAA Type III rainfall distribution. The outflow consisted of pumping at ~~either~~ the inflow rate up to a maximum of 133 cfs (60,000 gpm), the design rate for the pumping system. The spread sheet tracks volume of inflow and outflow and notes excess inflow as storage. The maximum accumulated storage is then compared to an elevation-storage curve to determine whether the dam is overtopped.

The preliminary results indicated that the 2- and 3-year floods did not overtop the dam, while the 4- and 5-year floods did overtop it. To determine the extent to which the 5-year flood overtopped the dam, an elevation-outflow curve was developed for flow over the dam acting as flow over a sharp-crested weir. The spread sheet was modified to determine the average cumulative storage volume for a particular

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By MT Date 6/1/92 Subject Grace South Street Sheet No. 2 of 12
Chkd. By PMC Date 6/1/92 Dam Water Balance Proj. No. 89-22...
1/4" X 1/4"

time-step. The WSEL that corresponds to this volume was then read from the elevation-storage curve. Then the outflow resulting from this elevation was read from the elevation-outflow curve and the change in storage deducted from the accumulated storage. This final stored volume was then used in the next time step.

Results: The results indicated that the 5-year flood would overtop the dam by 0.7 feet to reach a WSEL of 162.7 ft. The maximum outflow would be approximately 38.4 cfs.

The following pages describe the input required for the water balance spread sheet and the calculations performed by the spreadsheet.

Canonie

By MT Date 6/1/92 Subject Grace South Street Sheet No. 3 of 12
Chkd. By RMC Date 6/1/92 Dam Water Balance Proj. No. 89-223-17
1/4" X 1/4"

Peak Discharge Amounts

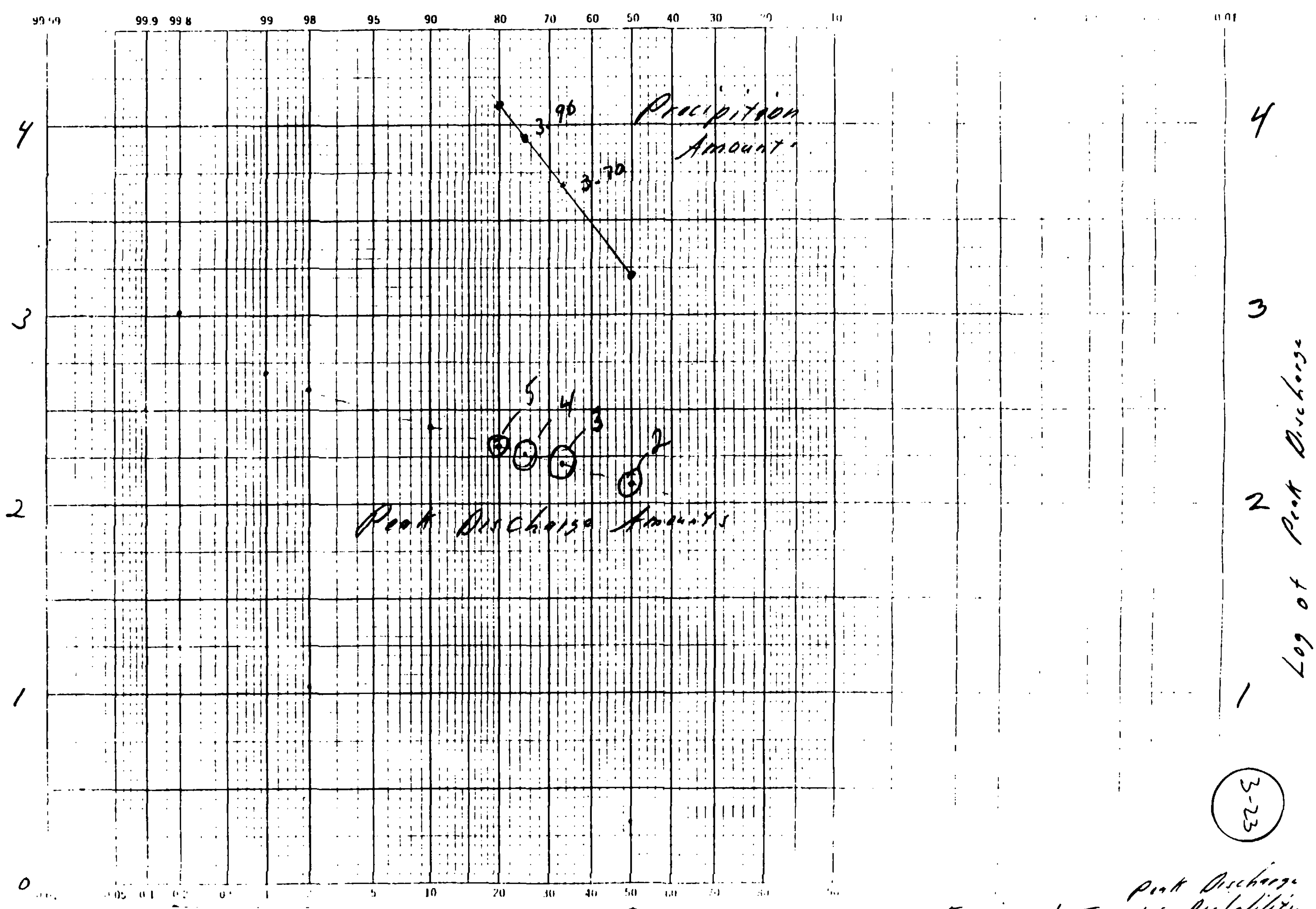
Page 9 of the FEMA report (see Ref 1-1) listed peak discharges for floods of various recurrence intervals

Recurrence Interval (yrs)	Peak Discharge (cfs)
10	261
50	416
100	498
500	1050

The logs of the peak discharges were plotted on probability paper so that the peak discharges of the 2-, 3-, 4-, and 5-year floods could be estimated. The plot is shown on Figure 1. The results are summarized below

Recurrence Interval (yrs)	Peak Discharge (cfs)
2	126
3	162
4	182
5	200

The peak discharge of the 2-year flood calculated above is reasonably close to the 133 cfs previously calculated. To be conservative the 133 cfs will be used herein.



Precipitation Amount
3.96
3.70
Peak Discharge Amounts
5
4
3
2
Log of Peak Discharge
3-23

Inflow Hydrographs

The FEMA Floodplain study used a hydrograph-generation model called SWAMP to account for the swampy areas in the Neponset River drainage area. Because this model was not available, the TR-55 model was used instead by adjusting the curve number to recreate the peak discharges for the 2, 3, 4, and 5-year flood events noted above. The spreadsheets show the input values used for generating the hydrographs. Columns 1, 2, and 3 show the results of the hydrograph.

The 24-hour precipitation amounts were taken from the U.S. Weather Bureau's TP-40. (see Ref 2-20) for the 4 yr & 5-year storms. These values were plotted on probability paper and the 3-year and 4-year amounts read from the graph. The results are summarized below:

Recurrence Interval (yrs)	24-hr Precipitation Amount (inches)	
2	3.20	
3	3.70	See Figure 1
4	3.90	See Figure 1
5	4.10	

The Curve Number was adjusted to reach the desired peak discharge. The S value was determined by the SCS formula [Ref 3-3]

$$S = \frac{1000}{CN} - 10$$

Canonie

3-25

By MT Date 6/1/92 Subject Grace South Street Sheet No. 7 of 12
Chkd. By RMC Date 6/ Dam Water Balance Proj. No. 89-223-07
1/4" X 1/4"

The I_a value was taken from Table 4-1 of TR-55 for each curve number [Ref. 3-3]

Q is calculated from Equation 2-1 of TR-55 as follows

$$Q = \frac{(P - I_a)^2}{P - I_a + S}$$

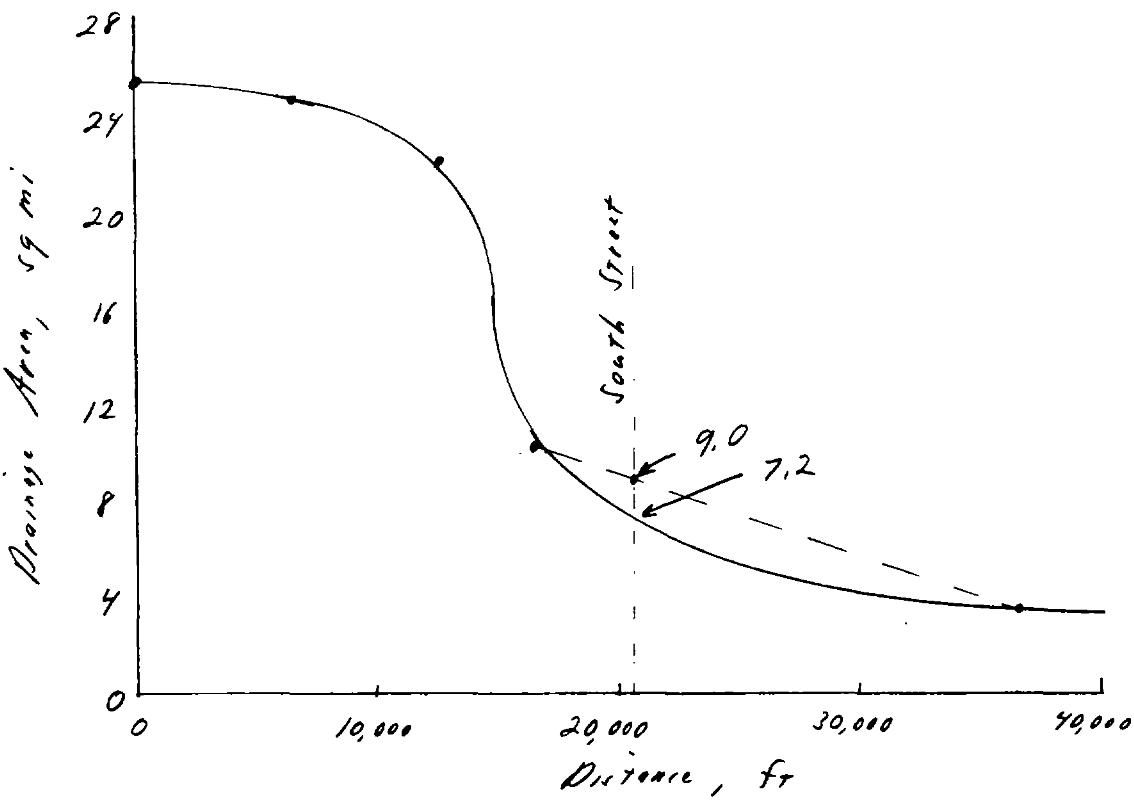
where Q = runoff amount, inches
P = precipitation amount, inches
I_a = Initial abstractions, inches
S = Retention, inches

The area of the Neponset River drainage area was determined by using information provided in the FEMA reports. The distance from the corporate limits was plotted against the drainage areas upstream from several locations on the Neponset River. The drainage area above South Street Bridge was estimated to be 9.0 sq mi. This is a conservative estimate since the plot indicates that the drainage area could be as low as 7.2 sq. mi.

The time of concentration (Tc) for this drainage area was estimated to be 2 hrs based on a channel length of 45,000 feet and an average flow velocity of 6.25 fps. This Tc is conservative considering the ~~num~~ number of dams + bridges on the river.

Canonie

By MT Date 5/29/92 Subject Grace - South Street Sheet No. 8 of 12
Chkd. By RMC Date 6/19/92 Estimation of Drainage Area at South Street Proj. No. 89-223-17
1/4" X 1/4"



Distance (ft)	Area (sq mi)
0	25.8
6,500	24.9
12,600	22.2
16,200	10.6
36,600	3.5

∴ For conservatism, use 9.0 sq.mi.

Reference: FEMA Report, page 9 and channel profiles

Canonie

By MT Date 6/1/92 Subject Grace South Street Sheet No. 9 of 12
Chkd. By KMC Date 6/1/92 Dam Water Balance Proj. No. 89-227-17
1/4" X 1/4"

The tabular values shown in Exhibit 5-III [Ref 3-3] of TR-55 at a T_c of 2.0 hours and I_a/P values of 0.5 were used to generate the hydrograph at south street. The hydrograph was extended to 30 hours and a time increment of 0.1 hours was used to provide a complete hydrograph. Column 1 provides the time increment while Column 2 provides the unit discharge values. For each time increment the discharge was calculated as the unit discharge value from Exhibit 5-III multiplied by the Q and Area values in inches and square miles, respectively. The discharge is provided in Column 3.

For each recurrence interval, the CN and Ia values were adjusted until the peak discharge approximates the desired or target peak discharge.

Canonie

By MT Date 6/1/92 Subject Grace South Street Sheet No. 10 of 12
Chkd. By RMC Date 6/1/92 Dam Water Balance Proj. No. 89-223-17
1/4" X 1/4"

Interim Water Balance

For each time increment the volume of inflow was calculated in Column 4 as the average of the beginning and ending inflow rates times the number of seconds (360) in the time increment.

The pumped volume was set at the inflow volume to reflect that the pumping rate can be set to match the inflow rate. The maximum pumped volume was set at 47880 cu ft (133 cfs x 360 sec). This is the design pumping rate for the pumping system. Column 5 shows the pumped volume.

Column 6 provides the change in storage volume as the difference between inflow volume and pumped volume. Column 7 shows the cumulative storage volume at the end of each time increment.

Preliminary Results

The preliminary results of the water balance calculations are summarized below. The WSEL's were determined from the elevation-storage curve determined for the area upstream from the South Street Bridge.

Recurrence Interval (yrs)	Maximum Stored Volume (cu ft)	Preliminary Maximum WSEL (ft)
2	0	~156 (Minimum Pool Elev)
3	112700	161.3
4	233000	162.7
5	367700	163.7

Canonie

By MT Date 6/1/92 Subject Grace South Street Sheet No. 11 of 12
Chkd. By RLC Date 6/ 92 Dam Water Balance Proj. No. 89-223-17
1/4" X 1/4"

Revised Water Balance

Thus, the 4- and 5-year floods will overtop the temporary dam. However, at this point the water balance does not account for the reduction in storage caused by water overflowing the dam. To account for this the water balance was modified by adding Columns 8-12 for the 5-year flood.

Column 8 provides the average stored volume for each time step by averaging the beginning and ending ~~en~~ accumulated volumes. Column 9 provides the WSEL for this volume from the elevation-storage curve. Column 10 provides the average outflow rate for the time increment from the elevation-outflow curve determined in pages — through — of this calc brief. Column 11 provides the storage volume removed during the time step as the outflow rate times 360. Column 12 provides the final storage volume calculated by subtracting the outflow volume (Column 11) from the accumulated storage volume (Column 7).

Column 7 was modified to reflect the addition of the incremental storage volume in Column 6 to the previous time step's final storage volume in Column 12.

Canonie

3-30

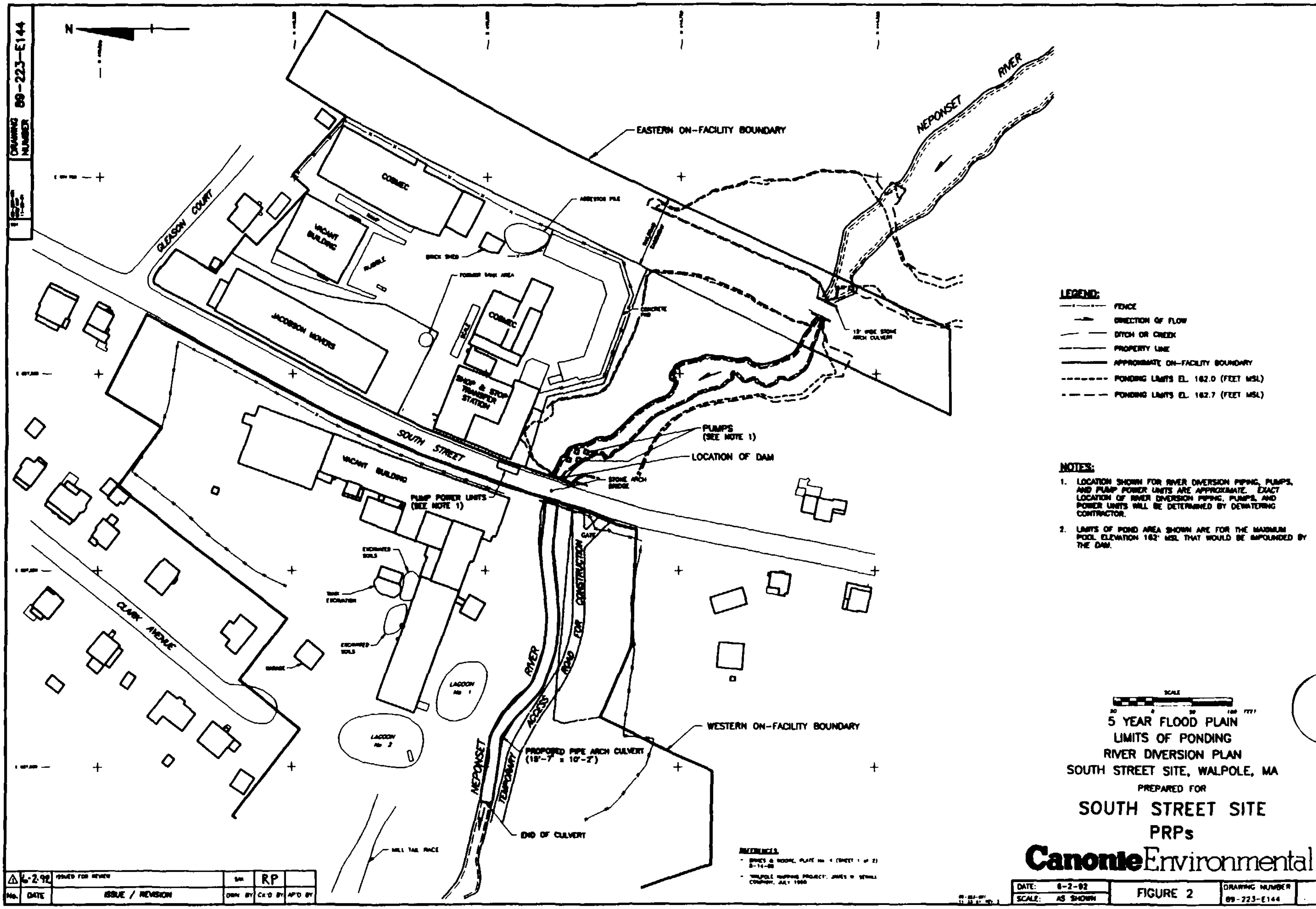
By MT Date 6/1/92 Subject Grace South Street Sheet No. 12 of 12

Chkd. By RLC Date [illegible] Dam Water Balance Proj. No. 87-223-17

1/4" X 1/4"

Results

The results indicate that the maximum WSEL for the 5-year flood will be 162.7 feet, that the maximum stored volume will be, 235,500 cfs, and the maximum flow rate over the dam will be about 38.4 cfs.



5 YEAR FLOOD PLAIN
LIMITS OF PONDING
RIVER DIVERSION PLAN
SOUTH STREET SITE, WALPOLE, MA
PREPARED FOR
SOUTH STREET SITE
PRPs

3-31

3-32

TABLE 1

Volume Calculations
South Street Site
Walpole, MA

Contour	Distance Between Contours	Cross-Sectional Areas (a) (Sq. Feet)	Average End Areas (Sq. Feet)	Incremental Volumes (Cu. Feet)	Cumulative Volumes (Cu. Feet)
156		1,186.33 sf			
	2'		5,197.82 sf	10,396 cf	
158		9,209.31 sf			10,396 cf
	2'		19,773.92 sf	39,548 cf	
160		30,338.52 sf			49,944 cf
	2'		61,839.03 sf	123,678 cf	
162		93,339.53 sf			173,622 cf
	2'		117,958.76 sf	235,918 cf	
164		142,577.98 sf			409,540 cf

Total Volume = 409,540 cf

Total Volume = 3.06 MGallons

Note:

(a) Areas Encompassed By Contours Were Calculated By CADD From Canonie Drawing Number 89-223-A1

CanonieEnvironmental

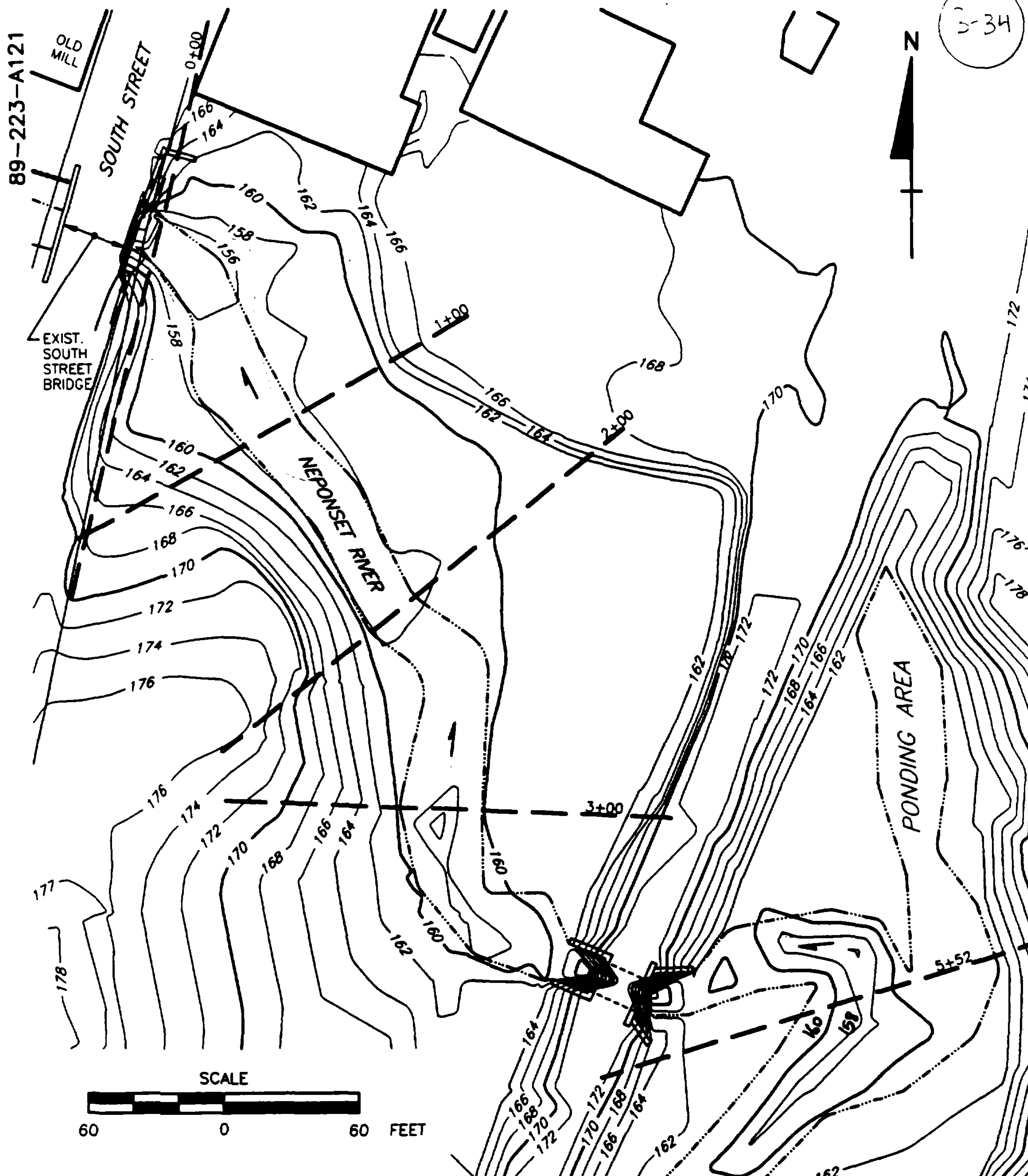
By M A M Date 5-28-92 Subject SOUTH STREET AREAS Sheet 1 of 1

Checked By ________ Date ________ Project No. 89-223-17

CONTOUR DESIGNATION	AREA (WEST)	AREA (EAST)	TOTAL AREA
156	1180.63	5.70	1186.33
158	8181.14	1028.18	9209.31
160	25910.39	4428.13	30338.52
162	49306.92	44023.62	93339.53
164	5~~0~~621.31 9	82956.67	142577.98

89-223-A121

3-34



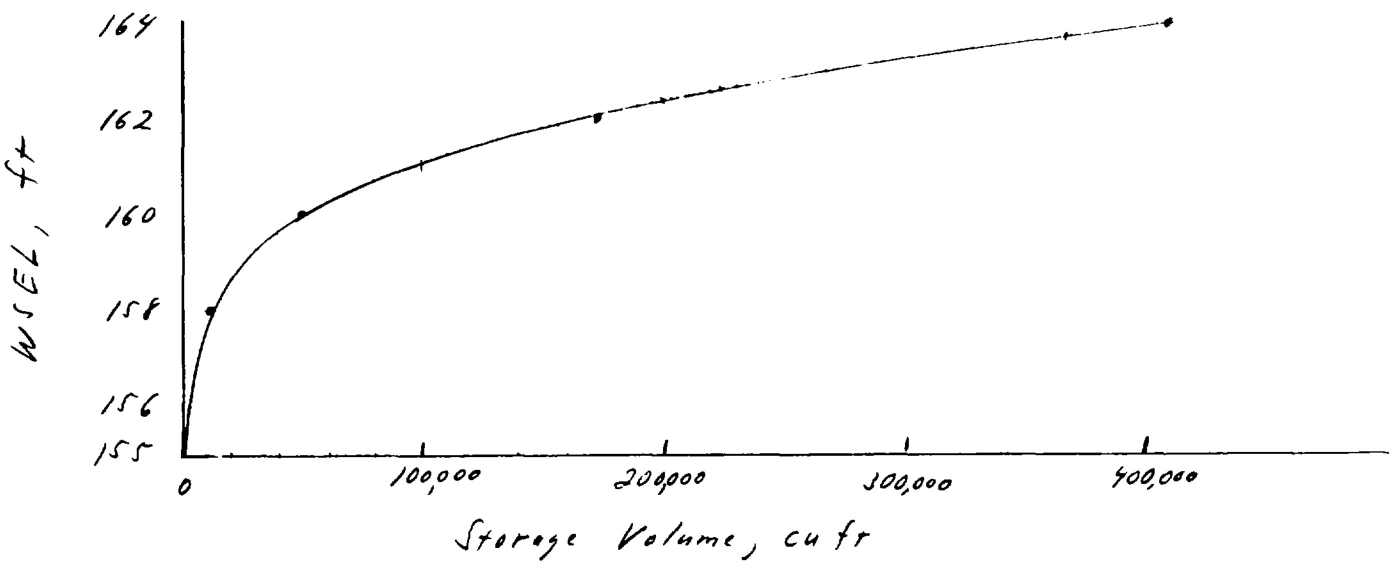
SECTION PLAN
NEPONSET RIVER AREA
UPSTREAM OF SOUTH STREET BRIDGE
WALPOLE, MA.

SOUTH STREET SITE
PRPs

Canonie Environmental

Canonie

By MT Date 5/29/92 Subject Grace - South Street Sheet No. 1 of 1
Chkd. By RNC Date 6/1/92 Elevation - Volume Curve above Temporary Dam Proj. No. 89-223-17
1/4" X 1/4"



Canonie

By MT Date 5/29/92 Subject Grace - South Street Sheet No. 1 of 3

Chkd. By RP Date 5/29/92 Elevation - Outflow Curve Proj. No. 89-223-17

Temporary Dam

1/4" X 1/4"

Purpose: The purpose of this calc brief is to develop the elevation-outflow curve for flows overtopping the temporary dam at the South Street bridge.

Methods: The flows consist of both the pumping (max = 153 cfs) from the river as well as potential flows that overtop the dam. The dam will act as a sharp-crested weir with 2, 1, or 0 end sills based on elevation of the water surface (WSEL). Sheets 10 and 11 of 12 of the construction drawings show the dam details. The top of dam elevation is 162.0'. Its nominal length is 19' between the sills.

The formulas for determining the discharge, Q, over the spillway for a given height of water, H, are taken from Chow (1959), p 362 as follows: [Ref 3-4]

$$Q = C L H^{1.5} \qquad (14\text{-}9)$$

where

Q = discharge, cfs

C = discharge coefficient

L = effective length of weir, ft

H = measured head above the crest, ft

$$L = L' - 0.1 N H \qquad (14\text{-}10)$$

where L' = measured length of weir = 19'

N = number of end contractions or sills

for the given situation, n is as follows

$162 < WSEL \leq 162.5$, $n = 2$

$162.5 < WSEL \leq 165.1$, $n = 1$

$WSEL > 165.1$, $n = 0$

Canonie

By M.T. Date 5/29/92 Subject Grace - South Street Sheet No. 2 of 3

Chkd. By ____ Date ______ Elevation - Outflow Curve Proj. No. 89-223-17

Temporary Dam 1/4" X 1/4"

$$C = 3.27 + 0.40 \frac{H}{h} \qquad (14\text{-}11)$$

where h = height of weir, ft

$h = 162' - 155' = 7'$ (See Section A-A', Sheet 11)

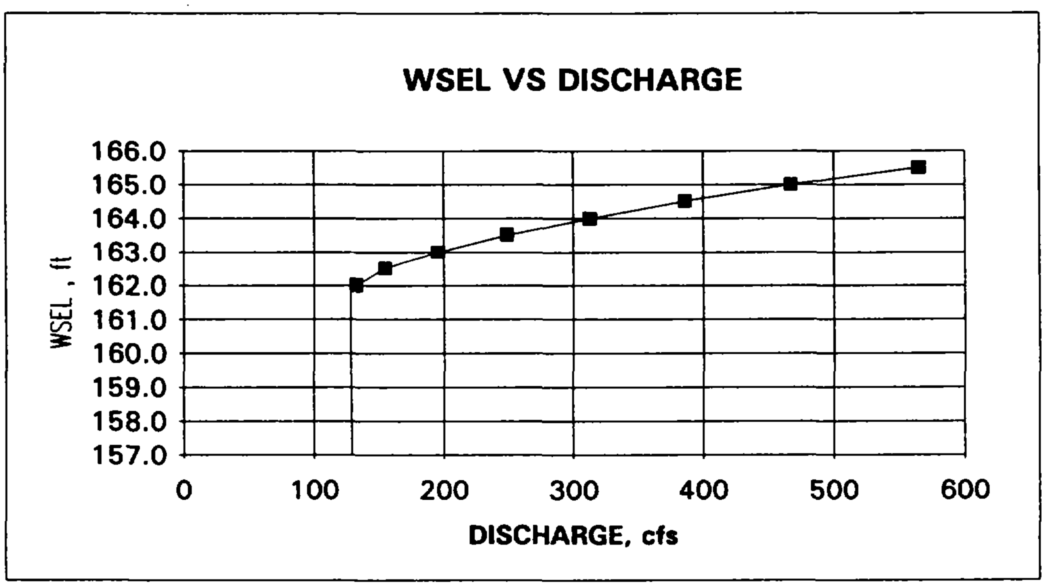
The flow over the concrete abutment extensions shown on Sheet 11 was not included in this calc brief because the one on the right side (looking upstream) is narrow (~1' wide) while the one on the left side is nearly as high as the top of the culvert.

Results : The table next page provides the results for these calculations. It shows selected WSEL's, H, L, C, ~~and~~ Q_w (discharge over the weir), Q_p (pumping rate), and Q_T (combined discharge). The graph on page 3 shows the elevation outflow curve for this structure.

3-38

ELEVATION - OUTFALL CURVE DATA FOR TEMPORARY DAM

WSEL	H	h	[H/h]	C	n	L	Qw	Qp	Qt
< 162.0	0.0	7.0						133	133
162.0	0.0	7.0	0.00	3.27	2	19.00	0	133	133
162.5	0.5	7.0	0.07	3.30	2	18.90	22	133	155
163.0	1.0	7.0	0.14	3.33	1	18.90	63	133	196
163.5	1.5	7.0	0.21	3.36	1	18.85	116	133	249
164.0	2.0	7.0	0.29	3.38	1	18.80	180	133	313
164.5	2.5	7.0	0.36	3.41	1	18.75	253	133	386
165.0	3.0	7.0	0.43	3.44	1	18.70	334	133	467
165.5	3.5	7.0	0.50	3.47	0	19.00	432	133	565



3-39

2-YEAR FLOOD

WATER BALANCE FOR AREA ABOVE TEMPORARY DAM

Recurrence interval (RI) = 2 yrs
Precipitation (P) = 3.20 in
Curve number (CN) = 50
Storage (S) = 10.00 in
Initial abstractions (Ia) = 2.00 in
Ia/P = 0.63
Runoff (Q) = 0.12857143 in
Drainage Area (A) = 9.00 sq mi

Column No. 1	2	3	4	5	6	7
Maximums =		132	47489	47489	0	0
Time hrs	Unit Discharge csm/in	Discharge cfs	Inflow Volume cu ft	Pumped Volume cu ft	Incremental Storage Volume cu ft	Cumulative Storage Volume cu ft
12.4	1	1.2	208	208	0	0
12.5	3	3.5	833	833	0	0
12.6	6	6.9	1875	1875	0	0
12.7	11	12.7	3541	3541	0	0
12.8	16	18.5	5624	5624	0	0
12.9	25	28.9	8540	8540	0	0
13	33	38.2	12081	12081	0	0
13.1	43	49.8	15830	15830	0	0
13.2	54	62.5	20204	20204	0	0
13.3	64	74.1	24578	24578	0	0
13.4	75	86.8	28952	28952	0	0
13.5	83	96.0	32909	32909	0	0
13.6	91	105.3	36242	36242	0	0
13.7	96	111.1	38949	38949	0	0
13.8	102	118.0	41241	41241	0	0
13.9	108	125.0	43740	43740	0	0
14	114	131.9	46239	46239	0	0
14.1	114	131.9	47489	47489	0	0
14.2	114	131.9	47489	47489	0	0
14.3	114	131.9	47489	47489	0	0
14.4	110	127.7	46725	46725	0	0
14.5	107	123.4	45198	45198	0	0
14.6	103	119.2	43671	43671	0	0
14.7	101	117.2	42542	42542	0	0
14.8	100	115.1	41813	41813	0	0
14.9	98	113.1	41084	41084	0	0
15	96	111.1	40355	40355	0	0
15.1	94	109.0	39616	39616	0	0
15.2	92	106.9	38866	38866	0	0
15.3	91	104.8	38116	38116	0	0
15.4	89	102.8	37366	37366	0	0
15.5	87	100.7	36617	36617	0	0

2-YEAR FLOOD

15.6	85	98.8	35908	35908	0	0
15.7	84	97.0	35242	35242	0	0
15.8	82	95.1	34575	34575	0	0
15.9	81	93.3	33909	33909	0	0
16	79	91.4	33242	33242	0	0
16.1	78	89.8	32618	32618	0	0
16.2	76	88.2	32034	32034	0	0
16.3	75	86.6	31451	31451	0	0
16.4	73	84.9	30868	30868	0	0
16.5	72	83.3	30285	30285	0	0
16.6	70	81.5	29660	29660	0	0
16.7	69	79.6	28993	28993	0	0
16.8	67	77.8	28327	28327	0	0
16.9	66	75.9	27660	27660	0	0
17	64	74.1	26994	26994	0	0
17.1	63	72.4	26369	26369	0	0
17.2	61	70.8	25786	25786	0	0
17.3	60	69.2	25203	25203	0	0
17.4	58	67.6	24619	24619	0	0
17.5	57	66.0	24036	24036	0	0
17.6	56	64.6	23495	23495	0	0
17.7	55	63.2	22995	22995	0	0
17.8	53	61.8	22495	22495	0	0
17.9	52	60.4	21995	21995	0	0
18	51	59.0	21495	21495	0	0
18.1	50	57.9	21037	21037	0	0
18.2	49	56.7	20620	20620	0	0
18.3	48	55.5	20204	20204	0	0
18.4	47	54.4	19787	19787	0	0
18.5	46	53.2	19371	19371	0	0
18.6	45	52.1	18954	18954	0	0
18.7	44	50.9	18537	18537	0	0
18.8	43	49.8	18121	18121	0	0
18.9	42	48.6	17704	17704	0	0
19	41	47.4	17288	17288	0	0
19.1	40	46.7	16954	16954	0	0
19.2	40	46.1	16705	16705	0	0
19.3	39	45.4	16455	16455	0	0
19.4	39	44.7	16205	16205	0	0
19.5	38	44.0	15955	15955	0	0
19.6	37	43.3	15705	15705	0	0
19.7	37	42.6	15455	15455	0	0
19.8	36	41.9	15205	15205	0	0
19.9	36	41.2	14955	14955	0	0
20	35	40.5	14705	14705	0	0
20.1	35	40.2	14518	14518	0	0
20.2	34	39.8	14393	14393	0	0
20.3	34	39.5	14268	14268	0	0
20.4	34	39.1	14143	14143	0	0
20.5	34	38.8	14018	14018	0	0
20.6	33	38.4	13893	13893	0	0

2-YEAR FLOOD

20.7	33	38.1	13768	13768	0	0
20.8	33	37.7	13643	13643	0	0
20.9	32	37.4	13518	13518	0	0
21	32	37.0	13393	13393	0	0
21.1	32	36.7	13268	13268	0	0
21.2	31	36.3	13143	13143	0	0
21.3	31	36.0	13018	13018	0	0
21.4	31	35.6	12893	12893	0	0
21.5	31	35.3	12768	12768	0	0
21.6	30	34.9	12643	12643	0	0
21.7	30	34.6	12518	12518	0	0
21.8	30	34.3	12393	12393	0	0
21.9	29	33.9	12268	12268	0	0
22	29	33.6	12143	12143	0	0
22.1	28	32.9	11961	11961	0	0
22.2	28	32.2	11721	11721	0	0
22.3	27	31.6	11482	11482	0	0
22.4	27	30.9	11242	11242	0	0
22.5	26	30.2	11003	11003	0	0
22.6	26	29.6	10763	10763	0	0
22.7	25	28.9	10524	10524	0	0
22.8	24	28.2	10284	10284	0	0
22.9	24	27.6	10045	10045	0	0
23	23	26.9	9805	9805	0	0
23.1	23	26.2	9566	9566	0	0
23.2	22	25.6	9326	9326	0	0
23.3	22	24.9	9086	9086	0	0
23.4	21	24.2	8847	8847	0	0
23.5	20	23.6	8607	8607	0	0
23.6	20	22.9	8368	8368	0	0
23.7	19	22.2	8128	8128	0	0
23.8	19	21.6	7889	7889	0	0
23.9	18	20.9	7649	7649	0	0
24	18	20.3	7410	7410	0	0
24.1	17	19.6	7170	7170	0	0
24.2	16	18.9	6931	6931	0	0
24.3	16	18.3	6691	6691	0	0
24.4	15	17.6	6452	6452	0	0
24.5	15	16.9	6212	6212	0	0
24.6	14	16.3	5973	5973	0	0
24.7	13	15.6	5733	5733	0	0
24.8	13	14.9	5494	5494	0	0
24.9	12	14.3	5254	5254	0	0
25	12	13.6	5014	5014	0	0
25.1	11	12.9	4775	4775	0	0
25.2	11	12.3	4535	4535	0	0
25.3	10	11.6	4296	4296	0	0
25.4	9	10.9	4056	4056	0	0
25.5	9	10.3	3817	3817	0	0
25.6	8	9.6	3577	3577	0	0
25.7	8	8.9	3338	3338	0	0

2-YEAR FLOOD

25.8	7	8.3	3098	3098	0	0
25.9	7	7.6	2859	2859	0	0
26	6	6.9	2619	2619	0	0
26.1	6	6.8	2468	2468	0	0
26.2	6	6.6	2406	2406	0	0
26.3	6	6.4	2343	2343	0	0
26.4	5	6.2	2281	2281	0	0
26.5	5	6.1	2218	2218	0	0
26.6	5	5.9	2156	2156	0	0
26.7	5	5.7	2093	2093	0	0
26.8	5	5.6	2031	2031	0	0
26.9	5	5.4	1968	1968	0	0
27	5	5.2	1906	1906	0	0
27.1	4	5.0	1843	1843	0	0
27.2	4	4.9	1781	1781	0	0
27.3	4	4.7	1718	1718	0	0
27.4	4	4.5	1656	1656	0	0
27.5	4	4.3	1593	1593	0	0
27.6	4	4.2	1531	1531	0	0
27.7	3	4.0	1468	1468	0	0
27.8	3	3.8	1406	1406	0	0
27.9	3	3.6	1343	1343	0	0
28	3	3.5	1281	1281	0	0
28.1	3	3.3	1218	1218	0	0
28.2	3	3.1	1156	1156	0	0
28.3	3	3.0	1094	1094	0	0
28.4	2	2.8	1031	1031	0	0
28.5	2	2.6	969	969	0	0
28.6	2	2.4	906	906	0	0
28.7	2	2.3	844	844	0	0
28.8	2	2.1	781	781	0	0
28.9	2	1.9	719	719	0	0
29	2	1.7	656	656	0	0
29.1	1	1.6	594	594	0	0
29.2	1	1.4	531	531	0	0
29.3	1	1.2	469	469	0	0
29.4	1	1.0	406	406	0	0
29.5	1	0.9	344	344	0	0
29.6	1	0.7	281	281	0	0
29.7	0	0.5	219	219	0	0
29.8	0	0.3	156	156	0	0
29.9	0	0.2	94	94	0	0
30	0	0.0	31	31	0	0
			2664391	2664391	0	

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3-YEAR FLOOD

WATER BALANCE FOR AREA ABOVE TEMPORARY DAM

Recurrence interval (RI) = 3 yrs
Precipitation (P) = 3.70 in
Curve number (CN) = 47
Storage (S) = 11.28 in
Initial abstractions (Ia) = 2.26 in
Ia/P = 0.61
Runoff (Q) = 0.16 in
Drainage Area (A) = 9.00 sq mi

Maximums =		167	60229	47880	12349	112665
Column No. 1	2	3	4	5	6	7
Time hrs	Unit Discharge csm/in	Discharge cfs	Inflow Volume cu ft	Pumped Volume cu ft	Incremental Storage Volume cu ft	Cumulative Storage Volume cu ft
12.4	1	1.5	264	264	0	0
12.5	3	4.4	1057	1057	0	0
12.6	6	8.8	2377	2377	0	0
12.7	11	16.1	4491	4491	0	0
12.8	16	23.5	7132	7132	0	0
12.9	25	36.7	10831	10831	0	0
13	33	48.4	15321	15321	0	0
13.1	43	63.1	20076	20076	0	0
13.2	54	79.2	25624	25624	0	0
13.3	64	93.9	31171	31171	0	0
13.4	75	110.1	36718	36718	0	0
13.5	83	121.8	41737	41737	0	0
13.6	91	133.5	45964	45964	0	0
13.7	96	140.9	49398	47880	1518	1518
13.8	102	149.7	52304	47880	4424	5942
13.9	108	158.5	55474	47880	7594	13536
14	114	167.3	58644	47880	10764	24300
14.1	114	167.3	60229	47880	12349	36649
14.2	114	167.3	60229	47880	12349	48997
14.3	114	167.3	60229	47880	12349	61346
14.4	110	161.9	59260	47880	11380	72726
14.5	107	156.5	57323	47880	9443	82169
14.6	103	151.2	55386	47880	7506	89675
14.7	101	148.6	53955	47880	6075	95750
14.8	100	146.0	53030	47880	5150	100900
14.9	98	143.5	52106	47880	4226	105126
15	96	140.9	51181	47880	3301	108427
15.1	94	138.2	50243	47880	2363	110791
15.2	92	135.6	49292	47880	1412	112203
15.3	91	133.0	48342	47880	462	112665
15.4	89	130.3	47391	47880	-489	112175
15.5	87	127.7	46440	47880	-1440	110735

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3-YEAR FLOOD

15.6	85	125.3	45541	47880	-2339	108396
15.7	84	123.0	44696	47880	-3184	105212
15.8	82	120.6	43851	47880	-4029	101183
15.9	81	118.3	43005	47880	-4875	96309
16	79	115.9	42160	47880	-5720	90589
16.1	78	113.9	41368	47880	-6512	84076
16.2	76	111.8	40628	47880	-7252	76824
16.3	75	109.8	39888	47880	-7992	68833
16.4	73	107.7	39149	47880	-8731	60102
16.5	72	105.7	38409	47880	-9471	50631
16.6	70	103.3	37617	47880	-10263	40367
16.7	69	101.0	36771	47880	-11109	29258
16.8	67	98.6	35926	47880	-11954	17304
16.9	66	96.3	35081	47880	-12799	4505
17	64	93.9	34235	38740	-4505	0
17.1	63	91.9	33443	33443	0	0
17.2	61	89.8	32703	32703	0	0
17.3	60	87.8	31964	31964	0	0
17.4	58	85.7	31224	31224	0	0
17.5	57	83.7	30484	30484	0	0
17.6	56	81.9	29797	29797	0	0
17.7	55	80.1	29163	29163	0	0
17.8	53	78.4	28529	28529	0	0
17.9	52	76.6	27895	27895	0	0
18	51	74.8	27261	27261	0	0
18.1	50	73.4	26680	26680	0	0
18.2	49	71.9	26152	26152	0	0
18.3	48	70.4	25624	25624	0	0
18.4	47	69.0	25095	25095	0	0
18.5	46	67.5	24567	24567	0	0
18.6	45	66.0	24039	24039	0	0
18.7	44	64.6	23510	23510	0	0
18.8	43	63.1	22982	22982	0	0
18.9	42	61.6	22454	22454	0	0
19	41	60.2	21925	21925	0	0
19.1	40	59.3	21503	21503	0	0
19.2	40	58.4	21186	21186	0	0
19.3	39	57.5	20869	20869	0	0
19.4	39	56.6	20552	20552	0	0
19.5	38	55.8	20235	20235	0	0
19.6	37	54.9	19918	19918	0	0
19.7	37	54.0	19601	19601	0	0
19.8	36	53.1	19284	19284	0	0
19.9	36	52.2	18967	18967	0	0
20	35	51.4	18650	18650	0	0
20.1	35	50.9	18412	18412	0	0
20.2	34	50.5	18254	18254	0	0
20.3	34	50.0	18095	18095	0	0
20.4	34	49.6	17937	17937	0	0
20.5	34	49.2	17778	17778	0	0
20.6	33	48.7	17620	17620	0	0

3-YEAR FLOOD

20.7	33	48.3	17461	17461	0	0
20.8	33	47.8	17303	17303	0	0
20.9	32	47.4	17144	17144	0	0
21	32	47.0	16986	16986	0	0
21.1	32	46.5	16827	16827	0	0
21.2	31	46.1	16669	16669	0	0
21.3	31	45.6	16510	16510	0	0
21.4	31	45.2	16352	16352	0	0
21.5	31	44.8	16193	16193	0	0
21.6	30	44.3	16035	16035	0	0
21.7	30	43.9	15876	15876	0	0
21.8	30	43.4	15718	15718	0	0
21.9	29	43.0	15559	15559	0	0
22	29	42.6	15401	15401	0	0
22.1	28	41.7	15169	15169	0	0
22.2	28	40.9	14866	14866	0	0
22.3	27	40.0	14562	14562	0	0
22.4	27	39.2	14258	14258	0	0
22.5	26	38.3	13954	13954	0	0
22.6	26	37.5	13651	13651	0	0
22.7	25	36.7	13347	13347	0	0
22.8	24	35.8	13043	13043	0	0
22.9	24	35.0	12739	12739	0	0
23	23	34.1	12435	12435	0	0
23.1	23	33.3	12132	12132	0	0
23.2	22	32.4	11828	11828	0	0
23.3	22	31.6	11524	11524	0	0
23.4	21	30.7	11220	11220	0	0
23.5	20	29.9	10916	10916	0	0
23.6	20	29.1	10613	10613	0	0
23.7	19	28.2	10309	10309	0	0
23.8	19	27.4	10005	10005	0	0
23.9	18	26.5	9701	9701	0	0
24	18	25.7	9398	9398	0	0
24.1	17	24.8	9094	9094	0	0
24.2	16	24.0	8790	8790	0	0
24.3	16	23.2	8486	8486	0	0
24.4	15	22.3	8182	8182	0	0
24.5	15	21.5	7879	7879	0	0
24.6	14	20.6	7575	7575	0	0
24.7	13	19.8	7271	7271	0	0
24.8	13	18.9	6967	6967	0	0
24.9	12	18.1	6663	6663	0	0
25	12	17.2	6360	6360	0	0
25.1	11	16.4	6056	6056	0	0
25.2	11	15.6	5752	5752	0	0
25.3	10	14.7	5448	5448	0	0
25.4	9	13.9	5145	5145	0	0
25.5	9	13.0	4841	4841	0	0
25.6	8	12.2	4537	4537	0	0
25.7	8	11.3	4233	4233	0	0

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3-YEAR FLOOD

25.8	7	10.5	3929	3929	0	0
25.9	7	9.6	3626	3626	0	0
26	6	8.8	3322	3322	0	0
26.1	6	8.6	3130	3130	0	0
26.2	6	8.4	3051	3051	0	0
26.3	6	8.1	2972	2972	0	0
26.4	5	7.9	2893	2893	0	0
26.5	5	7.7	2813	2813	0	0
26.6	5	7.5	2734	2734	0	0
26.7	5	7.3	2655	2655	0	0
26.8	5	7.0	2576	2576	0	0
26.9	5	6.8	2496	2496	0	0
27	5	6.6	2417	2417	0	0
27.1	4	6.4	2338	2338	0	0
27.2	4	6.2	2259	2259	0	0
27.3	4	5.9	2179	2179	0	0
27.4	4	5.7	2100	2100	0	0
27.5	4	5.5	2021	2021	0	0
27.6	4	5.3	1942	1942	0	0
27.7	3	5.1	1862	1862	0	0
27.8	3	4.8	1783	1783	0	0
27.9	3	4.6	1704	1704	0	0
28	3	4.4	1625	1625	0	0
28.1	3	4.2	1545	1545	0	0
28.2	3	4.0	1466	1466	0	0
28.3	3	3.7	1387	1387	0	0
28.4	2	3.5	1308	1308	0	0
28.5	2	3.3	1228	1228	0	0
28.6	2	3.1	1149	1149	0	0
28.7	2	2.9	1070	1070	0	0
28.8	2	2.6	991	991	0	0
28.9	2	2.4	911	911	0	0
29	2	2.2	832	832	0	0
29.1	1	2.0	753	753	0	0
29.2	1	1.8	674	674	0	0
29.3	1	1.5	594	594	0	0
29.4	1	1.3	515	515	0	0
29.5	1	1.1	436	436	0	0
29.6	1	0.9	357	357	0	0
29.7	0	0.7	277	277	0	0
29.8	0	0.4	198	198	0	0
29.9	0	0.2	119	119	0	0
30	0	0.0	40	40	0	0
			3379151	3379151	0	

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4-YEAR FLOOD

WATER BALANCE FOR AREA ABOVE TEMPORARY DAM

Recurrence interval (RI) = 4 yrs
Precipitation (P) = 3.90 in
Curve number (CN) = 46
Storage (S) = 11.74 in
Initial abstractions (Ia) = 2.35 in
Ia/P = 0.60
Runoff (Q) = 0.18 in
Drainage Area (A) = 9.00 sq mi

Maximums =		186	66938	47880	19058	232995
Column No. 1	2	3	4	5	6	7
Time hrs	Unit Discharge csm/in	Discharge cfs	Inflow Volume cu ft	Pumped Volume cu ft	Incremental Storage Volume cu ft	Cumulative Storage Volume cu ft
12.4	1	1.6	294	294	0	0
12.5	3	4.9	1174	1174	0	0
12.6	6	9.8	2642	2642	0	0
12.7	11	17.9	4991	4991	0	0
12.8	16	26.1	7927	7927	0	0
12.9	25	40.8	12037	12037	0	0
13	33	53.8	17028	17028	0	0
13.1	43	70.1	22313	22313	0	0
13.2	54	88.1	28478	28478	0	0
13.3	64	104.4	34643	34643	0	0
13.4	75	122.3	40809	40809	0	0
13.5	83	135.4	46387	46387	0	0
13.6	91	148.4	51084	47880	3204	3204
13.7	96	156.6	54901	47880	7021	10225
13.8	102	166.4	58130	47880	10250	20475
13.9	108	176.2	61653	47880	13773	34248
14	114	185.9	65176	47880	17296	51545
14.1	114	185.9	66938	47880	19058	70602
14.2	114	185.9	66938	47880	19058	89660
14.3	114	185.9	66938	47880	19058	108718
14.4	110	180.0	65861	47880	17981	126699
14.5	107	174.0	63708	47880	15828	142528
14.6	103	168.0	61555	47880	13675	156203
14.7	101	165.1	59965	47880	12085	168288
14.8	100	162.3	58938	47880	11058	179346
14.9	98	159.4	57910	47880	10030	189376
15	96	156.6	56882	47880	9002	198378
15.1	94	153.6	55840	47880	7960	206338
15.2	92	150.7	54783	47880	6903	213242
15.3	91	147.8	53726	47880	5846	219088
15.4	89	144.8	52669	47880	4789	223877
15.5	87	141.9	51613	47880	3733	227610

 CanonieEnvironmental

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4-YEAR FLOOD

15.6	85	139.3	50614	47880	2734	230344
15.7	84	136.7	49675	47880	1795	232139
15.8	82	134.1	48735	47880	855	232995
15.9	81	131.5	47796	47880	-84	232911
16	79	128.9	46856	47880	-1024	231887
16.1	78	126.6	45976	47880	-1904	229983
16.2	76	124.3	45154	47880	-2726	227256
16.3	75	122.0	44332	47880	-3548	223708
16.4	73	119.7	43510	47880	-4370	219337
16.5	72	117.4	42688	47880	-5192	214145
16.6	70	114.8	41807	47880	-6073	208072
16.7	69	112.2	40867	47880	-7013	201059
16.8	67	109.6	39928	47880	-7952	193107
16.9	66	107.0	38988	47880	-8892	184215
17	64	104.4	38049	47880	-9831	174384
17.1	63	102.1	37168	47880	-10712	163672
17.2	61	99.8	36346	47880	-11534	152138
17.3	60	97.5	35524	47880	-12356	139782
17.4	58	95.3	34702	47880	-13178	126604
17.5	57	93.0	33880	47880	-14000	112604
17.6	56	91.0	33117	47880	-14763	97841
17.7	55	89.1	32412	47880	-15468	82373
17.8	53	87.1	31707	47880	-16173	66200
17.9	52	85.1	31003	47880	-16877	49323
18	51	83.2	30298	47880	-17582	31741
18.1	50	81.6	29652	47880	-18228	13513
18.2	49	79.9	29065	42578	-13513	0
18.3	48	78.3	28478	28478	0	0
18.4	47	76.7	27891	27891	0	0
18.5	46	75.0	27304	27304	0	0
18.6	45	73.4	26716	26716	0	0
18.7	44	71.8	26129	26129	0	0
18.8	43	70.1	25542	25542	0	0
18.9	42	68.5	24955	24955	0	0
19	41	66.9	24368	24368	0	0
19.1	40	65.9	23898	23898	0	0
19.2	40	64.9	23546	23546	0	0
19.3	39	63.9	23193	23193	0	0
19.4	39	63.0	22841	22841	0	0
19.5	38	62.0	22489	22489	0	0
19.6	37	61.0	22136	22136	0	0
19.7	37	60.0	21784	21784	0	0
19.8	36	59.0	21432	21432	0	0
19.9	36	58.1	21080	21080	0	0
20	35	57.1	20727	20727	0	0
20.1	35	56.6	20463	20463	0	0
20.2	34	56.1	20287	20287	0	0
20.3	34	55.6	20111	20111	0	0
20.4	34	55.1	19935	19935	0	0
20.5	34	54.6	19758	19758	0	0
20.6	33	54.2	19582	19582	0	0

4-YEAR FLOOD

20.7	33	53.7	19406	19406	0	0
20.8	33	53.2	19230	19230	0	0
20.9	32	52.7	19054	19054	0	0
21	32	52.2	18878	18878	0	0
21.1	32	51.7	18701	18701	0	0
21.2	31	51.2	18525	18525	0	0
21.3	31	50.7	18349	18349	0	0
21.4	31	50.2	18173	18173	0	0
21.5	31	49.7	17997	17997	0	0
21.6	30	49.3	17821	17821	0	0
21.7	30	48.8	17645	17645	0	0
21.8	30	48.3	17468	17468	0	0
21.9	29	47.8	17292	17292	0	0
22	29	47.3	17116	17116	0	0
22.1	28	46.4	16859	16859	0	0
22.2	28	45.4	16522	16522	0	0
22.3	27	44.5	16184	16184	0	0
22.4	27	43.5	15846	15846	0	0
22.5	26	42.6	15509	15509	0	0
22.6	26	41.7	15171	15171	0	0
22.7	25	40.7	14833	14833	0	0
22.8	24	39.8	14496	14496	0	0
22.9	24	38.9	14158	14158	0	0
23	23	37.9	13821	13821	0	0
23.1	23	37.0	13483	13483	0	0
23.2	22	36.0	13145	13145	0	0
23.3	22	35.1	12808	12808	0	0
23.4	21	34.2	12470	12470	0	0
23.5	20	33.2	12132	12132	0	0
23.6	20	32.3	11795	11795	0	0
23.7	19	31.4	11457	11457	0	0
23.8	19	30.4	11120	11120	0	0
23.9	18	29.5	10782	10782	0	0
24	18	28.5	10444	10444	0	0
24.1	17	27.6	10107	10107	0	0
24.2	16	26.7	9769	9769	0	0
24.3	16	25.7	9431	9431	0	0
24.4	15	24.8	9094	9094	0	0
24.5	15	23.9	8756	8756	0	0
24.6	14	22.9	8419	8419	0	0
24.7	13	22.0	8081	8081	0	0
24.8	13	21.0	7743	7743	0	0
24.9	12	20.1	7406	7406	0	0
25	12	19.2	7068	7068	0	0
25.1	11	18.2	6730	6730	0	0
25.2	11	17.3	6393	6393	0	0
25.3	10	16.4	6055	6055	0	0
25.4	9	15.4	5718	5718	0	0
25.5	9	14.5	5380	5380	0	0
25.6	8	13.5	5042	5042	0	0
25.7	8	12.6	4705	4705	0	0

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4-YEAR FLOOD

25.8	7	11.7	4367	4367	0	0
25.9	7	10.7	4029	4029	0	0
26	6	9.8	3692	3692	0	0
26.1	6	9.5	3479	3479	0	0
26.2	6	9.3	3391	3391	0	0
26.3	6	9.1	3303	3303	0	0
26.4	5	8.8	3215	3215	0	0
26.5	5	8.6	3127	3127	0	0
26.6	5	8.3	3039	3039	0	0
26.7	5	8.1	2951	2951	0	0
26.8	5	7.8	2862	2862	0	0
26.9	5	7.6	2774	2774	0	0
27	5	7.3	2686	2686	0	0
27.1	4	7.1	2598	2598	0	0
27.2	4	6.9	2510	2510	0	0
27.3	4	6.6	2422	2422	0	0
27.4	4	6.4	2334	2334	0	0
27.5	4	6.1	2246	2246	0	0
27.6	4	5.9	2158	2158	0	0
27.7	3	5.6	2070	2070	0	0
27.8	3	5.4	1982	1982	0	0
27.9	3	5.1	1894	1894	0	0
28	3	4.9	1806	1806	0	0
28.1	3	4.6	1717	1717	0	0
28.2	3	4.4	1629	1629	0	0
28.3	3	4.2	1541	1541	0	0
28.4	2	3.9	1453	1453	0	0
28.5	2	3.7	1365	1365	0	0
28.6	2	3.4	1277	1277	0	0
28.7	2	3.2	1189	1189	0	0
28.8	2	2.9	1101	1101	0	0
28.9	2	2.7	1013	1013	0	0
29	2	2.4	925	925	0	0
29.1	1	2.2	837	837	0	0
29.2	1	2.0	749	749	0	0
29.3	1	1.7	661	661	0	0
29.4	1	1.5	572	572	0	0
29.5	1	1.2	484	484	0	0
29.6	1	1.0	396	396	0	0
29.7	0	0.7	308	308	0	0
29.8	0	0.5	220	220	0	0
29.9	0	0.2	132	132	0	0
30	0	0.0	44	44	0	0
			3755562	3755562	0	

5-YEAR FLOOD

WATER BALANCE FOR AREA ABOVE TEMPORARY DAM

Recurrence interval (RI) = 5 yrs
Precipitation (P) = 4.10 in
Curve number (CN) = 45
Storage (S) = 12.22 in
Initial abstractions (Ia) = 2.44 in
Ia/P = 0.60
Runoff (Q) = 0.20 in
Drainage Area (A) = 9.00 sq mi

Maximums =		203	72986	47880	25106	367742
Column No. 1	2	3	4	5	6	7
Time hrs	Unit Discharge csm/in	Discharge cfs	Inflow Volume cu ft	Pumped Volume cu ft	Incremental Storage Volume cu ft	Cumulative Storage Volume cu ft
12.4	1	1.8	320	320	0	0
12.5	3	5.3	1280	1280	0	0
12.6	6	10.7	2881	2881	0	0
12.7	11	19.6	5442	5442	0	0
12.8	16	28.5	8643	8643	0	0
12.9	25	44.5	13125	13125	0	0
13	33	58.7	18566	18566	0	0
13.1	43	76.5	24329	24329	0	0
13.2	54	96.0	31051	31051	0	0
13.3	64	113.8	37773	37773	0	0
13.4	75	133.4	44496	44496	0	0
13.5	83	147.6	50578	47880	2698	2698
13.6	91	161.8	55699	47880	7819	10517
13.7	96	170.7	59861	47880	11981	22498
13.8	102	181.4	63382	47880	15502	38000
13.9	108	192.1	67224	47880	19344	57344
14	114	202.7	71065	47880	23185	80529
14.1	114	202.7	72986	47880	25106	105634
14.2	114	202.7	72986	47880	25106	130740
14.3	114	202.7	72986	47880	25106	155845
14.4	110	196.2	71812	47880	23932	179777
14.5	107	189.7	69464	47880	21584	201361
14.6	103	183.2	67117	47880	19237	220598
14.7	101	180.1	65383	47880	17503	238101
14.8	100	177.0	64262	47880	16382	254483
14.9	98	173.8	63142	47880	15262	269745
15	96	170.7	62022	47880	14142	283887
15.1	94	167.5	60885	47880	13005	296892
15.2	92	164.3	59733	47880	11853	308745
15.3	91	161.1	58580	47880	10700	319446
15.4	89	157.9	57428	47880	9548	328994
15.5	87	154.7	56276	47880	8396	337389

5-YEAR FLOOD

15.6	85	151.9	55187	47880	7307	344697
15.7	84	149.0	54163	47880	6283	350980
15.8	82	146.2	53139	47880	5259	356238
15.9	81	143.3	52114	47880	4234	360473
16	79	140.5	51090	47880	3210	363682
16.1	78	138.0	50130	47880	2250	365932
16.2	76	135.5	49233	47880	1353	367285
16.3	75	133.0	48337	47880	457	367742
16.4	73	130.5	47441	47880	-439	367303
16.5	72	128.0	46544	47880	-1336	365967
16.6	70	125.2	45584	47880	-2296	363671
16.7	69	122.4	44560	47880	-3320	360350
16.8	67	119.5	43535	47880	-4345	356006
16.9	66	116.7	42511	47880	-5369	350636
17	64	113.8	41487	47880	-6393	344243
17.1	63	111.3	40526	47880	-7354	336889
17.2	61	108.8	39630	47880	-8250	328639
17.3	60	106.3	38734	47880	-9146	319493
17.4	58	103.9	37837	47880	-10043	309450
17.5	57	101.4	36941	47880	-10939	298511
17.6	56	99.2	36109	47880	-11771	286739
17.7	55	97.1	35340	47880	-12540	274200
17.8	53	95.0	34572	47880	-13308	260892
17.9	52	92.8	33804	47880	-14076	246816
18	51	90.7	33036	47880	-14844	231971
18.1	50	88.9	32331	47880	-15549	216422
18.2	49	87.1	31691	47880	-16189	200234
18.3	48	85.4	31051	47880	-16829	183404
18.4	47	83.6	30411	47880	-17469	165935
18.5	46	81.8	29770	47880	-18110	147825
18.6	45	80.0	29130	47880	-18750	129076
18.7	44	78.2	28490	47880	-19390	109686
18.8	43	76.5	27850	47880	-20030	89655
18.9	42	74.7	27210	47880	-20670	68985
19	41	72.9	26569	47880	-21311	47674
19.1	40	71.8	26057	47880	-21823	25851
19.2	40	70.8	25673	47880	-22207	3644
19.3	39	69.7	25289	28933	-3644	0
19.4	39	68.6	24905	24905	0	0
19.5	38	67.6	24521	24521	0	0
19.6	37	66.5	24136	24136	0	0
19.7	37	65.4	23752	23752	0	0
19.8	36	64.4	23368	23368	0	0
19.9	36	63.3	22984	22984	0	0
20	35	62.2	22600	22600	0	0
20.1	35	61.7	22312	22312	0	0
20.2	34	61.2	22120	22120	0	0
20.3	34	60.6	21928	21928	0	0
20.4	34	60.1	21736	21736	0	0
20.5	34	59.6	21544	21544	0	0
20.6	33	59.0	21351	21351	0	0

5-YEAR FLOOD

20.7	33	58.5	21159	21159	0	0
20.8	33	58.0	20967	20967	0	0
20.9	32	57.4	20775	20775	0	0
21	32	56.9	20583	20583	0	0
21.1	32	56.4	20391	20391	0	0
21.2	31	55.8	20199	20199	0	0
21.3	31	55.3	20007	20007	0	0
21.4	31	54.8	19815	19815	0	0
21.5	31	54.2	19623	19623	0	0
21.6	30	53.7	19431	19431	0	0
21.7	30	53.2	19239	19239	0	0
21.8	30	52.6	19047	19047	0	0
21.9	29	52.1	18855	18855	0	0
22	29	51.6	18663	18663	0	0
22.1	28	50.6	18382	18382	0	0
22.2	28	49.5	18014	18014	0	0
22.3	27	48.5	17646	17646	0	0
22.4	27	47.5	17278	17278	0	0
22.5	26	46.5	16910	16910	0	0
22.6	26	45.4	16542	16542	0	0
22.7	25	44.4	16174	16174	0	0
22.8	24	43.4	15806	15806	0	0
22.9	24	42.4	15437	15437	0	0
23	23	41.3	15069	15069	0	0
23.1	23	40.3	14701	14701	0	0
23.2	22	39.3	14333	14333	0	0
23.3	22	38.3	13965	13965	0	0
23.4	21	37.3	13597	13597	0	0
23.5	20	36.2	13229	13229	0	0
23.6	20	35.2	12860	12860	0	0
23.7	19	34.2	12492	12492	0	0
23.8	19	33.2	12124	12124	0	0
23.9	18	32.1	11756	11756	0	0
24	18	31.1	11388	11388	0	0
24.1	17	30.1	11020	11020	0	0
24.2	16	29.1	10652	10652	0	0
24.3	16	28.1	10284	10284	0	0
24.4	15	27.0	9915	9915	0	0
24.5	15	26.0	9547	9547	0	0
24.6	14	25.0	9179	9179	0	0
24.7	13	24.0	8811	8811	0	0
24.8	13	22.9	8443	8443	0	0
24.9	12	21.9	8075	8075	0	0
25	12	20.9	7707	7707	0	0
25.1	11	19.9	7339	7339	0	0
25.2	11	18.9	6970	6970	0	0
25.3	10	17.8	6602	6602	0	0
25.4	9	16.8	6234	6234	0	0
25.5	9	15.8	5866	5866	0	0
25.6	8	14.8	5498	5498	0	0
25.7	8	13.7	5130	5130	0	0

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5-YEAR FLOOD

25.8	7	12.7	4762	4762	0	0
25.9	7	11.7	4394	4394	0	0
26	6	10.7	4025	4025	0	0
26.1	6	10.4	3793	3793	0	0
26.2	6	10.1	3697	3697	0	0
26.3	6	9.9	3601	3601	0	0
26.4	5	9.6	3505	3505	0	0
26.5	5	9.3	3409	3409	0	0
26.6	5	9.1	3313	3313	0	0
26.7	5	8.8	3217	3217	0	0
26.8	5	8.5	3121	3121	0	0
26.9	5	8.3	3025	3025	0	0
27	5	8.0	2929	2929	0	0
27.1	4	7.7	2833	2833	0	0
27.2	4	7.5	2737	2737	0	0
27.3	4	7.2	2641	2641	0	0
27.4	4	6.9	2545	2545	0	0
27.5	4	6.7	2449	2449	0	0
27.6	4	6.4	2353	2353	0	0
27.7	3	6.1	2257	2257	0	0
27.8	3	5.9	2161	2161	0	0
27.9	3	5.6	2065	2065	0	0
28	3	5.3	1969	1969	0	0
28.1	3	5.1	1873	1873	0	0
28.2	3	4.8	1777	1777	0	0
28.3	3	4.5	1681	1681	0	0
28.4	2	4.3	1585	1585	0	0
28.5	2	4.0	1489	1489	0	0
28.6	2	3.7	1392	1392	0	0
28.7	2	3.5	1296	1296	0	0
28.8	2	3.2	1200	1200	0	0
28.9	2	2.9	1104	1104	0	0
29	2	2.7	1008	1008	0	0
29.1	1	2.4	912	912	0	0
29.2	1	2.1	816	816	0	0
29.3	1	1.9	720	720	0	0
29.4	1	1.6	624	624	0	0
29.5	1	1.3	528	528	0	0
29.6	1	1.1	432	432	0	0
29.7	0	0.8	336	336	0	0
29.8	0	0.5	240	240	0	0
29.9	0	0.3	144	144	0	0
30	0	0.0	48	48	0	0
			4094872	4094872	0	

5-YEAR FLOOD

WATER BALANCE FOR AREA ABOVE TEMPORARY DAM INCLUDING OUTFLOW OVER DAM

Recurrence interval (RI) =	5	yrs
Precipitation (P) =	4.10	in
Curve number (CN) =	45	
Storage (S) =	12.22	in
Initial abstractions (Ia) =	2.44	in
Ia/P =	0.60	
Runoff (Q) =	0.20	in
Drainage Area (A) =	9.00	sq mi

Column No. 1	2	3	4	5	6	7	8	9.0	10.0	11	12
Maximums =		203	73068	47880	25188	235544	235350	162.7	38.4	13824	221720
Time hrs	Unit Discharge csm/in	Discharge cfs	Inflow Volume cu ft	Pumped Volume cu ft	Incremental Storage Volume cu ft	Cumulative Storage Volume cu ft	Average Storage Volume cu ft	Average WSEL ft	Outflow Rate cfs	Outflow Volume cu ft	Final Storage Volume cu ft
12.4	1	1.8	320	320	0	0	0	156.0	0.0	0	0
12.5	3	5.3	1282	1282	0	0	0	156.0	0.0	0	0
12.6	6	10.7	2884	2884	0	0	0	156.0	0.0	0	0
12.7	11	19.6	5448	5448	0	0	0	156.0	0.0	0	0
12.8	16	28.5	8653	8653	0	0	0	156.0	0.0	0	0
12.9	25	44.5	13139	13139	0	0	0	156.0	0.0	0	0
13	33	58.8	18588	18588	0	0	0	156.0	0.0	0	0
13.1	43	76.6	24356	24356	0	0	0	156.0	0.0	0	0
13.2	54	96.1	31086	31086	0	0	0	156.0	0.0	0	0
13.3	64	113.9	37816	37816	0	0	0	156.0	0.0	0	0
13.4	75	133.5	44546	44546	0	0	0	156.0	0.0	0	0
13.5	83	147.8	50635	47880	2755	2755	1378	156.0	0.0	0	2755
13.6	91	162.0	55763	47880	7883	10638	6697	157.0	0.0	0	10638
13.7	96	170.9	59929	47880	12049	22687	16662	158.2	0.0	0	22687
13.8	102	181.6	63454	47880	15574	38261	30474	159.3	0.0	0	38261
13.9	108	192.3	67300	47880	19420	57681	47971	159.7	0.0	0	57681
14	114	203.0	71146	47880	23266	80946	69314	160.4	0.0	0	80946
14.1	114	203.0	73068	47880	25188	106135	93541	161.0	0.0	0	106135

14.2	114	203.0	73068	47880	25188	131323	118729	161.4	0.0	0	131323
14.3	114	203.0	73068	47880	25188	156512	143917	161.8	0.0	0	156512
14.4	110.33333	196.4	71893	47880	24013	180525	168518	162.0	0.0	0	180525
14.5	106.66667	189.9	69543	47880	21663	202188	191357	162.2	8.8	3168	199020
14.6	103	183.4	67193	47880	19313	218333	210261	162.5	22.0	7920	210413
14.7	101.25	180.3	65457	47880	17577	227990	223162	162.6	30.0	10800	217190
14.8	99.5	177.2	64335	47880	16455	233646	230818	162.7	38.4	13824	219822
14.9	97.75	174.0	63214	47880	15334	235156	234401	162.7	38.4	13824	221332
15	96	170.9	62092	47880	14212	235544	235350	162.7	38.4	13824	221720
15.1	94.2	167.7	60954	47880	13074	234794	235169	162.7	38.4	13824	220970
15.2	92.4	164.5	59801	47880	11921	232891	233843	162.7	38.4	13824	219067
15.3	90.6	161.3	58647	47880	10767	229834	231363	162.7	38.4	13824	216010
15.4	88.8	158.1	57493	47880	9613	225623	227729	162.6	30.0	10800	214823
15.5	87	154.9	56340	47880	8460	223283	224453	162.6	30.0	10800	212483
15.6	85.4	152.0	55250	47880	7370	219853	221568	162.6	30.0	10800	209053
15.7	83.8	149.2	54224	47880	6344	215397	217625	162.5	22.0	7920	207477
15.8	82.2	146.4	53199	47880	5319	212796	214097	162.5	22.0	7920	204876
15.9	80.6	143.5	52173	47880	4293	209170	210983	162.5	22.0	7920	201250
16	79	140.7	51148	47880	3268	204518	206844	162.4	17.6	6336	198182
16.1	77.6	138.2	50186	47880	2306	200488	202503	162.3	13.2	4752	195736
16.2	76.2	135.7	49289	47880	1409	197145	198817	162.3	13.2	4752	192393
16.3	74.8	133.2	48392	47880	512	192905	195025	162.3	13.2	4752	188153
16.4	73.4	130.7	47494	47880	-386	187768	190336	162.2	8.8	3168	184600
16.5	72	128.2	46597	47880	-1283	183317	185542	162.1	4.4	1584	181733
16.6	70.4	125.3	45636	47880	-2244	179488	181403	162.1	4.4	1584	177904
16.7	68.8	122.5	44610	47880	-3270	174635	177061	162.1	4.4	1584	173051
16.8	67.2	119.6	43585	47880	-4295	168755	171695	162.0	0.0	0	168755
16.9	65.6	116.8	42559	47880	-5321	163434	166095	162.0	0.0	0	163434
17	64	113.9	41534	47880	-6346	157088	160261	162.0	0.0	0	157088
17.1	62.6	111.5	40572	47880	-7308	149780	153434	NA	0.0	0	149780
17.2	61.2	109.0	39675	47880	-8205	141575	145678	NA	0.0	0	141575
17.3	59.8	106.5	38778	47880	-9102	132473	137024	NA	0.0	0	132473
17.4	58.4	104.0	37880	47880	-10000	122473	127473	NA	0.0	0	122473
17.5	57	101.5	36983	47880	-10897	111576	117024	NA	0.0	0	111576
17.6	55.8	99.3	36150	47880	-11730	99845	105711	NA	0.0	0	99845
17.7	54.6	97.2	35381	47880	-12499	87346	93596	NA	0.0	0	87346
17.8	53.4	95.1	34611	47880	-13269	74077	80712	NA	0.0	0	74077

Canonie Environmental

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5-YEAR FLOOD

17.9	52.2	92.9	33842	47880	-14038	60039	67058	NA	0.0	0	60039
18	51	90.8	33073	47880	-14807	45233	52636	NA	0.0	0	45233
18.1	50	89.0	32368	47880	-15512	29721	37477	NA	0.0	0	29721
18.2	49	87.2	31727	47880	-16153	13568	21644	NA	0.0	0	13568
18.3	48	85.5	31086	44654	-13568	0	6784	NA	0.0	0	0
18.4	47	83.7	30445	30445	0	0	0	NA	0.0	0	0
18.5	46	81.9	29804	29804	0	0	0	NA	0.0	0	0
18.6	45	80.1	29163	29163	0	0	0	NA	0.0	0	0
18.7	44	78.3	28522	28522	0	0	0	NA	0.0	0	0
18.8	43	76.6	27881	27881	0	0	0	NA	0.0	0	0
18.9	42	74.8	27240	27240	0	0	0	NA	0.0	0	0
19	41	73.0	26599	26599	0	0	0	NA	0.0	0	0
19.1	40.4	71.9	26087	26087	0	0	0	NA	0.0	0	0
19.2	39.8	70.9	25702	25702	0	0	0	NA	0.0	0	0
19.3	39.2	69.8	25318	25318	0	0	0	NA	0.0	0	0
19.4	38.6	68.7	24933	24933	0	0	0	NA	0.0	0	0
19.5	38	67.7	24548	24548	0	0	0	NA	0.0	0	0
19.6	37.4	66.6	24164	24164	0	0	0	NA	0.0	0	0
19.7	36.8	65.5	23779	23779	0	0	0	NA	0.0	0	0
19.8	36.2	64.5	23395	23395	0	0	0	NA	0.0	0	0
19.9	35.6	63.4	23010	23010	0	0	0	NA	0.0	0	0
20	35	62.3	22626	22626	0	0	0	NA	0.0	0	0
20.1	34.7	61.8	22337	22337	0	0	0	NA	0.0	0	0
20.2	34.4	61.2	22145	22145	0	0	0	NA	0.0	0	0
20.3	34.1	60.7	21953	21953	0	0	0	NA	0.0	0	0
20.4	33.8	60.2	21760	21760	0	0	0	NA	0.0	0	0
20.5	33.5	59.6	21568	21568	0	0	0	NA	0.0	0	0
20.6	33.2	59.1	21376	21376	0	0	0	NA	0.0	0	0
20.7	32.9	58.6	21183	21183	0	0	0	NA	0.0	0	0
20.8	32.6	58.0	20991	20991	0	0	0	NA	0.0	0	0
20.9	32.3	57.5	20799	20799	0	0	0	NA	0.0	0	0
21	32	57.0	20607	20607	0	0	0	NA	0.0	0	0
21.1	31.7	56.4	20414	20414	0	0	0	NA	0.0	0	0
21.2	31.4	55.9	20222	20222	0	0	0	NA	0.0	0	0
21.3	31.1	55.4	20030	20030	0	0	0	NA	0.0	0	0
21.4	30.8	54.8	19837	19837	0	0	0	NA	0.0	0	0
21.5	30.5	54.3	19645	19645	0	0	0	NA	0.0	0	0

21.6	30.2	53.8	19453	19453	0	0	0	NA	0.0	0	0
21.7	29.9	53.2	19261	19261	0	0	0	NA	0.0	0	0
21.8	29.6	52.7	19068	19068	0	0	0	NA	0.0	0	0
21.9	29.3	52.2	18876	18876	0	0	0	NA	0.0	0	0
22	29	51.6	18684	18684	0	0	0	NA	0.0	0	0
22.1	28.425	50.6	18403	18403	0	0	0	NA	0.0	0	0
22.2	27.85	49.6	18035	18035	0	0	0	NA	0.0	0	0
22.3	27.275	48.6	17666	17666	0	0	0	NA	0.0	0	0
22.4	26.7	47.5	17298	17298	0	0	0	NA	0.0	0	0
22.5	26.125	46.5	16929	16929	0	0	0	NA	0.0	0	0
22.6	25.55	45.5	16561	16561	0	0	0	NA	0.0	0	0
22.7	24.975	44.5	16192	16192	0	0	0	NA	0.0	0	0
22.8	24.4	43.4	15823	15823	0	0	0	NA	0.0	0	0
22.9	23.825	42.4	15455	15455	0	0	0	NA	0.0	0	0
23	23.25	41.4	15086	15086	0	0	0	NA	0.0	0	0
23.1	22.675	40.4	14718	14718	0	0	0	NA	0.0	0	0
23.2	22.1	39.3	14349	14349	0	0	0	NA	0.0	0	0
23.3	21.525	38.3	13981	13981	0	0	0	NA	0.0	0	0
23.4	20.95	37.3	13612	13612	0	0	0	NA	0.0	0	0
23.5	20.375	36.3	13244	13244	0	0	0	NA	0.0	0	0
23.6	19.8	35.3	12875	12875	0	0	0	NA	0.0	0	0
23.7	19.225	34.2	12507	12507	0	0	0	NA	0.0	0	0
23.8	18.65	33.2	12138	12138	0	0	0	NA	0.0	0	0
23.9	18.075	32.2	11769	11769	0	0	0	NA	0.0	0	0
24	17.5	31.2	11401	11401	0	0	0	NA	0.0	0	0
24.1	16.925	30.1	11032	11032	0	0	0	NA	0.0	0	0
24.2	16.35	29.1	10664	10664	0	0	0	NA	0.0	0	0
24.3	15.775	28.1	10295	10295	0	0	0	NA	0.0	0	0
24.4	15.2	27.1	9927	9927	0	0	0	NA	0.0	0	0
24.5	14.625	26.0	9558	9558	0	0	0	NA	0.0	0	0
24.6	14.05	25.0	9190	9190	0	0	0	NA	0.0	0	0
24.7	13.475	24.0	8821	8821	0	0	0	NA	0.0	0	0
24.8	12.9	23.0	8453	8453	0	0	0	NA	0.0	0	0
24.9	12.325	21.9	8084	8084	0	0	0	NA	0.0	0	0
25	11.75	20.9	7715	7715	0	0	0	NA	0.0	0	0
25.1	11.175	19.9	7347	7347	0	0	0	NA	0.0	0	0
25.2	10.6	18.9	6978	6978	0	0	0	NA	0.0	0	0

5-YEAR FLOOD

25.3	10.025	17.8	6610	6610	0	0	0	NA	0.0	0	0
25.4	9.45	16.8	6241	6241	0	0	0	NA	0.0	0	0
25.5	8.875	15.8	5873	5873	0	0	0	NA	0.0	0	0
25.6	8.3	14.8	5504	5504	0	0	0	NA	0.0	0	0
25.7	7.725	13.8	5136	5136	0	0	0	NA	0.0	0	0
25.8	7.15	12.7	4767	4767	0	0	0	NA	0.0	0	0
25.9	6.575	11.7	4399	4399	0	0	0	NA	0.0	0	0
26	6	10.7	4030	4030	0	0	0	NA	0.0	0	0
26.1	5.85	10.4	3798	3798	0	0	0	NA	0.0	0	0
26.2	5.7	10.1	3701	3701	0	0	0	NA	0.0	0	0
26.3	5.55	9.9	3605	3605	0	0	0	NA	0.0	0	0
26.4	5.4	9.6	3509	3509	0	0	0	NA	0.0	0	0
26.5	5.25	9.3	3413	3413	0	0	0	NA	0.0	0	0
26.6	5.1	9.1	3317	3317	0	0	0	NA	0.0	0	0
26.7	4.95	8.8	3221	3221	0	0	0	NA	0.0	0	0
26.8	4.8	8.5	3125	3125	0	0	0	NA	0.0	0	0
26.9	4.65	8.3	3028	3028	0	0	0	NA	0.0	0	0
27	4.5	8.0	2932	2932	0	0	0	NA	0.0	0	0
27.1	4.35	7.7	2836	2836	0	0	0	NA	0.0	0	0
27.2	4.2	7.5	2740	2740	0	0	0	NA	0.0	0	0
27.3	4.05	7.2	2644	2644	0	0	0	NA	0.0	0	0
27.4	3.9	6.9	2548	2548	0	0	0	NA	0.0	0	0
27.5	3.75	6.7	2452	2452	0	0	0	NA	0.0	0	0
27.6	3.6	6.4	2355	2355	0	0	0	NA	0.0	0	0
27.7	3.45	6.1	2259	2259	0	0	0	NA	0.0	0	0
27.8	3.3	5.9	2163	2163	0	0	0	NA	0.0	0	0
27.9	3.15	5.6	2067	2067	0	0	0	NA	0.0	0	0
28	3	5.3	1971	1971	0	0	0	NA	0.0	0	0
28.1	2.85	5.1	1875	1875	0	0	0	NA	0.0	0	0
28.2	2.7	4.8	1779	1779	0	0	0	NA	0.0	0	0
28.3	2.55	4.5	1682	1682	0	0	0	NA	0.0	0	0
28.4	2.4	4.3	1586	1586	0	0	0	NA	0.0	0	0
28.5	2.25	4.0	1490	1490	0	0	0	NA	0.0	0	0
28.6	2.1	3.7	1394	1394	0	0	0	NA	0.0	0	0
28.7	1.95	3.5	1298	1298	0	0	0	NA	0.0	0	0
28.8	1.8	3.2	1202	1202	0	0	0	NA	0.0	0	0
28.9	1.65	2.9	1106	1106	0	0	0	NA	0.0	0	0

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5-YEAR FLOOD

29	1.5	2.7	1009	1009	0	0	0	NA	0.0	0	0
29.1	1.35	2.4	913	913	0	0	0	NA	0.0	0	0
29.2	1.2	2.1	817	817	0	0	0	NA	0.0	0	0
29.3	1.05	1.9	721	721	0	0	0	NA	0.0	0	0
29.4	0.9	1.6	625	625	0	0	0	NA	0.0	0	0
29.5	0.75	1.3	529	529	0	0	0	NA	0.0	0	0
29.6	0.6	1.1	433	433	0	0	0	NA	0.0	0	0
29.7	0.45	0.8	336	336	0	0	0	NA	0.0	0	0
29.8	0.3	0.5	240	240	0	0	0	NA	0.0	0	0
29.9	0.15	0.3	144	144	0	0	0	NA	0.0	0	0
30	0	0.0	48	48	0	0	0	NA	0.0	0	0
			4099523	3910019	189504	7221209	7221209			189504	

SECTION 3 REFERENCES

REFERENCE 3-1

American Institute of Steel Construction, Inc., 1980, Manual of Steel Construction, 8th Edition.

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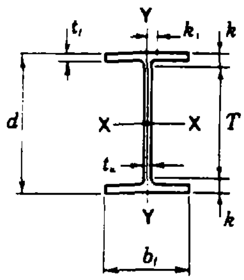

HP SHAPES
Dimensions

Designation	Area A	Depth d		Web Thickness t_w		Web $\frac{t_w}{2}$	Flange Width b_f		Flange Thickness t_f		Distance T	Distance k	Distance k_1
	In.^{2}	In.		In.		In.	In.		In.		In.	In.	In.
HP 14x117	34.4	14.21	14 1/4	0.805	13/16	7/16	14.885	14 7/8	0.805	13/16	11 1/4	1 1/2	1 1/16
x102	30.0	14.01	14	0.705	11/16	3/8	14.785	14 3/4	0.705	11/16	11 1/4	1 3/8	1
x 89	26.1	13.83	13 7/8	0.615	5/8	5/16	14.695	14 3/4	0.615	5/8	11 1/4	1 5/16	15/16
x 73	21.4	13.61	13 5/8	0.505	1/2	1/4	14.585	14 5/8	0.505	1/2	11 1/4	1 3/16	7/8
HP 13x100	29.4	13.15	13 1/8	0.765	3/4	3/8	13.205	13 1/4	0.765	3/4	10 1/4	1 7/16	1
x 87	25.5	12.95	13	0.665	11/16	3/8	13.105	13 1/8	0.665	11/16	10 1/4	1 3/8	15/16
x 73	21.6	12.75	12 3/4	0.565	9/16	5/16	13.005	13	0.565	9/16	10 1/4	1 1/4	15/16
x 60	17.5	12.54	12 1/2	0.460	7/16	1/4	12.900	12 7/8	0.460	7/16	10 1/4	1 1/8	7/8
HP 12x 84	24.6	12.28	12 1/4	0.685	11/16	3/8	12.295	12 1/4	0.685	11/16	9 1/2	1 3/8	1
x 74	21.8	12.13	12 1/8	0.605	5/8	5/16	12.215	12 1/4	0.610	5/8	9 1/2	1 5/16	15/16
x 63	18.4	11.94	12	0.515	1/2	1/4	12.125	12 1/8	0.515	1/2	9 1/2	1 1/4	7/8
x 53	15.5	11.78	11 3/4	0.435	7/16	1/4	12.045	12	0.435	7/16	9 1/2	1 1/8	7/8
HP 10x 57	16.8	9.99	10	0.565	9/16	5/16	10.225	10 1/4	0.565	9/16	7 5/8	1 3/16	13/16
x 42	12.4	9.70	9 3/4	0.415	7/16	1/4	10.075	10 1/8	0.420	7/16	7 5/8	1 1/16	3/4
HP 8x 36	10.6	8.02	8	0.445	7/16	1/4	8.155	8 1/8	0.445	7/16	6 1/8	15/16	5/8

"*Sketch*" plates (i.e., plates whose dimensions and cuts are detailed), exclusive of those with re-entrant cuts, can be supplied by most mills by shearing or gas cutting, depending on thickness.

"*Full circles*" are also available, either by shearing up to 1 in. thickness, or by gas cutting for heavier gages.

Invoicing

Standard practice is to invoice plates to the fabricator at theoretical weight at point of shipment. Permissible variations in weight are in accordance with the tables of ASTM Specification A6.

All sketch plates, including circles, are invoiced at theoretical weight and, except as noted, are subject to the same weight variations as apply to rectangular plates. Odd shapes in most instances require gas cutting, for which gas cutting extras are applicable.

All plates ordered gas cut for whatever reason, or beyond published shearing limits, take extras for gas cutting in addition to all other extras. Rolled steel bearing plates are often gas cut to prevent distortion due to shearing but would also take the regular extra for the thickness involved.

Extras for thickness, width, length, cutting, quality and quantity, etc., which are added to the base price of plates, are subject to revision, and should be obtained by inquiry to the producer. The foregoing general statements are made as a guide toward economy in design.

FLOOR PLATES

Floor plates having raised patterns are available from several mills, each offering their own style of surface projections and in a variety of widths, thicknesses, and lengths. A maximum width of 96 in. and a maximum thickness of 1 in. are available, but availability of matching widths, thicknesses, and lengths should be checked with the producer. Floor plates are generally not specified to chemical composition limits or mechanical property requirements; a commercial grade of carbon steel is furnished. However, when strength or corrosion resistance is a consideration, raised pattern floor plates are procurable in any of the regular steel specifications. As in the case of plain plates, the individual manufacturers should be consulted for precise information. The nominal or ordered thickness is that of the flat plate, exclusive of the height of raised pattern. The usual weights are as follows:

THEORETICAL WEIGHTS OF ROLLED FLOOR PLATES

Gauge No.	Theoretical Weight per Sq. Ft., Lb.	Nominal Thickness, In.	Theoretical Weight per Sq. Ft., Lb.	Nominal Thickness, In.	Theoretical Weight per Sq. Ft., Lb.
18	2.40	1/8	6.16	1/2	21.47
16	3.00	3/16	8.71	9/16	24.02
14	3.75	1/4	11.26	5/8	26.58
13	4.50	5/16	13.81	3/4	31.68
12	5.25	3/8	16.37	7/8	36.78
		7/16	18.92	1	41.89

Note: Thickness is measured near the edge of the plate, exclusive of raised pattern.

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REDUCTION FACTORS FOR TABULATED DEFLECTION CONSTANT, D_c

	Unbraced length, L_b	Compact	Non-compact
F_y = 36 ksi	$L_c \geq L_b$	1.0	
	$L_u \geq L_b > L_c$	$\frac{22}{24}$	$\frac{22}{F_b^*}$
F_y = 50 ksi	$L_c \geq L_b$	1.0	
	$L_u \geq L_b > L_c$	$\frac{30}{33}$	$\frac{30}{F_b^*}$

* The value of F_b is 24 ksi for F_y = 36 ksi, 30 ksi for F_y = 50 ksi, or computed from AISC Specification Formula (1.5-5a).

The live load deflection of floor beams supporting plastered ceilings should be limited to not more than $1/360$ of the span length. This limit is generally not reached for these span lengths until the ratio of live load to dead load is approximately 1.0.

Deflection constants are not tabulated for angles included in the tables. When required, they may be calculated from the general expression for deflection given above.

CONCENTRATED LOAD CONDITIONS

The load tables are also applicable to laterally supported simple beams with equal concentrated loads spaced as shown in the table of concentrated load equivalents, pg. 2-113. Except for spans where shear controls the design, the tables may be entered with a uniform load constant equal to the equivalent uniform load times the span. For span lengths less than L_v, beam capacity is limited by shear in the web. For this case, the sum of the concentrated loads must be equal to or less than $2V$.

Deflection constants listed in the tables must be multiplied by the appropriate deflection coefficient to determine the concentrated load deflection.

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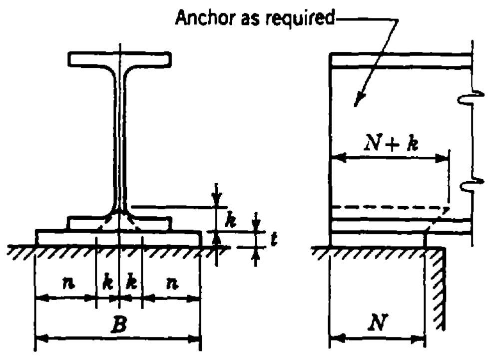
BEAMS

Design of bearing plates

When a beam is supported by a masonry wall or pilaster, it is essential that the beam reaction be distributed over an area sufficient to keep the average pressure on the masonry within allowable limits. In the absence of code provisions, an allowable F_p, depending on the type of construction, may be selected from AISC Specification Section 1.5.5.

The following method of design is recommended for most masonry.*



R = Reaction of beam, kips

A = $B \times N$ = Area of plate, square inches

F_b = Allowable bending stress in plate, ksi

F_p = Allowable bearing pressure on support, ksi

f_p = Actual bearing pressure on support, ksi

k = Distance from bottom of beam to web toe of fillet, inches (from beam dimension tables)

t = Thickness of plate, inches

The beam reaction, R, is assumed to be uniformly distributed to the plate over the area $N \times 2k$. The bearing plate is assumed to distribute this load uniformly over the masonry support.

Calculate minimum bearing length N, based on web crippling by rearranging AISC Specification Formula (1.10-9).

$$N = \frac{R}{0.75F_y \times \text{web thickness}} - k$$

Establish F_b, ksi and F_p, ksi.

Determine the required area, $A = R/F_p$, square inches.

Establish N and solve for $B = A/N$. The length of bearing, N, is usually governed by the available wall thickness or some other structural consideration. B and N should preferably be in full inches, and B rounded off so that $B \times N \geq A$.

Determine the actual bearing pressure, $f_p = R/(B \times N)$.

Determine $n = (B/2) - k$ and, using the actual f_p, solve for t in the formula:

$$t = \sqrt{\frac{3f_p n^2}{F_b}}$$

* For concrete supports where the bearing plate does not cover the full concrete area, see Example 2.

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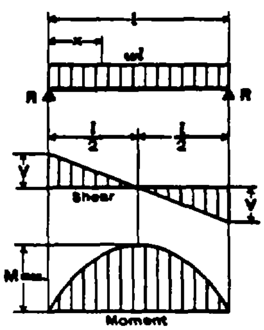
BEAM DIAGRAMS AND FORMULAS
For various static loading conditions

For meaning of symbols, see page 2-111.

1. SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load . . . . $= wl$

$R = V$ . . . . $= \frac{wl}{2}$

V_x . . . . $= w\left(\frac{l}{2} - x\right)$

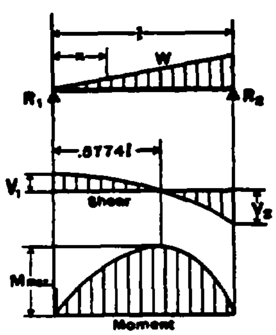
M max. (at center) . . . . . $= \frac{wl^2}{8}$

M_x . . . . $= \frac{wx}{2}(l - x)$

Δmax. (at center) . . . . . $= \frac{5wl^4}{384EI}$

Δx . . . . $= \frac{wx}{24EI}(l^3 - 2lx^2 + x^3)$

2. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load . . . . $= \frac{16W}{9\sqrt{3}} = 1.0264W$

$R_1 = V_1$ . . . . $= \frac{W}{3}$

$R_2 = V_2$ max. . . . . $= \frac{2W}{3}$

V_x . . . . $= \frac{W}{3} - \frac{Wx^2}{l^2}$

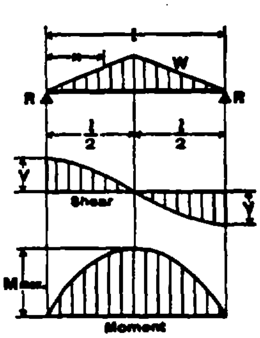
M max. $\left(\text{at } x = \frac{l}{\sqrt{3}} = .5774l\right)$ . . . $= \frac{2Wl}{9\sqrt{3}} = .1283\,Wl$

M_x . . . . $= \frac{Wx}{3l^2}(l^2 - x^2)$

Δmax. $\left(\text{at } x = l\sqrt{1 - \sqrt{\frac{8}{15}}} = .5193l\right) = .01304\,\frac{Wl^3}{EI}$

Δx . . . . $= \frac{Wx}{180EIl^2}(3x^4 - 10l^2x^2 + 7l^4)$

3. SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO CENTER



Total Equiv. Uniform Load . . . . $= \frac{4W}{3}$

$R = V$ . . . . $= \frac{W}{2}$

V_x $\left(\text{when } x < \frac{l}{2}\right)$ . . . . . $= \frac{W}{2l^2}(l^2 - 4x^2)$

M max. (at center) . . . . . $= \frac{Wl}{6}$

M_x $\left(\text{when } x < \frac{l}{2}\right)$ . . . . . $= Wx\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$

Δmax. (at center) . . . . . $= \frac{Wl^3}{60EI}$

Δx $\left(\text{when } x < \frac{l}{2}\right)$ . . . . . $= \frac{Wx}{480\,EI\,l^2}(5l^2 - 4x^2)^2$

FOR

Specification for Mild Steel Covered Arc-Welding Electrodes, AWS A5.1
Specification for Low-Alloy Steel Covered Arc-Welding Electrodes, AWS A5.5
Specification for Bare Mild Steel Electrodes and Fluxes for Submerged-Arc Welding, AWS A5.17
Specification for Mild Steel Electrodes for Gas Metal-Arc Welding, AWS A5.18
Specification for Mild Steel Electrodes for Flux-Cored Arc Welding, AWS A5.20
Specification for Bare Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding, AWS A5.23

Manufacturer's certification shall constitute sufficient evidence of conformity with the specifications.

1.4.6 Stud Shear Connectors

Steel stud shear connectors shall conform to the requirements of Articles 4.26 and 4.27, *Structural Welding Code*, AWS D1.1-77, of the American Welding Society.

Manufacturer's certification shall constitute sufficient evidence of conformity with the code.

SECTION 1.5 ALLOWABLE STRESSES*

Except as provided in Sects. 1.6, 1.7, 1.10, 1.11, 1.16.4, and in Part 2, all components of the structure shall be so proportioned that the stress, in kips per square inch, shall not exceed the following values, except as rounded off in Appendix A. See Appendix D for allowable stresses for web-tapered members.

1.5.1 Structural Steel

1.5.1.1 Tension

Except for pin-connected members, F_t shall not exceed $0.60F_y$ on the gross area nor $0.50F_u$ on the effective net area.**

For pin-connected members: $F_t = 0.45F_y$ on the net area.**

For tension on threaded parts: See Table 1.5.2.1

1.5.1.2 Shear†

1.5.1.2.1 Except as provided in Sects. 1.5.1.2.2 and 1.10.5.2, on the cross-sectional area effective in resisting shear:

$$F_v = 0.40F_y$$

The effective area in resisting shear of rolled and fabricated shapes may be taken as the overall depth times the web thickness.

* See Appendix A for tables of numerical values for various grades of steel corresponding to provisions of this Section.

** For determination of effective net area, see Sect. 1.14.

† See Commentary Sect. 1.5.1.2.

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1.5.1.4 Bending

1.5.1.4.1 Tension and compression on extreme fibers of compact hot-rolled or built-up members (except hybrid girders and members of A514 steel) symmetrical about, and loaded in, the plane of their minor axis and meeting the requirements of this section:

$$F_b = 0.66F_y$$

In order to qualify under this section, a member must meet the following requirements:

1. The flanges shall be continuously connected to the web or webs.
2. The width-thickness ratio of unstiffened projecting elements of the compression flange, as defined in Sect. 1.9.1.1, shall not exceed $65/\sqrt{F_y}$.
3. The width-thickness ratio of stiffened elements of the compression flange, as defined in Sect. 1.9.2.1, shall not exceed $190/\sqrt{F_y}$.
4. The depth-thickness ratio of the web or webs shall not exceed the value given by Formula (1.5-4a) or (1.5-4b), as applicable.

$$d/t = \frac{640}{\sqrt{F_y}}\left(1 - 3.74\frac{f_a}{F_y}\right) \qquad \text{when } f_a/F_y \le 0.16 \qquad (1.5\text{-}4a)$$

$$d/t = 257/\sqrt{F_y} \qquad \text{when } f_a/F_y > 0.16 \qquad (1.5\text{-}4b)$$

5. The laterally unsupported length of the compression flange of members other than circular or box members shall not exceed the value

$$\frac{76b_f}{\sqrt{F_y}} \quad \text{nor} \quad \frac{20{,}000}{(d/A_f)F_y}$$

6. The laterally unsupported length of the compression flange of a box-shaped member of rectangular cross section whose depth is not more than 6 times the width and whose flange thickness is not more than 2 times the web thickness shall not exceed the value

$$\left(1950 + 1200\frac{M_1}{M_2}\right)\frac{b}{F_y}$$

except that it need not be less than $1200(b/F_y)$.

7. The diameter-thickness ratio of hollow circular sections shall not exceed $3300/F_y$.

Except for hybrid girders and members of A514 steel, beams and girders (including members designed on the basis of composite action) which meet the requirements of subparagraphs 1 through 7, above, and are continuous over supports or are rigidly framed to columns by means of rivets, high-strength bolts, or welds, may be proportioned for 9/10 of the negative moments produced by gravity loading which are maximum at points of support, provided that, for such members, the maximum positive moment shall be increased by 1/10 of the average negative moments. This reduction shall not apply to moments produced by loading on cantilevers. *If the negative moment is resisted by a column rigidly framed to the beam or girder, the 1/10 reduction may be used in proportioning the column for the combined axial and bending loading, provided that the stress, f_a, due to any concurrent axial load on the member, does not exceed $0.15F_a$.*

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REFERENCE 3-2

American Concrete Institute, Building Code Requirements for Reinforced Concrete ACI 318-83, Revised 1986.

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10.14.7.1 – Specified compressive strength of concrete f'_c shall be not less than 2500 psi.

10.14.7.2 – Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

10.14.7.3 – Spiral reinforcement shall conform to Section 10.9.3.

10.14.7.4. – Longitudinal bars located within the spiral shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.14.7.5 – Longitudinal bars located within the spiral may be considered in computing A_t and I_t.

10.14.8 – Tie reinforcement around structural steel core

A composite member with laterally tied concrete around a structural steel core shall conform to the following.

10.14.8.1 – Specified compressive strength of concrete f'_c shall be not less than 2500 psi.

10.14.8.2 – Design yield strength of structural steel core shall be the specified minimum yield strength for grade of structural steel used but not to exceed 50,000 psi.

10.14.8.3 – Lateral ties shall extend completely around the structural steel core.

10.14.8.4 – Lateral ties shall have a diameter not less than 1/50 times the greatest side dimension of composite member, except that ties shall not be smaller than #3 and need not be larger than #5. Welded wire fabric of equivalent area may be used.

10.14.8.5 – Vertical spacing of lateral ties shall not exceed 16 longitudinal bar diameters, 48 tie bar diameters, or 1/2 times the least side dimension of the composite member.

10.14.8.6 – Longitudinal bars located within the ties shall be not less than 0.01 nor more than 0.08 times net area of concrete section.

10.14.8.7 – A longitudinal bar shall be located at every corner of a rectangular cross section, with other longitudinal bars spaced not farther apart than one-half the least side dimension of the composite member.

10.14.8.8 – Longitudinal bars located within the ties may be considered in computing A_t for strength but not in computing I_t for evaluation of slenderness effects.

10.15 – Bearing strength

10.15.1 – Design bearing strength on concrete shall not exceed $\phi(0.85f'_cA_1)$, except when the supporting surface is wider on all sides than the loaded area, design bearing strength on the loaded area may be multiplied by $\sqrt{A_2/A_1}$ but not more than 2.

10.15.2 – Section 10.15 does not apply to post-tensioning anchorages.

REFERENCE 3-3

Soil Conservation Service, 1986, 210-VI-TR-55, Second Edition, June.

3-72

Chapter 2: Estimating runoff

SCS Runoff Curve Number method

The SCS Runoff Curve Number (CN) method is described in detail in NEH-4 (SCS 1985). The SCS runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \quad \text{[Eq. 2-1]}$$

where

Q = runoff (in),
P = rainfall (in),
S = potential maximum retention after runoff begins (in), and
I_a = initial abstraction (in).

Initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. I_a is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, I_a was found to be approximated by the following empirical equation:

$$I_a = 0.2S. \quad \text{[Eq. 2-2]}$$

By removing I_a as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting equation 2-2 into equation 2-1 gives

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \quad \text{[Eq. 2-3]}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and S is related to CN by

$$S = \frac{1000}{CN} - 10. \quad \text{[Eq. 2-4]}$$

Figure 2-1 and table 2-1 solve equations 2-3 and 2-4 for a range of CN's and rainfall.

Factors considered in determining runoff curve numbers

The major factors that determine CN are the hydrologic soil group (HSG), cover type, treatment, hydrologic condition, and antecedent runoff condition (ARC). Another factor considered is whether impervious areas outlet directly to the drainage system (connected) or whether the flow spreads over pervious areas before entering the drainage system (unconnected). Figure 2-2 is provided to aid in selecting the appropriate figure or table for determining curve numbers.

CN's in table 2-2 (*a* to *d*) represent average antecedent runoff condition for urban, cultivated agricultural, other agricultural, and arid and semiarid rangeland uses. Table 2-2 assumes impervious areas are directly connected. The following sections explain how to determine CN's and how to modify them for urban conditions.

Hydrologic soil groups

Infiltration rates of soils vary widely and are affected by subsurface permeability as well as surface intake rates. Soils are classified into four HSG's (A, B, C, and D) according to their *minimum* infiltration rate, which is obtained for bare soil after prolonged wetting. Appendix A defines the four groups and provides a list of most of the soils in the United States and their group classification. The soils in the area of interest may be identified from a soil survey report, which can be obtained from local SCS offices or soil and water conservation district offices.

Most urban areas are only partially covered by impervious surfaces: the soil remains an important factor in runoff estimates. Urbanization has a greater effect on runoff in watersheds with soils having high infiltration rates (sands and gravels) than in watersheds predominantly of silts and clays, which generally have low infiltration rates.

Any disturbance of a soil profile can significantly change its infiltration characteristics. With urbanization, native soil profiles may be mixed or removed or fill material from other areas may be introduced. Therefore, a method based on soil

Chapter 4: Graphical Peak Discharge method

This chapter presents the Graphical Peak Discharge method for computing peak discharge from rural and urban areas. The Graphical method was developed from hydrograph analyses using TR-20, "Computer Program for Project Formulation—Hydrology" (SCS 1983). The peak discharge equation used is

$$q_p = q_u A_m Q F_p \qquad \text{[Eq. 4-1]}$$

where

q_p = peak discharge (cfs);
q_u = unit peak discharge (csm/in);
A_m = drainage area (mi^2);
Q = runoff (in); and
F_p = pond and swamp adjustment factor.

The input requirements for the Graphical method are as follows: (1) T_c (hr), (2) drainage area (mi^2), (3) appropriate rainfall distribution (I, IA, II, or III), (4) 24-hour rainfall (in), and (5) CN. If pond and swamp areas are spread throughout the watershed and are not considered in the T_c computation, an adjustment for pond and swamp areas is also needed.

Peak discharge computation

For a selected rainfall frequency, the 24-hour rainfall (P) is obtained from appendix B or more detailed local precipitation maps. CN and total runoff (Q) for the watershed are computed according to the methods outlined in chapter 2. The CN is used to determine the initial abstraction (I_a) from table 4-1. I_a/P is then computed.

If the computed I_a/P ratio is outside the range shown in exhibit 4 (4-I, 4-IA, 4-II, and 4-III) for the rainfall distribution of interest, then the limiting value should be used. If the ratio falls between the limiting values, use linear interpolation. Figure 4-1 illustrates the sensitivity of I_a/P to CN and P.

Peak discharge per square mile per inch of runoff (q_u) is obtained from exhibit 4-I, 4-IA, 4-II, or 4-III by using T_c (chapter 3), rainfall distribution type, and I_a/P ratio. The pond and swamp adjustment factor is obtained from table 4-2 (rounded to the nearest table value). Use worksheet 4 in appendix D to aid in computing the peak discharge using the Graphical method.

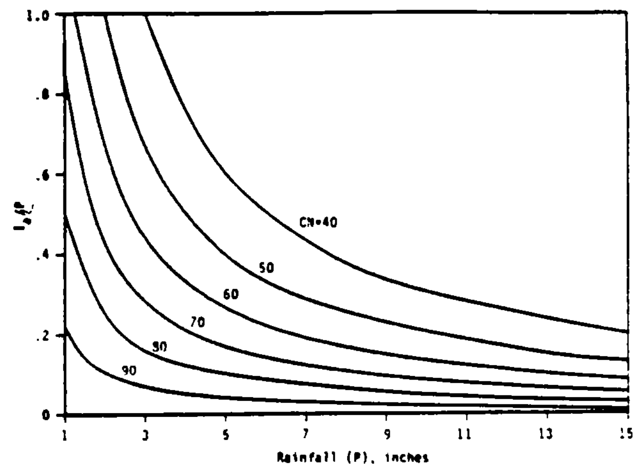

Figure 4-1.—Variation of I_a/P for P and CN.

Table 4-1.—I_a values for runoff curve numbers

Curve number	I_a (in)	Curve number	I_a (in)
40	3.000	70	0.857
41	2.878	71	0.817
42	2.762	72	0.778
43	2.651	73	0.740
44	2.545	74	0.703
45	2.444	75	0.667
46	2.348	76	0.632
47	2.255	77	0.597
48	2.167	78	0.564
49	2.082	79	0.532
50	2.000	80	0.500
51	1.922	81	0.469
52	1.846	82	0.439
53	1.774	83	0.410
54	1.704	84	0.381
55	1.636	85	0.353
56	1.571	86	0.326
57	1.509	87	0.299
58	1.448	88	0.273
59	1.390	89	0.247
60	1.333	90	0.222
61	1.279	91	0.198
62	1.226	92	0.174
63	1.175	93	0.151
64	1.125	94	0.128
65	1.077	95	0.105
66	1.030	96	0.083
67	0.985	97	0.062
68	0.941	98	0.041
69	0.899		

Exhibit 5-III, continued: Tabular hydrograph unit discharges (csm/in) for type III rainfall distribution

TRVL TIME (HR)	11.0	11.3	11.6	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	13.0	13.2	13.4	13.6	13.8	14.0	14.3	14.6	15.0	15.5	16.0	16.5	17.0	17.5	18.0	19.0	20.0	22.0	26.0
IA/P = 0.10														* * * TC =2.0 HR * * *														IA/P = 0.10				
0.0	9	11	15	19	21	23	27	31	39	48	60	75	91	129	164	187	200	191	178	147	119	92	69	55	45	37	31	26	20	16	13	3
.10	8	10	13	17	18	20	22	25	29	36	44	55	68	101	139	170	189	197	188	163	132	101	75	59	47	39	33	28	21	17	13	3
.20	7	10	13	16	18	19	21	24	28	33	40	50	62	93	129	162	185	195	190	168	137	104	77	60	48	40	33	28	21	17	13	4
.30	7	9	12	15	17	18	20	23	26	31	37	46	56	85	120	154	179	193	191	172	142	108	80	62	49	41	34	29	21	17	13	4
.40	6	8	11	14	15	16	18	19	22	25	29	34	42	64	94	129	161	183	192	183	157	120	87	66	53	43	36	30	22	17	13	5
.50	6	8	10	13	14	16	17	19	21	23	27	32	38	58	87	121	153	177	191	185	162	124	90	68	54	44	36	31	22	18	13	5
.75	5	6	8	11	12	13	14	15	16	18	20	23	26	37	55	81	113	144	169	186	179	147	106	78	61	49	40	33	24	19	14	6
1.0	4	5	7	9	10	11	12	13	14	15	17	18	20	27	38	56	82	113	143	176	185	164	121	88	67	53	43	36	26	20	14	7
1.5	2	3	4	5	6	7	7	8	9	10	10	11	12	15	18	23	31	45	65	106	148	180	166	126	92	69	55	44	31	23	15	9
2.0	1	1	2	3	4	4	5	5	6	6	7	8	8	10	12	14	18	23	31	52	87	139	176	160	122	90	68	54	36	26	16	10
2.5	0	0	1	2	2	2	2	3	3	4	4	5	5	6	7	9	11	13	16	22	36	71	132	172	161	126	94	71	45	31	18	11
3.0	0	0	0	1	1	1	1	1	2	2	2	3	3	4	5	6	7	9	10	14	19	35	78	136	168	156	123	92	54	36	20	11
IA/P = 0.30														* * * TC =2.0 HR * * *														IA/P = 0.30				
0.0	0	0	0	0	0	0	1	2	6	11	18	29	41	75	111	140	159	170	163	145	124	103	84	70	60	51	44	39	30	25	20	4
.10	0	0	0	0	0	0	0	1	2	4	9	15	24	50	84	118	145	160	167	155	134	110	89	74	63	54	46	40	31	25	20	5
.20	0	0	0	0	0	0	0	0	1	3	7	12	20	43	76	110	138	157	165	157	138	113	91	75	64	55	47	41	32	26	20	5
.30	0	0	0	0	0	0	0	0	1	3	5	10	17	38	68	101	131	152	164	159	141	116	93	77	65	56	48	41	32	26	20	6
.40	0	0	0	0	0	0	0	0	0	1	2	4	8	22	45	76	109	137	155	163	151	125	99	81	68	58	50	43	33	26	20	7
.50	0	0	0	0	0	0	0	0	0	1	1	3	7	18	39	69	101	130	151	162	153	128	101	83	69	59	51	44	34	27	20	7
.75	0	0	0	0	0	0	0	0	0	0	0	1	2	6	17	36	63	93	122	151	158	143	114	92	76	64	55	47	36	28	21	9
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	18	37	63	93	132	157	153	125	100	82	68	58	50	38	29	21	11
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	4	12	26	59	100	142	154	128	102	83	70	59	44	34	23	15
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	5	18	43	93	142	150	125	101	82	69	50	38	24	16
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	12	41	98	141	147	122	99	81	58	43	26	17
3.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	8	38	92	135	144	124	102	69	50	30	18
IA/P = 0.50														* * * TC =2.0 HR * * *														IA/P = 0.50				
0.0	0	0	0	0	0	0	0	0	1	3	6	11	16	33	54	75	91	102	114	114	103	96	87	79	72	64	57	51	41	35	29	6
.10	0	0	0	0	0	0	0	0	1	2	5	9	14	29	49	70	87	99	106	113	104	97	88	80	72	65	58	52	42	35	29	5
.20	0	0	0	0	0	0	0	0	0	1	2	4	7	18	34	54	74	90	101	112	107	99	90	82	75	67	60	53	43	36	29	8
.30	0	0	0	0	0	0	0	0	0	0	1	3	6	15	30	49	69	86	98	111	108	100	91	83	75	68	60	54	43	36	29	8
.40	0	0	0	0	0	0	0	0	0	0	0	1	2	8	19	35	55	73	89	105	110	103	94	86	78	70	62	56	45	37	30	10
.50	0	0	0	0	0	0	0	0	0	0	0	1	2	6	16	31	50	69	85	102	109	104	95	86	78	71	63	56	45	37	30	11
.75	0	0	0	0	0	0	0	0	0	0	0	0	1	4	10	21	37	55	73	93	107	107	97	89	81	73	65	58	47	38	30	13
1.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	15	29	46	72	93	107	103	94	86	78	70	62	50	40	31	16
1.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	2	7	15	34	59	89	105	101	93	85	77	69	55	44	32	21
2.0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	3	10	25	55	90	104	100	92	84	76	61	49	34	23
2.5	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	2	7	24	59	91	103	99	91	83	68	54	36	25
3.0	0	0	0	0	C	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	7	28	62	91	101	98	90	75	60	39	26
RAINFALL TYPE = III														* * * TC = 2.0 HR * * *														SHEET 10 OF 10				

3-74

REFERENCE 3-4

Chow, V.T., Open-Channel Hydraulics, McGraw-Hill, New York.

3-76

OPEN-CHANNEL HYDRAULICS

VEN TE CHOW, Ph.D.

Professor of Hydraulic Engineering
University of Illinois

A McGRAW-HILL CLASSIC TEXTBOOK REISSUE

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nappe profile becomes essentially a function of the Froude number rather than a function of the boundary geometry as described above [4, p. 533].

Many experimental formulas for the discharge over sharp-crested weir have been developed.[1] Most such formulas can be expressed in the general form.[2]

$$Q = CLH^{1.5} \tag{14-9}$$

where C is the discharge coefficient, L is the effective length of the weir crest, and H is the measured head above the crest, *excluding* the velocity head. The effective length of the weir may be computed by

$$L = L' - 0.1NH \tag{14-10}$$

where L' is the measured length of the crest and N is the number of contractions. For two end contractions, $N = 2$. For one end contraction, $N = 1$. When no contractions are present at the two ends, $N = 0$.

According to a well-known weir formula of Rehbock [10], the coefficient C in Eq. (14-9) is approximately

$$C = 3.27 + 0.40\frac{H}{h} \tag{14-11}$$

where h is the height of weir. Measurements by Rouse [4, p. 532] indicate that this equation holds up to $H/h = 5$ but can be extended to $H/h = 10$ with fair approximation. For H/h greater than about 15, the weir becomes a sill, and the discharge is controlled by a critical section immediately upstream from the sill. The critical depth of the section is approximately equal to $H + h$. By the critical depth–discharge relationship, it can be shown that the coefficient C is

$$C = 5.68\left(1 + \frac{h}{H}\right)^{1.5} \tag{14-12}$$

The transition between weir and sill (between $H/h = 10$ and 15), however, has not yet been clearly defined.

Experiments have shown that the coefficient C in Eq. (14-9) remains approximately constant for sharp-crested weirs under varying heads if the nappe is aerated.

14-2. Aeration of the Nappe. In the preceding article the overfalling nappe is considered aerated; that is, the upper and lower nappe surfaces are subject to full atmospheric pressure. Insufficient aeration below the

[1] For a general description of sharp-crested-weir experiments and formulas, see [6]. For further studies of the discharge characteristics of sharp-crested weirs, see [7] and [8].

[2] The derivation of a theoretical weir-discharge formula can be found in many hydraulics textbooks. The first mathematical analysis on discharge of weirs was performed by Boussinesq [9].

SECTION 4

4.0 EXCAVATION VOLUME

In order to design the final cap contours, it was necessary to first determine the amount of asbestos material that will be excavated. The asbestos material will be excavated from various locations and transported to the containment area. The amount of asbestos material to be excavated from each location was estimated independently using sampling information from previous investigations.

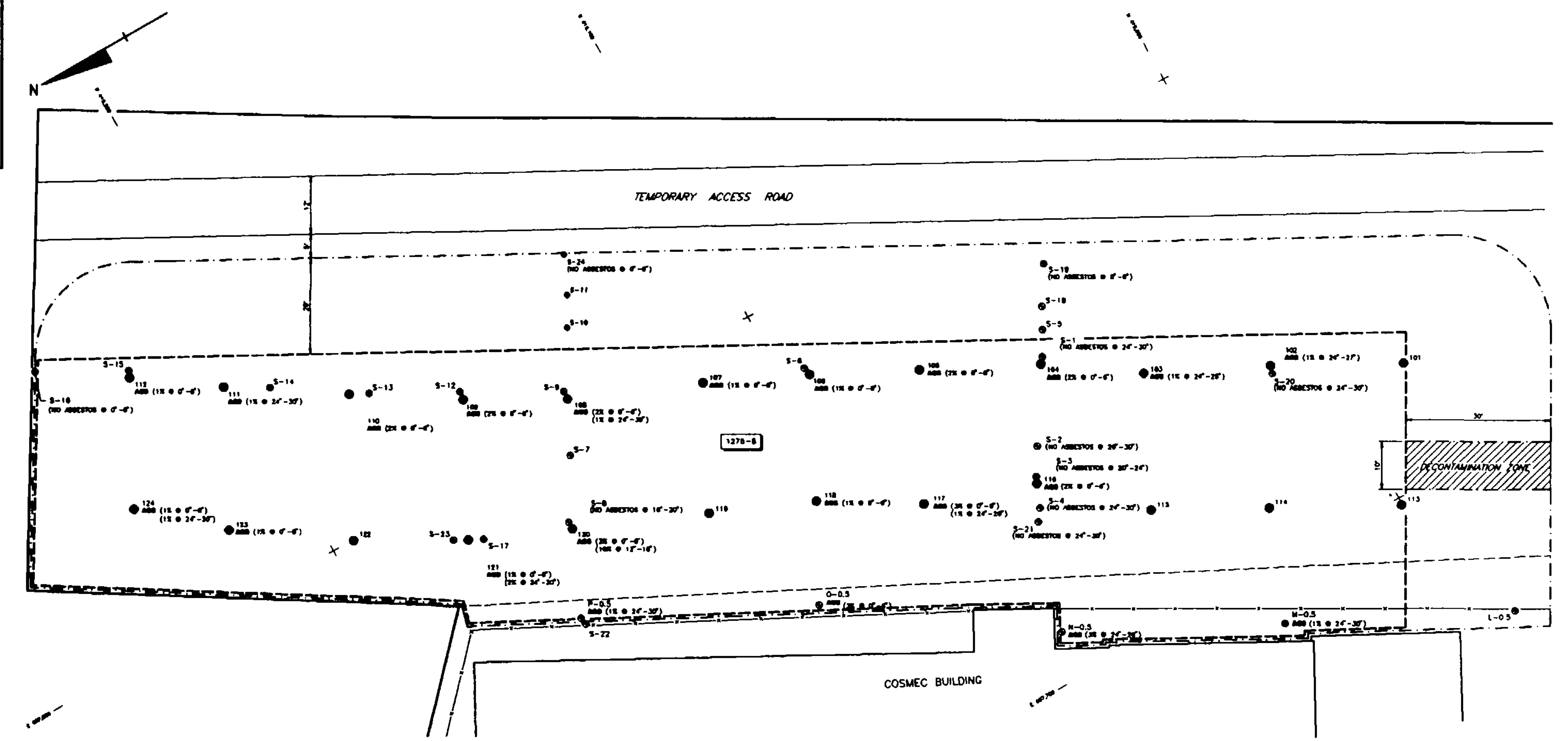
The basic approach used to estimate the excavation volume was to determine both the horizontal and vertical extent of the asbestos material located in each area as indicated in the corresponding soil samples. The in-situ excavation volume was then computed by multiplying the horizontal area by the vertical depth of asbestos material. Since the material will experience a relief in pressure after being excavated, the volume will increase. A percentage of the increase in volume will be offset once the material has been placed in the designated containment area and lightly compacted by vehicular traffic. The final excavation estimate calculated was 10,550 cubic yards.

4.1 EXCAVATION VOLUME
OF ASBESTOS CONTAINING SOILS
SOUTH STREET SITE
WALPOLE, MASSACHUSETTS

Location Designation	Canonie Drawing No.	Area (sq ft)	Depth (ft)	In-Situ Excavation Volume (cu ft)	In-Situ Excavation Volume (cu yd)
Lots 1235-3 and 1275-6	89-223-E50 Revision 1	1,520	1	1,520	56
Gleason Court (Lot 1232-2)	89-223-E51 Revision 1	870 1,240	0.5 5 (avg)	435 6,200	16 230
Vacant Lot (Lot 1232-1A)	89-223-E53 Revision 1	2,500	3	7,500	278
Orlando Property (Lots 1245-8 and 1245-9)	89-223-E54 Revision 1	24,980	3	74,940	2,776
Cosmec/RR Easement (Lot 1275-5)	89-223-E49 Revision 1	16,060	3	48,180	1,784
Jacobsen Movers	89-223-E59				
Q-4.5, R-6	Revision 1	50	1	50	2
NA-4.5, LA-6.5		50	3	150	6
South Cosmec Building	89-223-E59 Revision 1				
L-1.5		25	3	75	3
MW-13		25	4	100	4
Asbestos Pile		Cone Shape Pile			250
West Concrete Pad (H-4, HA-35)	89-223-E59 Revision 1	50	3	150	6
Area Between Vacant Building/South Street (LA-7)	89-223-E60 Revision 1	25	1	25	1
Area West of Vacant Building	89-223-E60 Revision 1	16,050	1	16,050	594
Area Near Garage (M-12)	89-223-E60 Revision 1	25	1	25	1
Southwest River Location (F-16)	89-223-E60 Revision 1	25	1	25	1
Mill Tail Race	89-223-E60 Revision 1	6,900	4	27,600	1,022
		TOTAL			7,030

Backfill volume without heavy compaction = in-situ excavation volume plus 25% bulking factor = 7,030 x 1.25 = 8,788 cubic yards. Add 20 percent for excess excavation material = 8,788 x 1.20 = 10,595 cubic yards, say 10,550 cubic yards.

DRAWING NUMBER 89-223-E49



LEGEND:

- FENCE
- APPROXIMATE ON-FACILITY BOUNDARY
- APPROXIMATE LIMITS OF EXCAVATION
- ANTICIPATED EXCLUSION ZONE/ TEMPORARY FENCE
- SOIL BORING (SEE NOTE 1)
- SUPPLEMENTAL SOIL BORING (SEE NOTE 2)
- S-1 PRE-CONSTRUCTION SAMPLING SOIL BORING (SEE NOTE 4)
- ACS ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION (SEE NOTE 3)
- 1275-5 LOT NUMBER

NOTES:

1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION (JANUARY 17, 1989 TO APRIL 7, 1989). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS
2. SOIL BORINGS WERE TAKEN AS PART OF A SUPPLEMENTARY INVESTIGATION (JUNE 7, 1990 TO JUNE 13, 1990). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
3. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION.
4. SOIL BORINGS WITH A 'S' DESIGNATION WERE TAKEN BY CANONIE ON LOT 1275-5 APRIL 10, 1991.

SCALE 10 0 10 20 FEET

4-3

COSMEC BUILDING/RAILROAD EASEMENT EXCAVATION PLAN
SOUTH STREET SITE, WALPOLE, MA

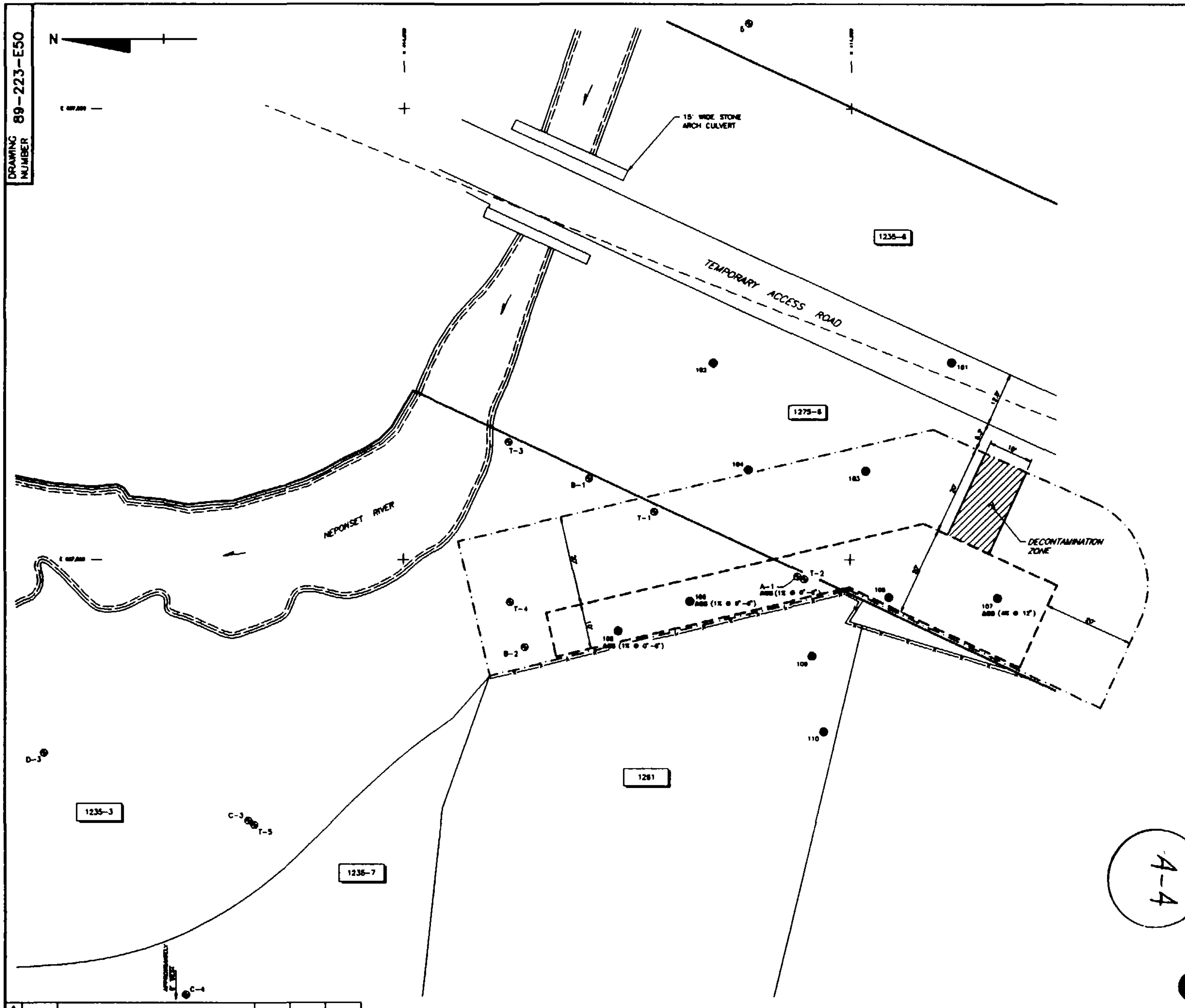
PREPARED FOR

SOUTH STREET SITE PRPs

Canonie Environmental

No.	DATE	ISSUE / REVISION	DWN. BY	CH'D BY	APP'D BY

DATE:	10-25-91
SCALE:	AS SHOWN
DRAWING NUMBER	89-223-E49



DRAWING NUMBER 89-223-E50

LEGEND:

- FENCE
- APPROXIMATE ON-FACILITY BOUNDARY
- APPROXIMATE LIMIT OF EXCAVATION
- ANTICIPATED EXCLUSION ZONE/ TEMPORARY FENCE
- 1235-3 LOT NUMBER
- DIRECTION OF FLOW
- SOIL BORING (SEE NOTE 1)
- SUPPLEMENTAL SOIL BORING (SEE NOTE 2)
- T-1 PRE-CONSTRUCTION SAMPLING SOIL BORING (SEE NOTE 4)
- ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION (SEE NOTE 3)

NOTES:

1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION (JANUARY 17, 1989 TO APRIL 7, 1989). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
2. SOIL BORINGS WERE TAKEN AS PART OF A SUPPLEMENTARY INVESTIGATION (JUNE 7, 1990 TO JUNE 13, 1990). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS
3. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION
4. SOIL BORINGS WITH A 'T' DESIGNATION WERE TAKEN BY CANONIE ON LOT 1235-3 APRIL 11, 1991.

SCALE
10 0 10 20 FEET

4-4

LOTS 1235-3 AND 1275-6
EXCAVATION PLAN
SOUTH STREET SITE, WALPOLE, MA
PREPARED FOR

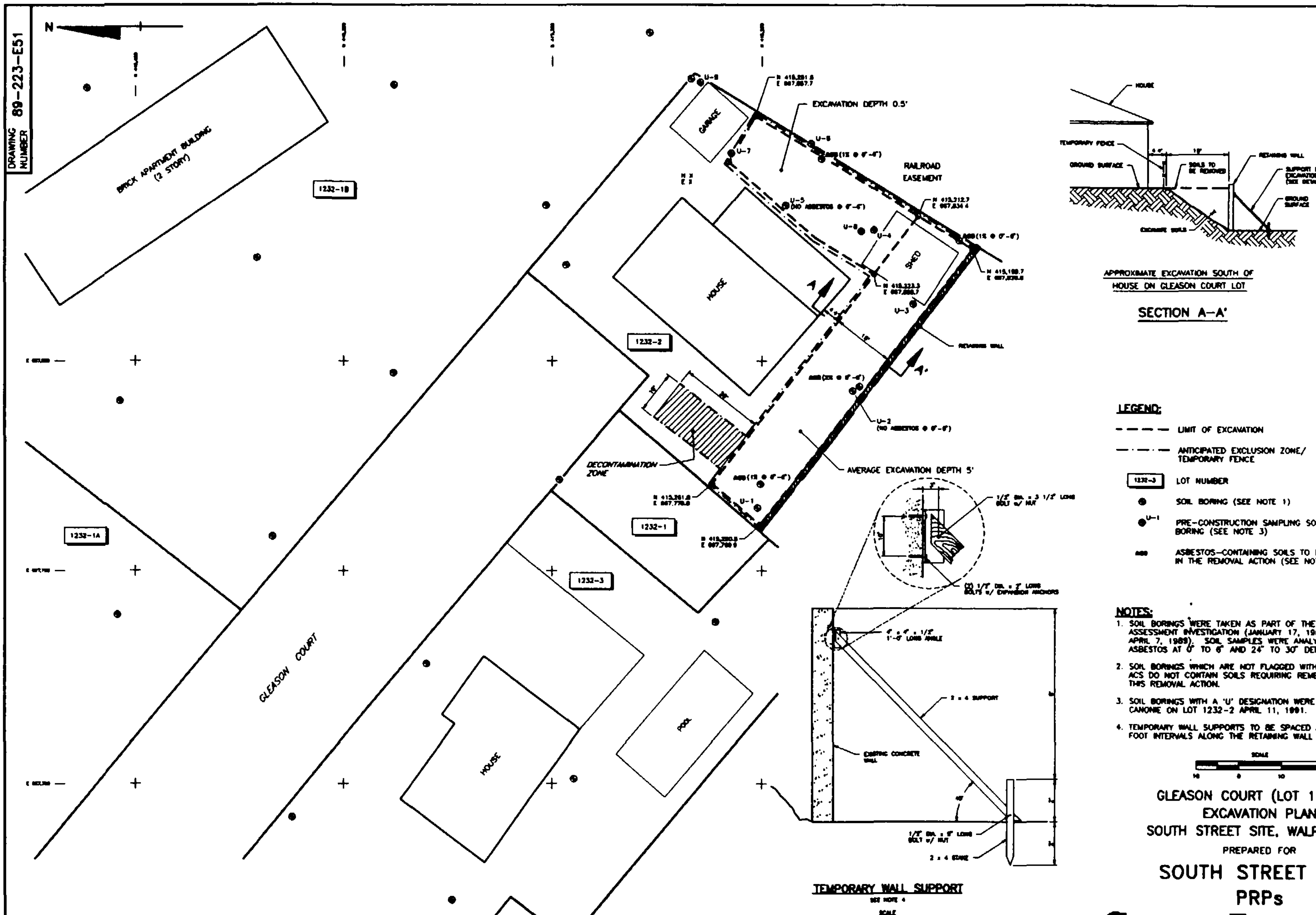
SOUTH STREET SITE PRPs

CanonieEnvironmental

No.	DATE	ISSUE / REVISION	DRN BY	CK'D BY	APP'D BY

DATE: 10-26-91		DRAWING NUMBER 89-223-E50
SCALE: AS SHOWN		

DRAWING NUMBER 89-223-E51



LEGEND:

- – – – – LIMIT OF EXCAVATION
- — · — · ANTICIPATED EXCLUSION ZONE/ TEMPORARY FENCE
- 1232-3 LOT NUMBER
- ● SOIL BORING (SEE NOTE 1)
- ● U-1 PRE-CONSTRUCTION SAMPLING SOIL BORING (SEE NOTE 3)
- ACS ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION (SEE NOTE 2 AND 3)

4-5

NOTES:

1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION (JANUARY 17, 1989 TO APRIL 7, 1989). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
2. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION.
3. SOIL BORINGS WITH A 'U' DESIGNATION WERE TAKEN BY CANONIE ON LOT 1232-2 APRIL 11, 1991.
4. TEMPORARY WALL SUPPORTS TO BE SPACED AT TEN FOOT INTERVALS ALONG THE RETAINING WALL

SCALE
10 0 10 30 FEET

GLEASON COURT (LOT 1232-2)
EXCAVATION PLAN
SOUTH STREET SITE, WALPOLE, MA

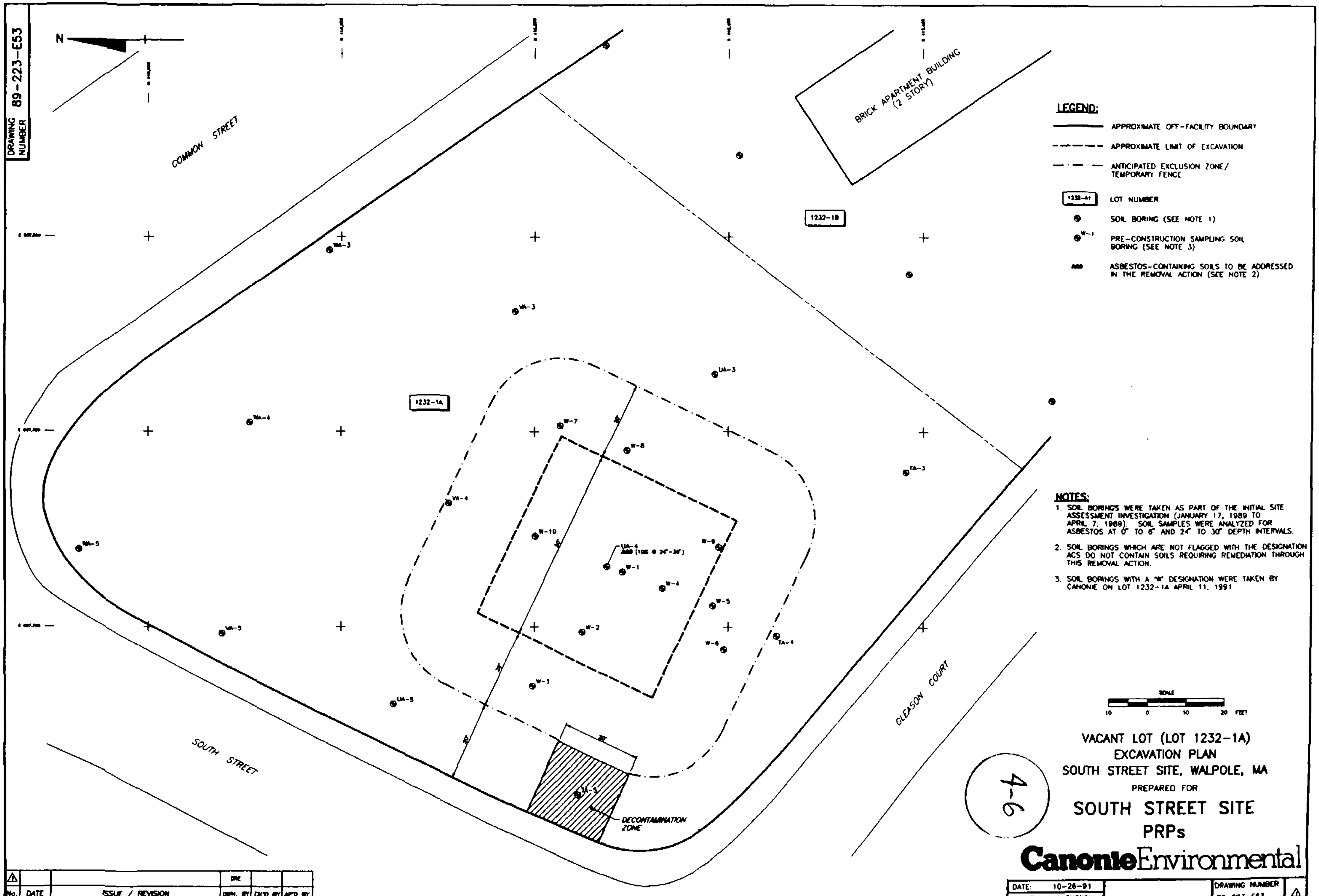
PREPARED FOR

SOUTH STREET SITE
PRPs

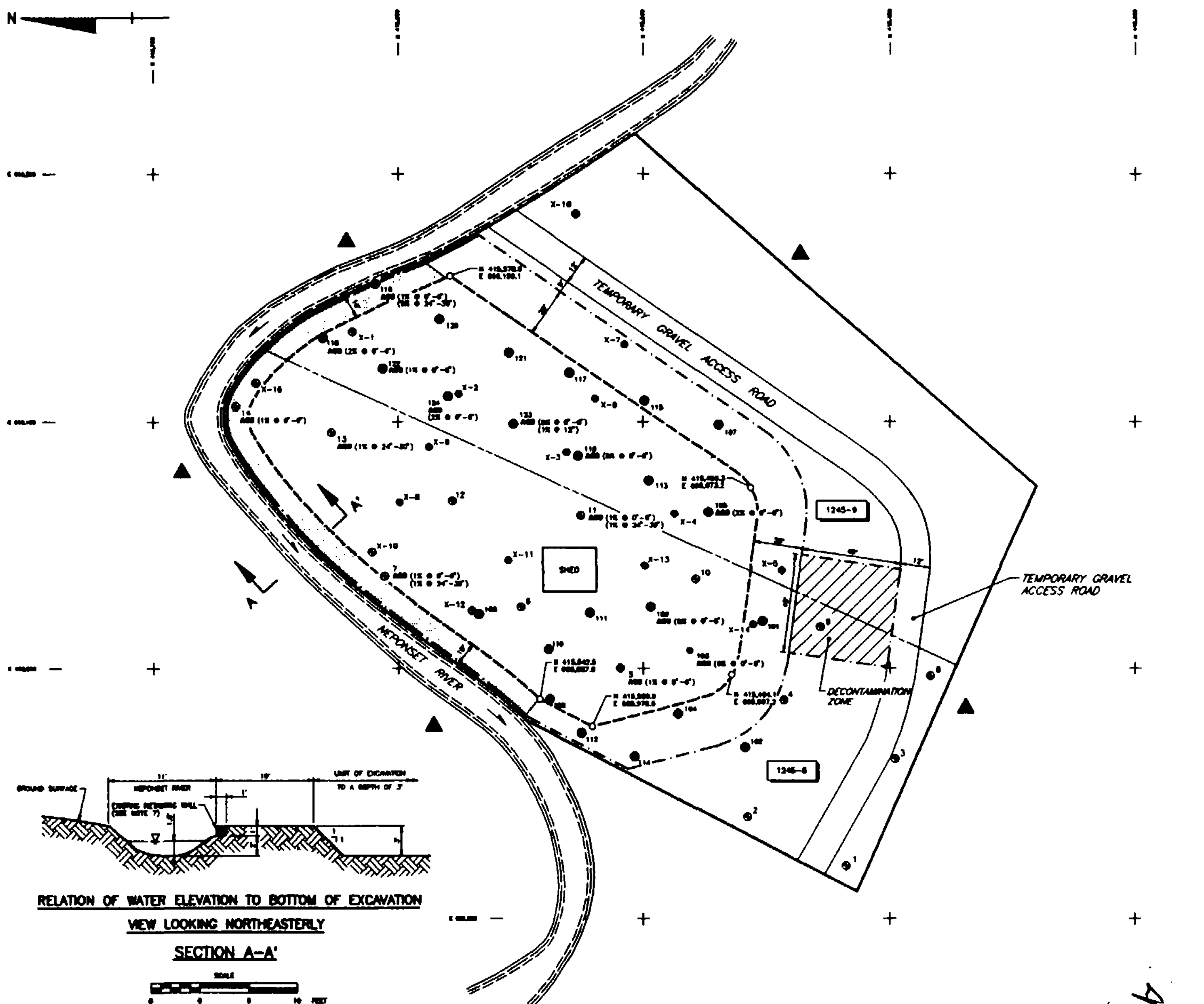
Canonie Environmental

No.	DATE	ISSUE / REVISION	DWN. BY	CH'D BY	APP'D BY
2		[illegible]	[illegible]		
1	11-27-91	[illegible]	[illegible]	[illegible]	

DATE: 10-26-91	SHEET 18	DRAWING NUMBER 89-223-E51	2
SCALE: AS SHOWN			


DRAWING NUMBER 89-223-E53
N
COMMON STREET
BRICK APARTMENT BUILDING (2 STORY)
1232-1B
1232-1A
SOUTH STREET
GLEASON COURT
DECONTAMINATION ZONE
W-1
W-2
W-3
W-4
W-5
W-6
W-7
W-8
W-9
W-10
UA-3
UA-4
UA-5
VA-3
VA-4
VA-5
TA-3
TA-4
TA-5
RA-3
RA-4
RA-5
LEGEND:
APPROXIMATE OFF-FACILITY BOUNDARY
APPROXIMATE LIMIT OF EXCAVATION
ANTICIPATED EXCLUSION ZONE/ TEMPORARY FENCE
LOT NUMBER
SOIL BORING (SEE NOTE 1)
PRE-CONSTRUCTION SAMPLING SOIL BORING (SEE NOTE 3)
ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION (SEE NOTE 2)
NOTES:
1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION (JANUARY 17, 1989 TO APRIL 7, 1989). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
2. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION.
3. SOIL BORINGS WITH A "W" DESIGNATION WERE TAKEN BY CANONIE ON LOT 1232-1A APRIL 11, 1991
SCALE
10 0 10 20 FEET
VACANT LOT (LOT 1232-1A)
EXCAVATION PLAN
SOUTH STREET SITE, WALPOLE, MA
PREPARED FOR
SOUTH STREET SITE PRPs
4-6
Canonie Environmental
DATE: 10-28-91
SCALE: AS SHOWN
DRAWING NUMBER 89-223-E53
No. DATE ISSUE / REVISION DWN. BY CK'D BY APP'D BY

DRAWING NUMBER 89-223-E54



RELATION OF WATER ELEVATION TO BOTTOM OF EXCAVATION

VIEW LOOKING NORTHEASTERLY

SECTION A-A'

LEGEND:

- OFF-FACILITY BOUNDARY
- LIMIT OF EXCAVATION TO A DEPTH OF 3'
- ANTICIPATED EXCLUSION ZONE/ TEMPORARY FENCE
- PROPERTY LINE
- LOT NUMBER
- SOIL BORING (SEE NOTE 1)
- SUPPLEMENTAL SOIL BORING (SEE NOTE 2)
- PRE-CONSTRUCTION SAMPLING SOIL BORING (SEE NOTE 4)
- ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION (SEE NOTE 3)
- DIRECTION OF FLOW
- PROPOSED AIR MONITORING STATION
- ADDITIONAL EXCAVATION AREA (SEE NOTE 6 AND 7)

NOTES:

1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION (JANUARY 17, 1989 TO APRIL 7, 1989). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
2. SOIL BORINGS WERE TAKEN AS PART OF A SUPPLEMENTARY INVESTIGATION (JUNE 7, 1990 TO JUNE 13, 1990). SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
3. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION.
4. SOIL BORINGS WITH A "X" DESIGNATION WERE TAKEN BY CANONIE ON LOT 1245-8 AND LOT 1245-9 APRIL 11, 1991.
5. EXCLUSION ZONE WILL BE EXPANDED AS NECESSARY TO MAINTAIN A MINIMUM OF 20' FROM EXCAVATION AREAS.
6. THE AREA ADJACENT TO THE NEPONSET RIVER SHALL BE EXCAVATED IN 20' LONG x 10' WIDE SECTIONS. THE EXCAVATED SOIL SHALL BE REPLACED WITH SPECIAL BORROW TYPE M1.02.0a BEFORE PROCEEDING TO THE NEXT SECTION.
7. THE EXISTING RETAINING WALL SHALL BE REBUILT UTILIZING THE ORIGINAL STONE MATERIAL.

SCALE

ORLANDO PROPERTY
(LOTS 1245-8 AND 1245-9)
EXCAVATION PLAN AND PROPOSED
AIR MONITORING STATION LOCATIONS
SOUTH STREET SITE, WALPOLE, MA

PREPARED FOR

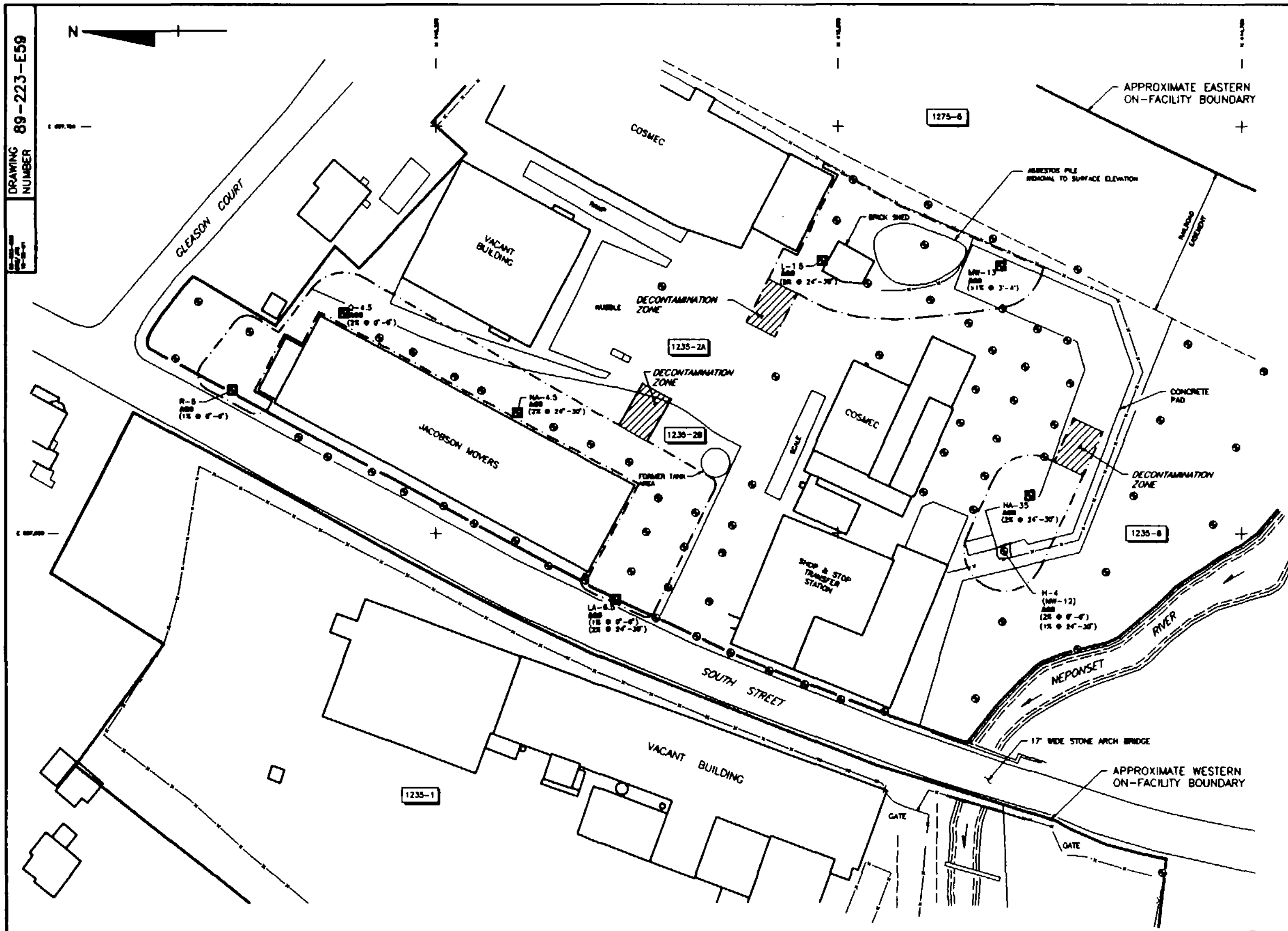
SOUTH STREET SITE
PRPs

Canonie Environmental

4-7

No.	DATE	ISSUE / REVISION	DWN BY	CK'D BY	APP'D BY
		ISSUED FOR REVIEW	EMC		
	11-27-91	ISSUED FOR CLIENT REVIEW	EMC		

DATE: 10-26-91	SHEET 20	DRAWING NUMBER 89-223-E54
SCALE AS SHOWN		



LEGEND:

- FENCE
- DIRECTION OF FLOW
- APPROXIMATE ON-FACILITY BOUNDARY
- LOT NUMBER
- SOIL BORING (SEE NOTE 1)
- EXCAVATION LOCATION, 5' x 5' x (ACS DEPTH)
- ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION
- ANTICIPATED EXCLUSION ZONE/TEMPORARY FENCE

NOTES:

1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION (JANUARY 17, 1989 TO APRIL 7, 1989)
2. SOIL BORINGS WERE TAKEN AS PART OF A SUPPLEMENTARY INVESTIGATION. (JUNE 7, 1990 TO JUNE 13, 1990)
3. SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS
4. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION

REFERENCES:

- DAMES & MOORE, PLATE No. 4 (SHEET 1 of 3), 8-14-88.
- "WALPOLE MAPPING PROJECT", JAMES W. SEWALL COMPANY, JULY 1988.

EASTERN ON-FACILITY AREA
EXCAVATION PLAN
SOUTH STREET SITE, WALPOLE, MA

PREPARED FOR

SOUTH STREET SITE
PRPs

Canonie Environmental

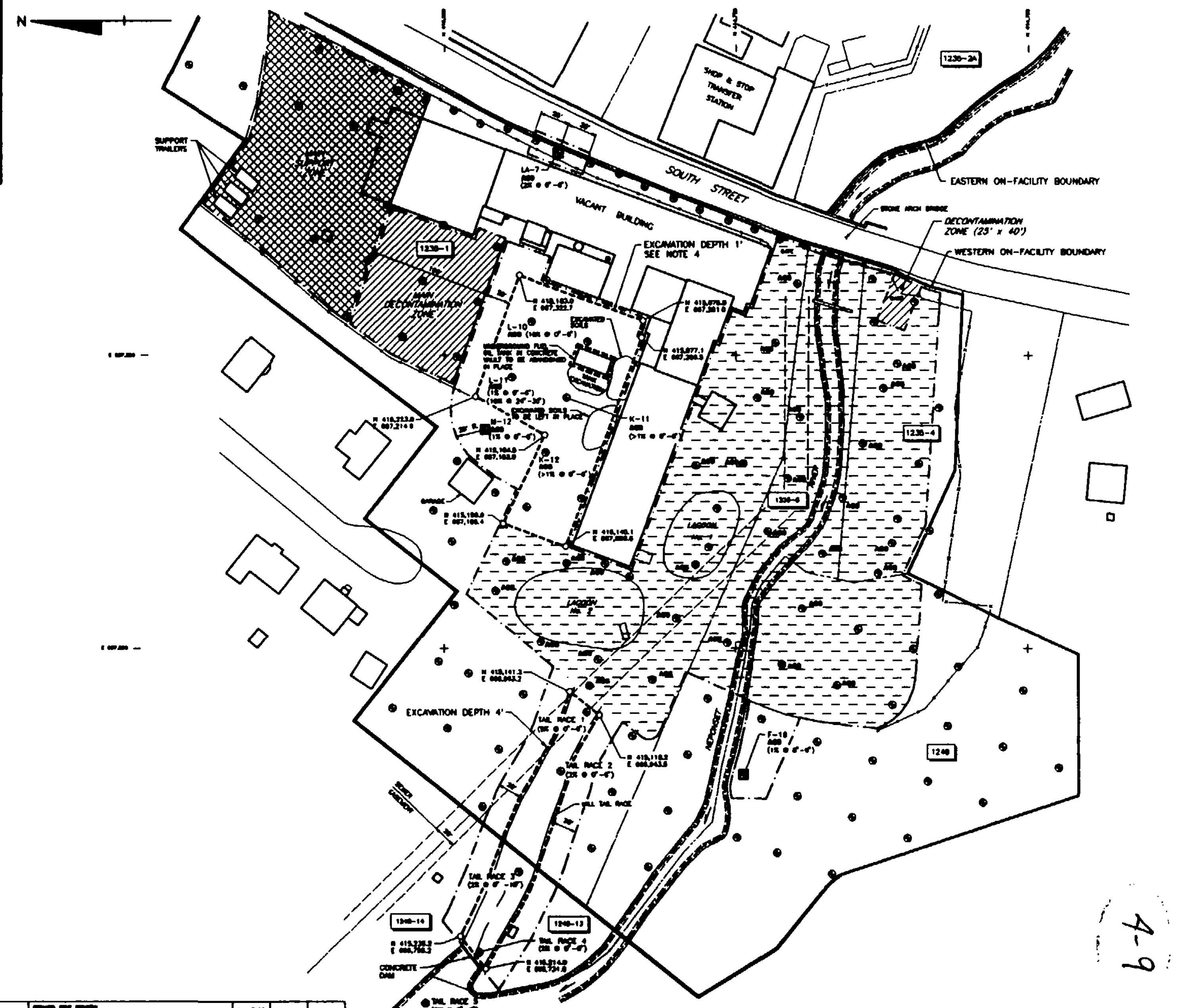
4-8

No.	DATE	ISSUE / REVISION	DWN BY	CK'D BY	APP'D BY

DATE:	10-28-91	DRAWING NUMBER	89-223-E59
SCALE:	AS SHOWN		

DRAWING NUMBER 89-223-E59

DRAWING NUMBER 89-223-E60



LEGEND:

- FENCE
- DIRECTION OF FLOW
- DITCH OR CREEK
- PROPERTY LINE
- ON-FACILITY BOUNDARY
- LOT NUMBER
- SOIL BORING (SEE NOTE 1)
- ASBESTOS-CONTAINING SOILS TO BE ADDRESSED IN THE REMOVAL ACTION (SEE NOTE 3)
- EXCAVATION LOCATION, 5' x 5' X (ACS DEPTH) (SEE TABLE 1)
- EXCAVATION BOUNDARY
- ANTICIPATED EXCLUSION ZONE/TEMPORARY FENCE
- AREA FOR CONSOLIDATION/CONTAINMENT OF EXCAVATED SOILS
- DECONTAMINATION ZONE
- SUPPORT ZONE

TABLE 1

SOIL BORING NO.	EXCAVATION DEPTH (FEET)	COORDINATES	
		NORTH	EAST
LA-7	1	415,151.2	667,430.6
M-12	1	415,215.0	667,166.7
F-10	1	414,863.5	666,892.9

NOTES:

1. SOIL BORINGS WERE TAKEN AS PART OF THE INITIAL SITE ASSESSMENT INVESTIGATION. (JANUARY 17, 1989 TO APRIL 7, 1989)
2. SOIL SAMPLES WERE ANALYZED FOR ASBESTOS AT 0" TO 6" AND 24" TO 30" DEPTH INTERVALS.
3. SOIL BORINGS WHICH ARE NOT FLAGGED WITH THE DESIGNATION ACS DO NOT CONTAIN SOILS REQUIRING REMEDIATION THROUGH THIS REMOVAL ACTION.
4. AREA EXCAVATED TO A DEPTH OF 1' SHALL BE REGRADED AND PAVED.

SCALE 40 0 40 80 FEET

REFERENCES:

- DAMES & MOORE, PLATE No. 4 (SHEET 1 of 2), 6-14-89
- "WALPOLE MAPPING PROJECT," JAMES W. SEWALL COMPANY, JULY 1988.

No.	DATE	ISSUE / REVISION	DWN. BY	CH'D BY	AP'D BY
3		ISSUED FOR REVIEW	[illegible]		
2	11-27-91	ISSUED FOR CLIENT REVIEW	[illegible]	[illegible]	
1	11-10-91	ISSUED TO NORMANDEAU ASSOCIATES	[illegible]	[illegible]	

WESTERN ON-FACILITY AREA
EXCAVATION PLAN
SOUTH STREET SITE, WALPOLE, MA

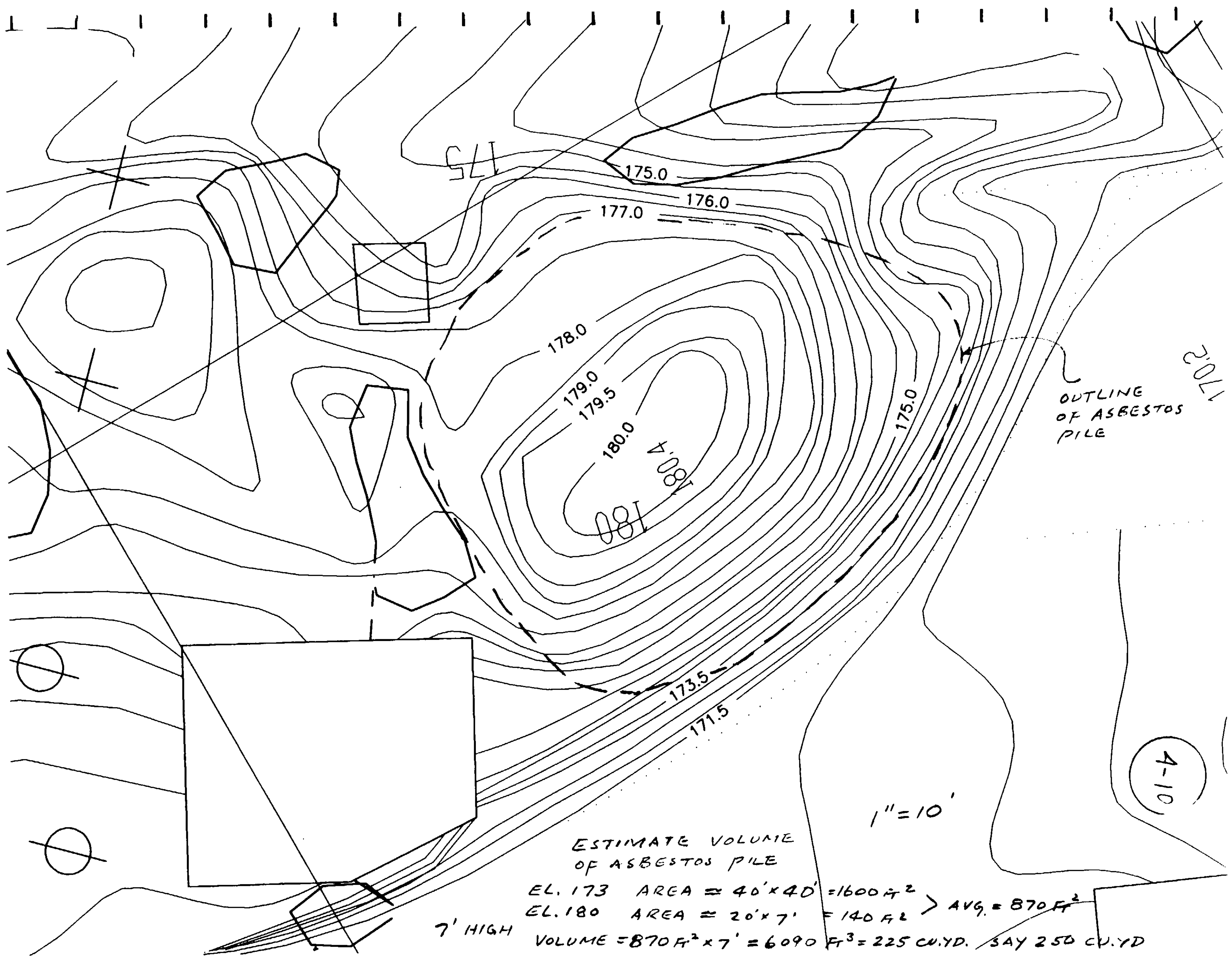
PREPARED FOR

SOUTH STREET SITE PRPs

Canonie Environmental

DATE: 10-28-91	SHEET 22	DRAWING NUMBER 89-223-E60
SCALE: AS SHOWN		

A-9


175.0
176.0
177.0
178.0
179.0
179.5
180.0
175.0
173.5
171.5
OUTLINE OF ASBESTOS PILE
4-10
1" = 10'
ESTIMATE VOLUME OF ASBESTOS PILE
EL. 173 AREA = 40' x 40' = 1600 ft²
EL. 180 AREA = 20' x 7' = 140 ft² > AVG. = 870 ft²
7' HIGH
VOLUME = 870 ft² x 7' = 6090 ft³ = 225 CU.YD. SAY 250 CU.YD

SECTION 5

5.0 SOIL CAP DESIGN

A final cap will be placed over the asbestos material after it has been placed in the containment area. The cap will be composed of two layers. The lower layer will be clean sand and will be covered by an upper layer of topsoil to promote vegetation growth. The cap will be contoured to promote drainage into the Neponset River at the end of the pipe arch culvert.

The purpose of the soil cap is to prevent any exposure to the asbestos material. It is not imperative that the cap be impermeable since asbestos is inert and will not react with the ground water. Therefore, the primary design criterion for the containment cap was the thickness. The cap thickness was designed to insure that the asbestos fibers can not be transported to the surface due to cyclic freezing and thawing. The cap thickness will provide sufficient overburden to ensure that the asbestos material is located below the frost line.

The results of the attached computer output for total frost penetration indicate a maximum of 26 inches frost penetration depth. To achieve the necessary depth of cover, the final soil cover cap design will consist of 24 inches of sand and 6 inches of topsoil.

The soil cap contours will provide a total net containment volume of 10,685 cubic yards. This volume was achieved by limiting the height of material above the crown of the pipe arch culvert to six feet. If the amount of asbestos material is greater than the 10,550 cubic yards estimated in Section 4.0, the height of material above the crown can be increased to 11 feet (see Seciton 2.2).

5.1 Cover Design

SUMMARY OF METHOD USED TO DETERMINE SOIL COVER DEPTH

THE DEPTH OF THE SOIL COVER IS DETERMINED BASED ON THE TOTAL FROST PENETRATION DEPTH. THE RATIONALE FOR DETERMINING SOIL DEPTH COVER FOR WASTE ASBESTOS WAS DEVELOPED BY US ARMY CORPS OF ENGINEERS. THE DETAILS OF THE METHODOLOGY IS PRESENTED IN THE REFERENCE 5-1 AND COMPARED WITH INFORMATION GIVEN IN REF 5-2

THE DEPTH OF FROST PENETRATION BELOW THE SURFACE IS RELATED TO THE METEOROLOGICAL VARIABLES THAT AFFECT HEAT LOSS AND THE PHYSICAL PROPERTIES OF THE SOIL. THE MAJOR VARIABLES ARE (1) THE SEVERITY OF THE WINTER AIR TEMPERATURES, and (2) THE AMOUNT OF SNOW COVER. THE SEVERITY OF THE AIR TEMPERATURE IS EXPRESSED IN TERMS OF AIR FREEZING INDEX FOR EACH WINTER THE EFFICIENCY OF THERMAL TRANSFER BETWEEN THE AIR AND GROUND OR SNOW IS EXPRESSED IN TERMS OF AN n- FACTOR WHERE n IS A FRACTION BETWEEN 0 AND 1. IN COMBINATION, THE TWO PARAMETERS RESULT IN A SURFACE FREEZING INDEX, WHICH IS A MEASURE OF THE AMOUNT OF HEAT LEAVING THE GROUND OVER AN ENTIRE WINTER. THE MANNER IN WHICH THESE VALUES ARE USED TO ESTIMATE THE DEPTH OF SOIL COVER FOR SAFE BURIAL OF WASTE ASBESTOS IS PRESENTED IN REFERENCE 5-1.

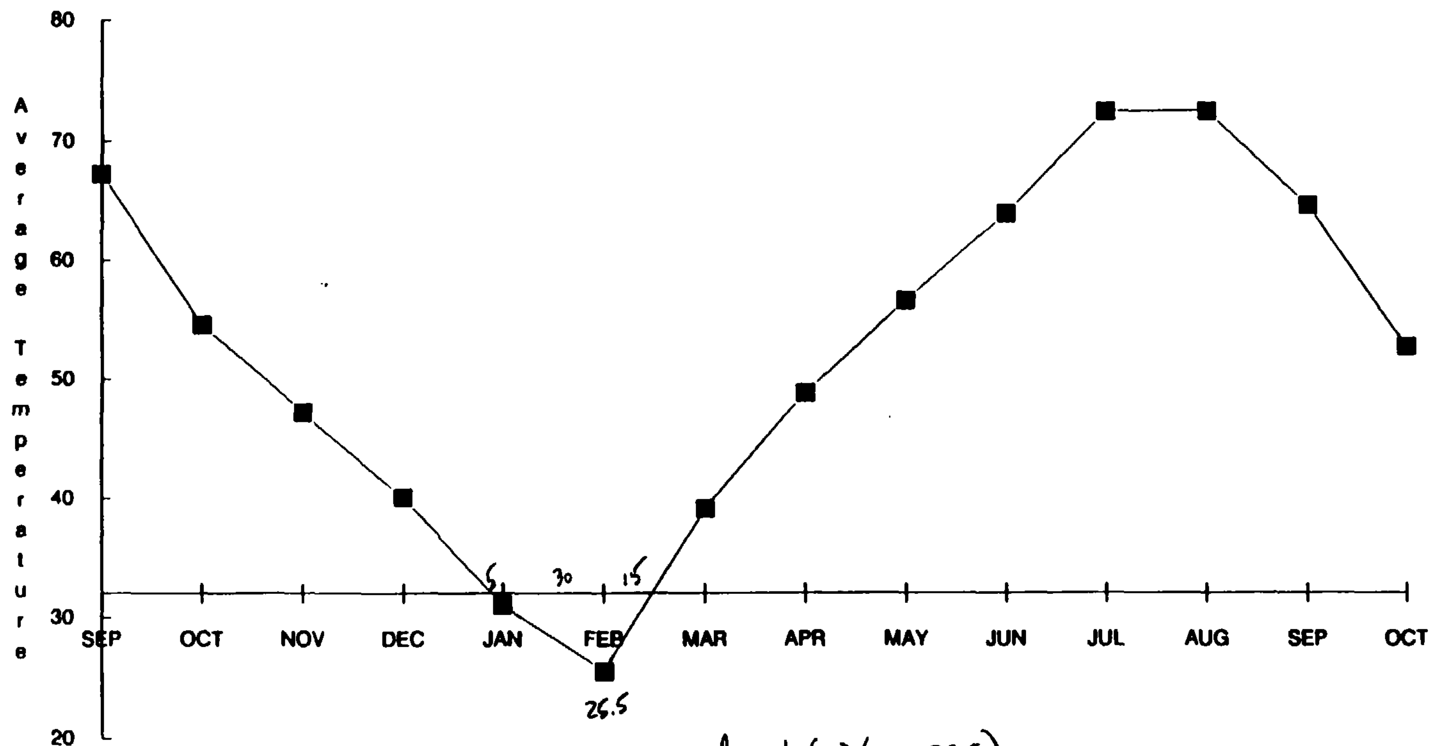
THE COMPUTER OUTPUT FOR TOTAL FROST PENETRATION DETERMINED USING THE MODIFIED BERGGREN SOLUTION IS PRESENTED AS AN ATTACHMENT.

5-3

YEAR	AIR FREEZING INDEX (DEGREE-DAYS)	RANK	PROBABILITY OF EXCEEDANCE (PERCENT)
1980-1981	550	1	1.67
1970-1971	339	2	5.00
1976-1977	335	3	8.33
1958-1959	331	4	11.67
1967-1968	316	5	15.00
1964-1965	288	6	18.33
1981-1982	273	7	21.67
1969-1970	266	8	25.00
1962-1963	255	9	28.33
1960-1961	254	10	31.67
1961-1962	219	11	35.00
1977-1978	208	12	38.33
1978-1979	196	13	41.67
1963-1964	174	14	45.00
1984-1985	171	15	48.33
1957-1958	163	16	51.67
1968-1969	157	17	55.00
1966-1967	149	18	58.33
1979-1980	143	19	61.67
1983-1984	120	20	65.00
1975-1976	104	21	68.33
1985-1986	93	22	71.67
1965-1966	72	23	75.00
1973-1974	58	24	78.33
1971-1972	47	25	81.67
1972-1973	42	26	85.00
1982-1983	7	27	88.33
1959-1960	6	28	91.67
1974-1975	0	29	95.00

163 Degree Days

1957-1958



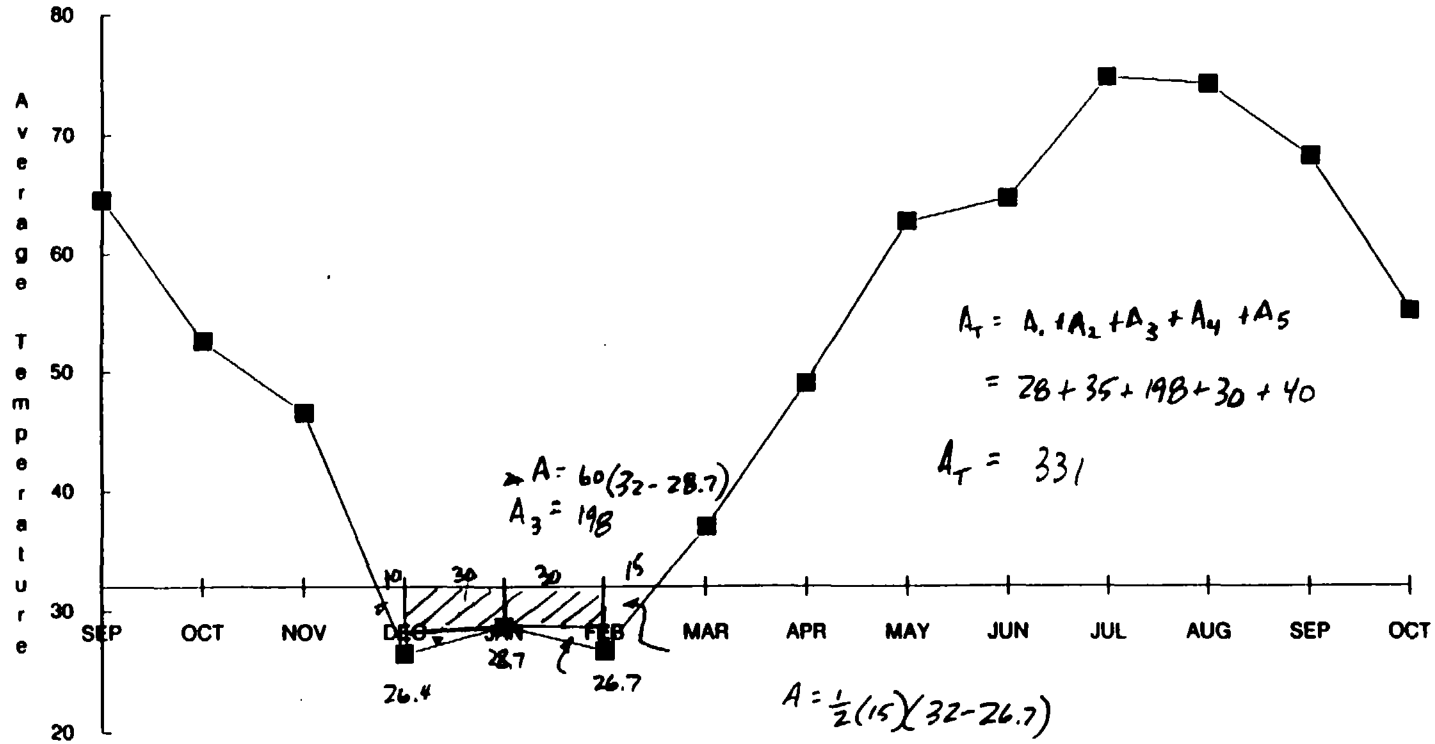
$A = \frac{1}{2}(50)(32 - 26.5)$

$A = 163$

5-4

331 Degree Days

1958-1959



$A = \frac{1}{2}(15)(32-26.7)$

$A_5 = 40$

$A = \frac{1}{2}(10)(32-26.4)$

$A_1 = 28$

$A = \frac{1}{2}(30)(28.7-26.4)$

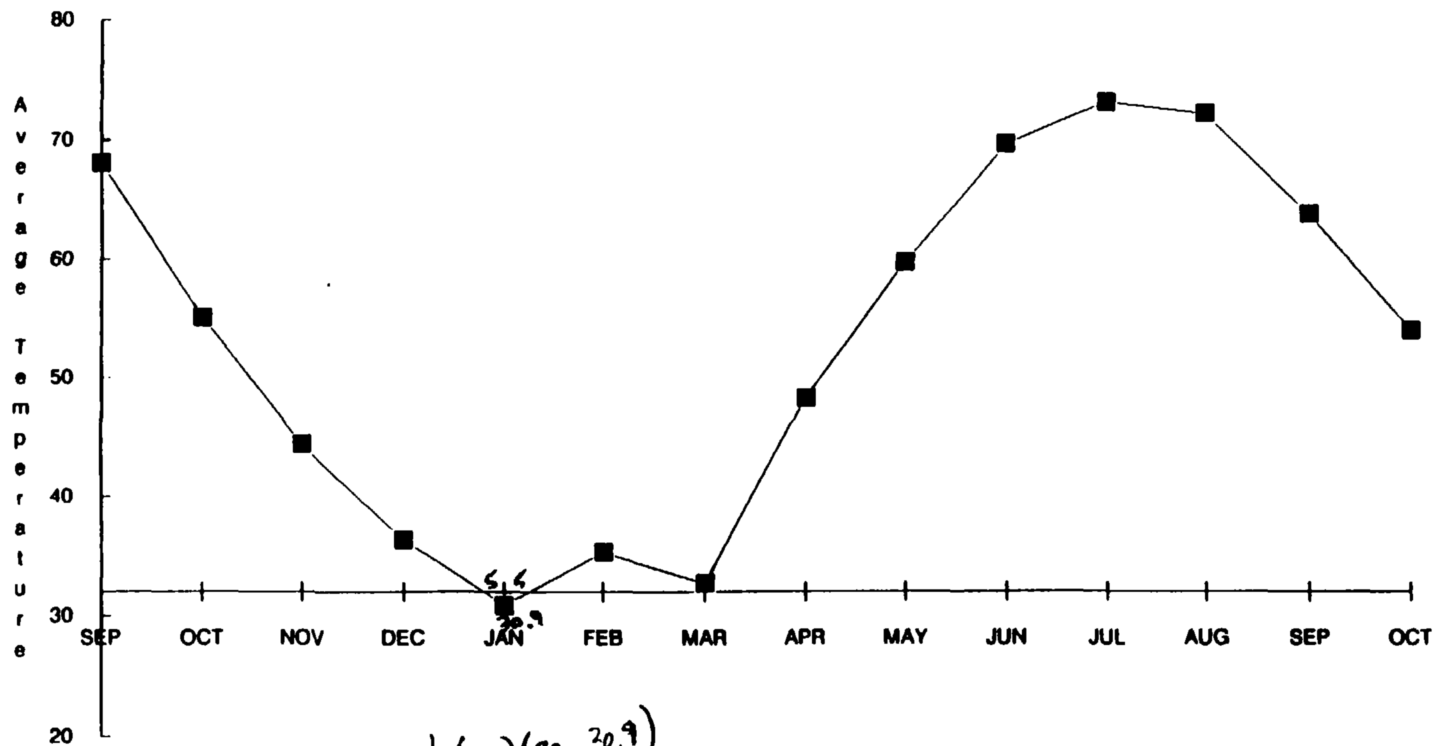
$A_2 = 35$

Page $A_4 = 30$

5-5

6 Degree Days

1959-1960



$A = \frac{1}{2}(10)(32 - 30.9)$

$A = 6$

5-6

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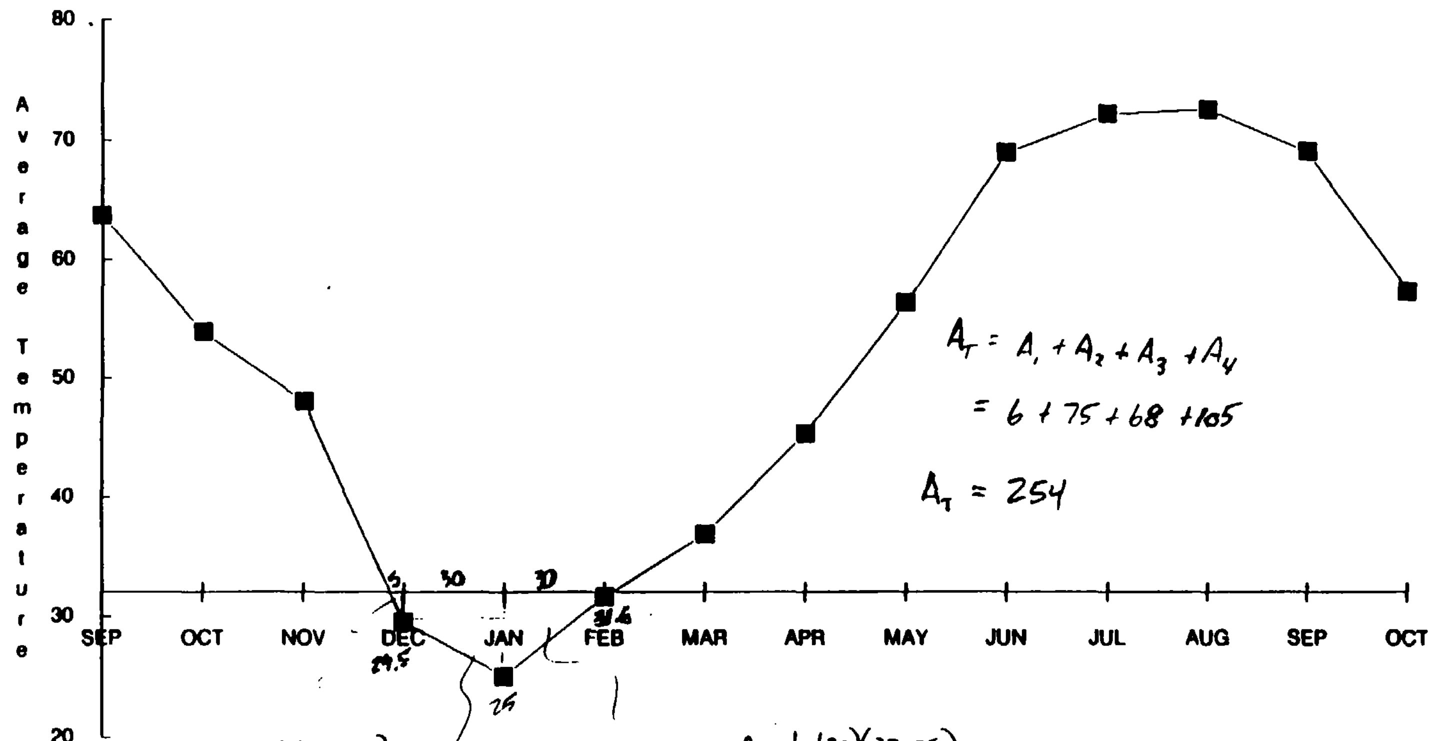
254 Degree Days

1960-1961



$A_T = A_1 + A_2 + A_3 + A_4$

$= 6 + 75 + 68 + 105$

$A_T = 254$

$A_1 = \frac{1}{2}(5)(32-29.5)$

$A_1 = 6$

$A = \frac{1}{2}(30)(29.5-25)$

$A_3 = 68$

$A = \frac{1}{2}(30)(32-25)$

$A_4 = 105$

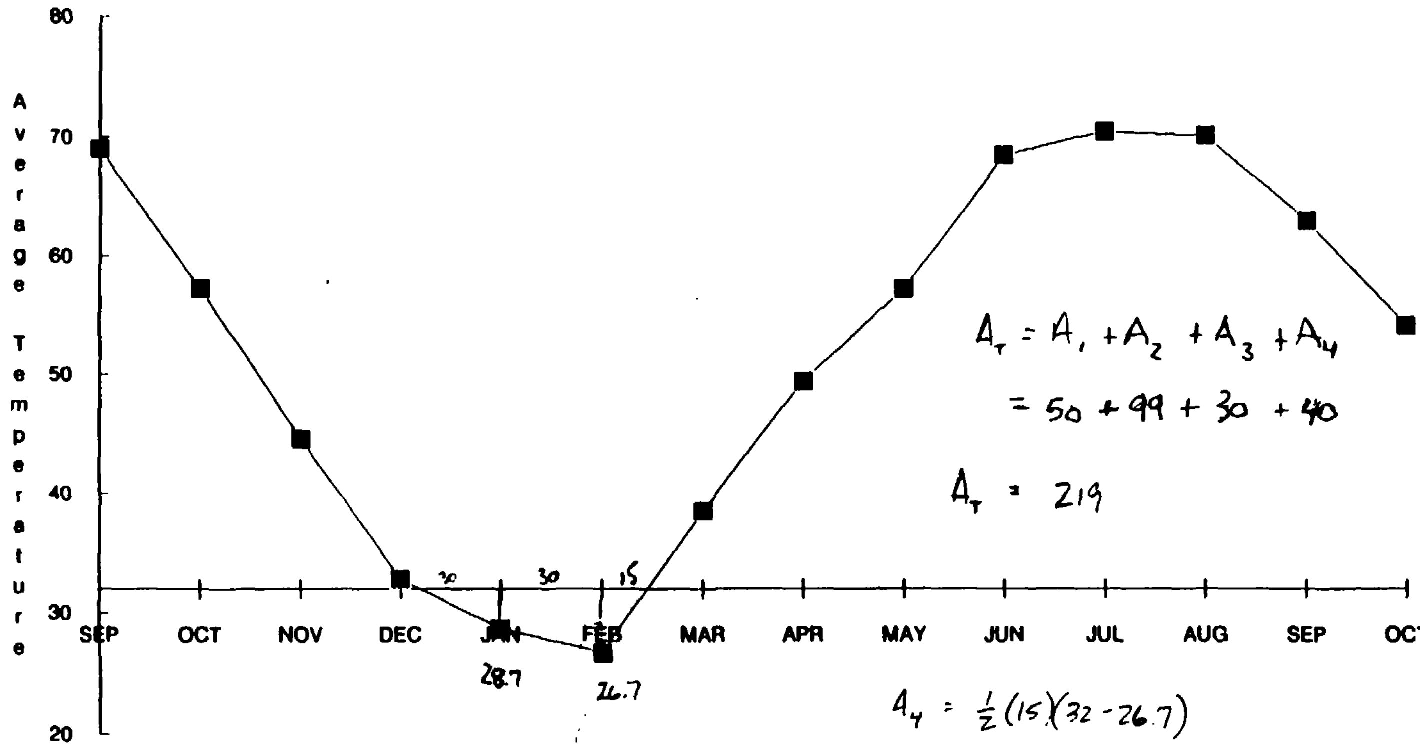
$A_2 = 30(32-29.5)$

$A_2 = 75$

5-7

219 Degree Days

1961-1962



$A_T = A_1 + A_2 + A_3 + A_4$

$= 50 + 99 + 30 + 40$

$A_T = 219$

$A_4 = \frac{1}{2}(15)(32 - 26.7)$

$A_4 = 40$

$A_3 = \frac{1}{2}(30)(28.7 - 26.7)$

$A_3 = 30$

$A = \frac{1}{2}(30)(32 - 28.7)$

$A_1 = 50$

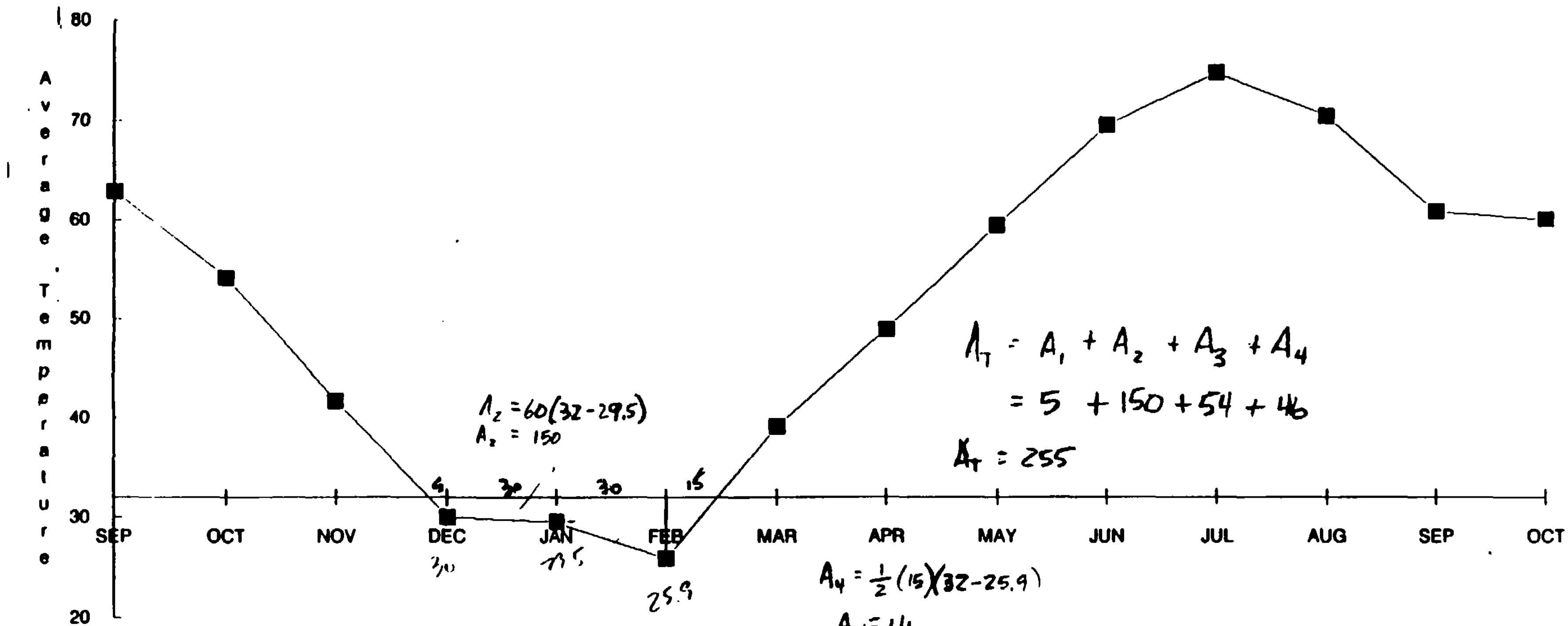
$A = 30(32 - 28.7)$

$A_2 = 99$

5-88

255 Degree Days

1962-1963



$A_2 = 60(32-29.5)$
$A_2 = 150$

$A_T = A_1 + A_2 + A_3 + A_4$
$= 5 + 150 + 54 + 46$
$A_T = 255$

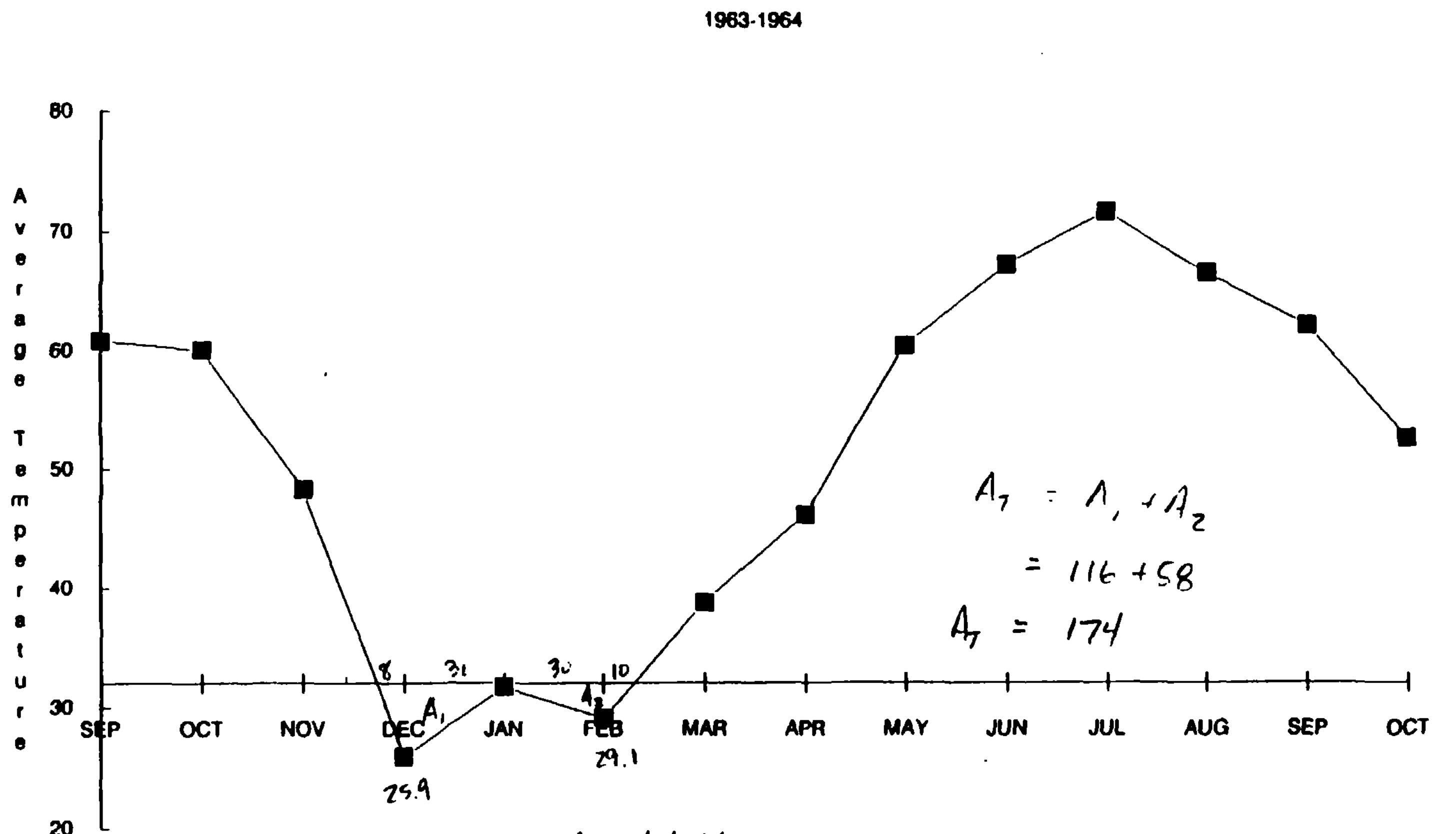
$A_4 = \frac{1}{2}(15)(32-25.9)$
$A_4 = 46$

$A_1 = \frac{1}{2}(5)(32-30)$
$A_1 = 5$

$A_3 = \frac{1}{2}(30)(29.5-25.9)$
$A_3 = 54$

5-9

174 Degree Days



$A_1 = \frac{1}{2}(38)(32-25.9)$

$A_1 = 116$

$A_2 = \frac{1}{2}(40)(32-29.1)$

$A_2 = 58$

5-10

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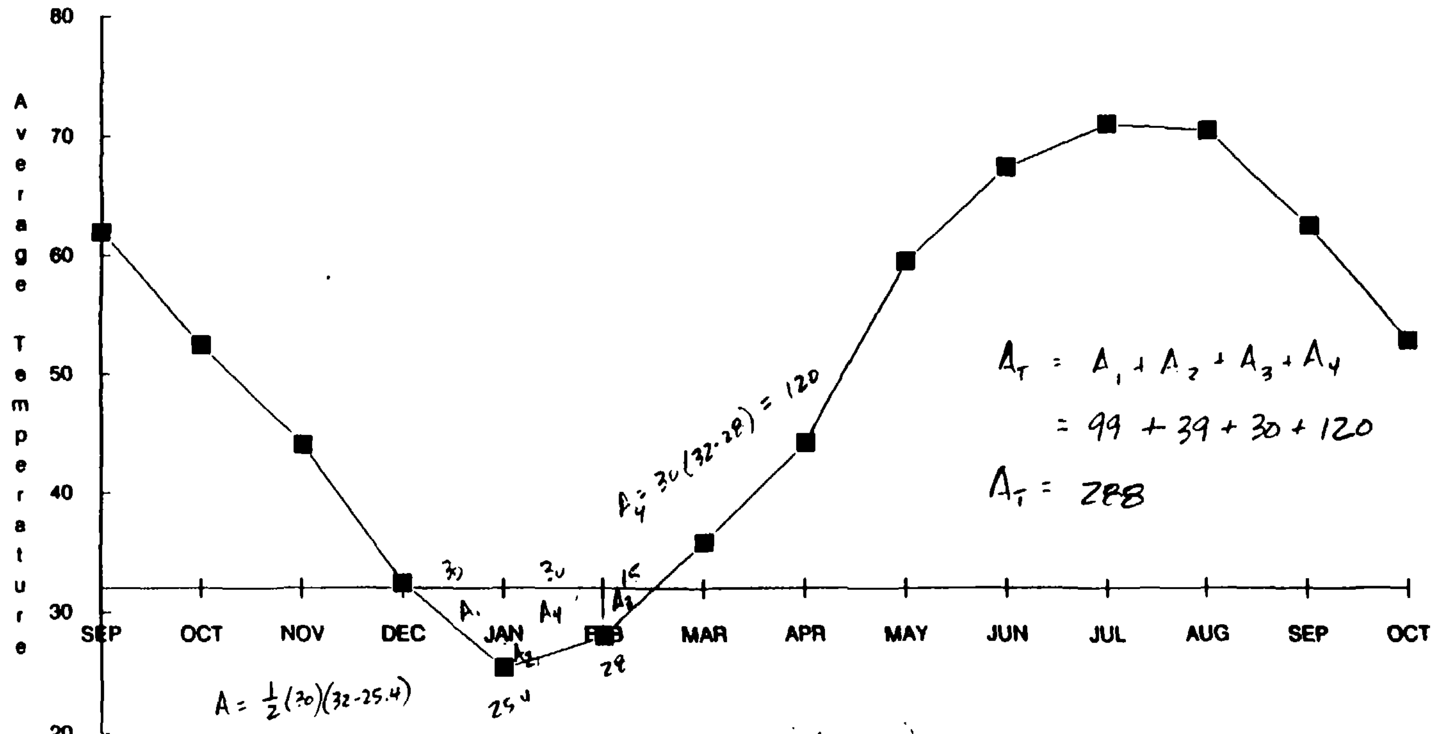
288 Degree Days

1964-1965



$A_T = A_1 + A_2 + A_3 + A_4$

$= 99 + 39 + 30 + 120$

$A_T = 288$

$A = \frac{1}{2}(30)(32-25.4)$

$A_1 = 99$

$A = \frac{1}{2}(2.6)(30)$

$A_2 = 39$

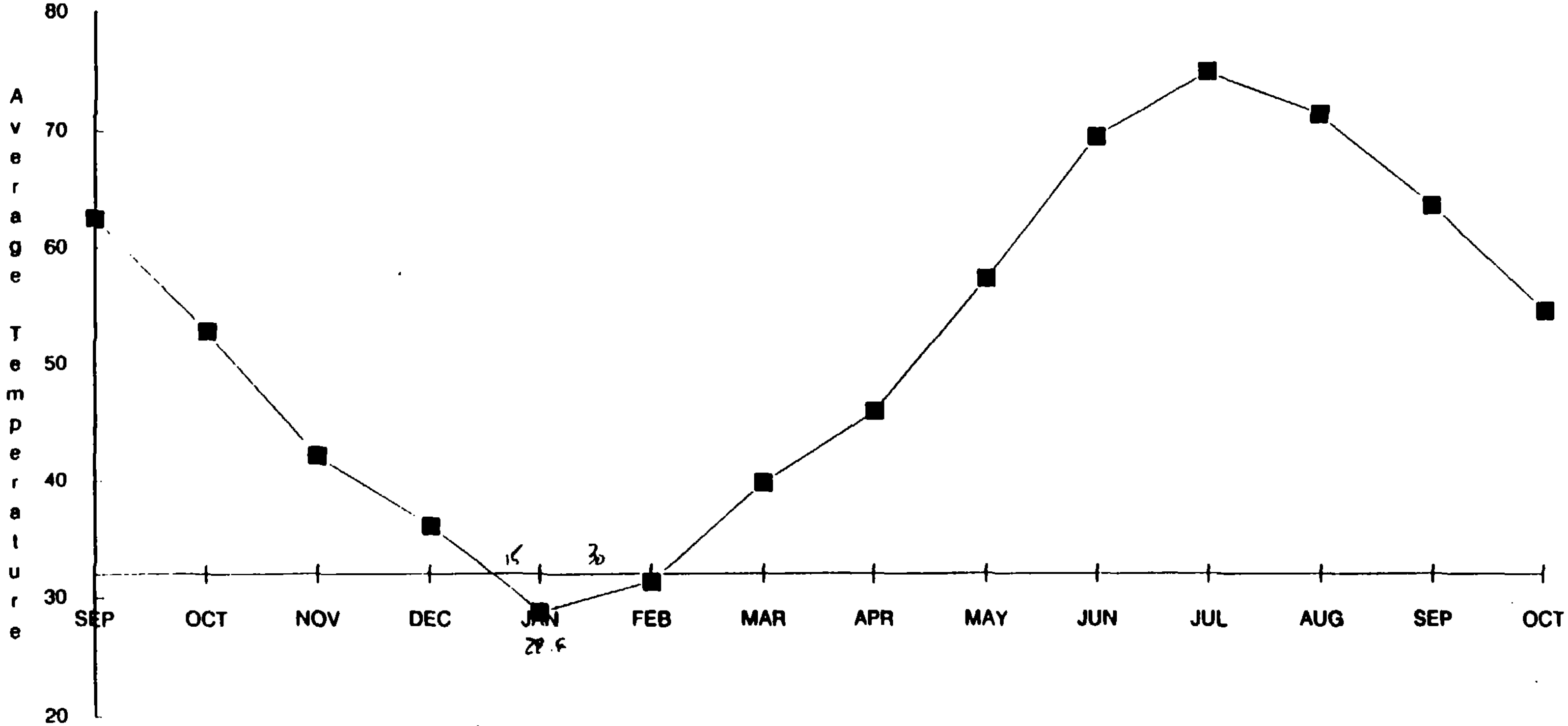
$A = \frac{1}{2}(15)(32-28)$

$A_3 = 30$

CanonieEnvironmental

72 Degree Days

1965-1966



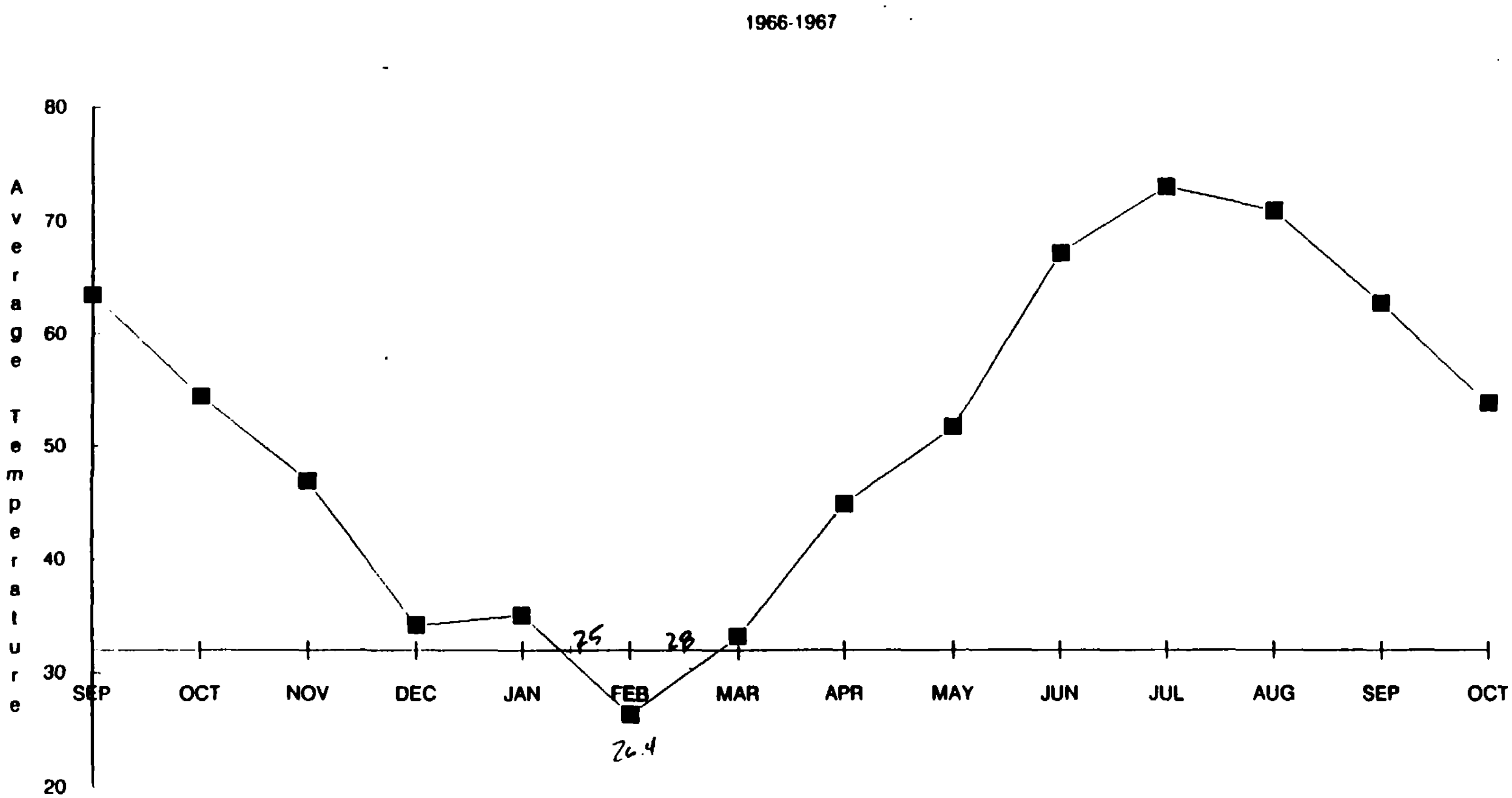
A: $\frac{1}{2}(45)(32 - 28.8)$

A: 72

5-12

Canonie Environmental

149 Degree Days

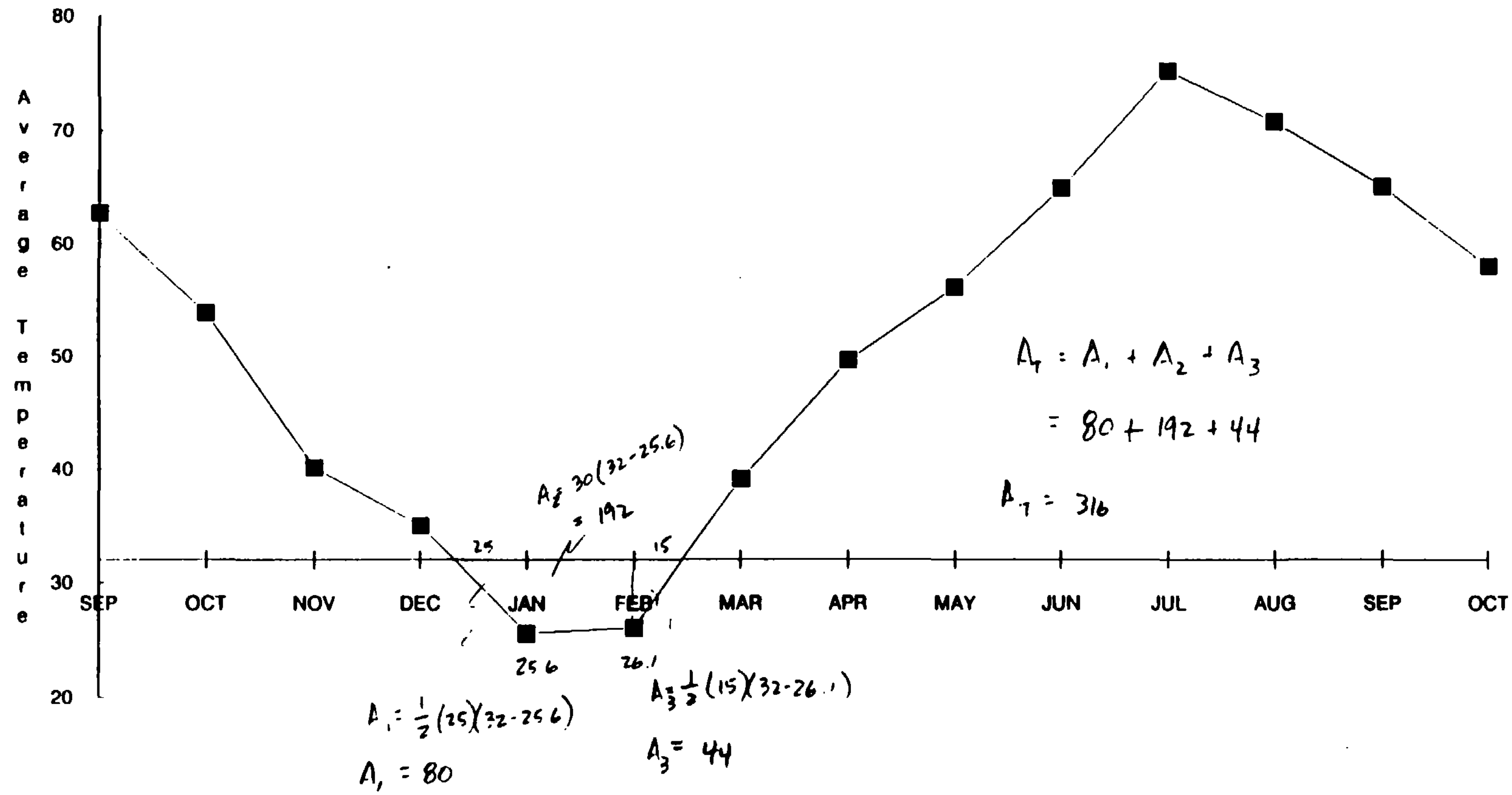


$A = \frac{1}{2}(53)(32 - 26.4) = 149$

(5-13)

316 Degree Days

1967-1968

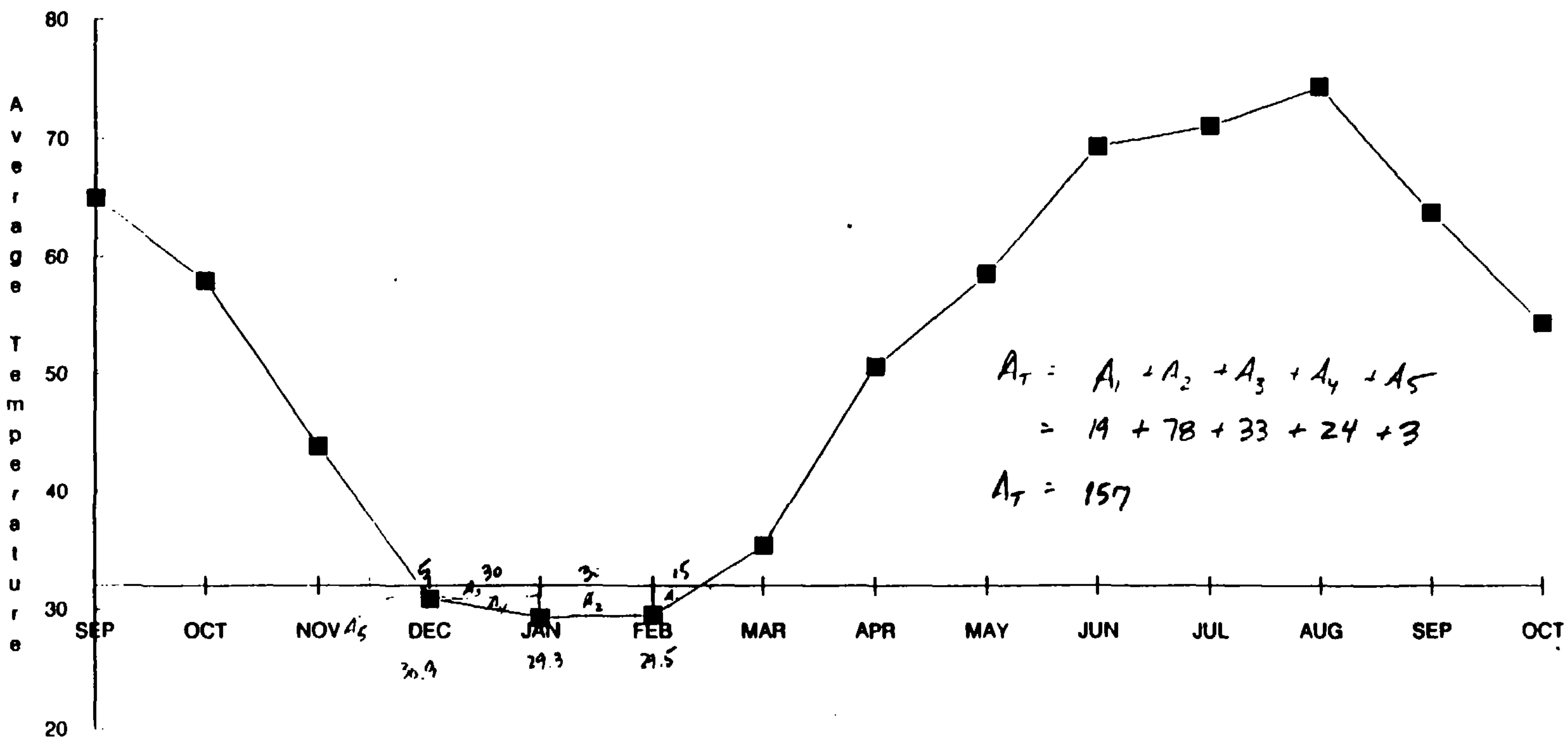


5-14

CanonieEnvironmental

157 Degree Days

1968-1969



$A_T = A_1 + A_2 + A_3 + A_4 + A_5$
$= 19 + 78 + 33 + 24 + 3$
$A_T = 157$

$A_3 = 30(32-30.9)$

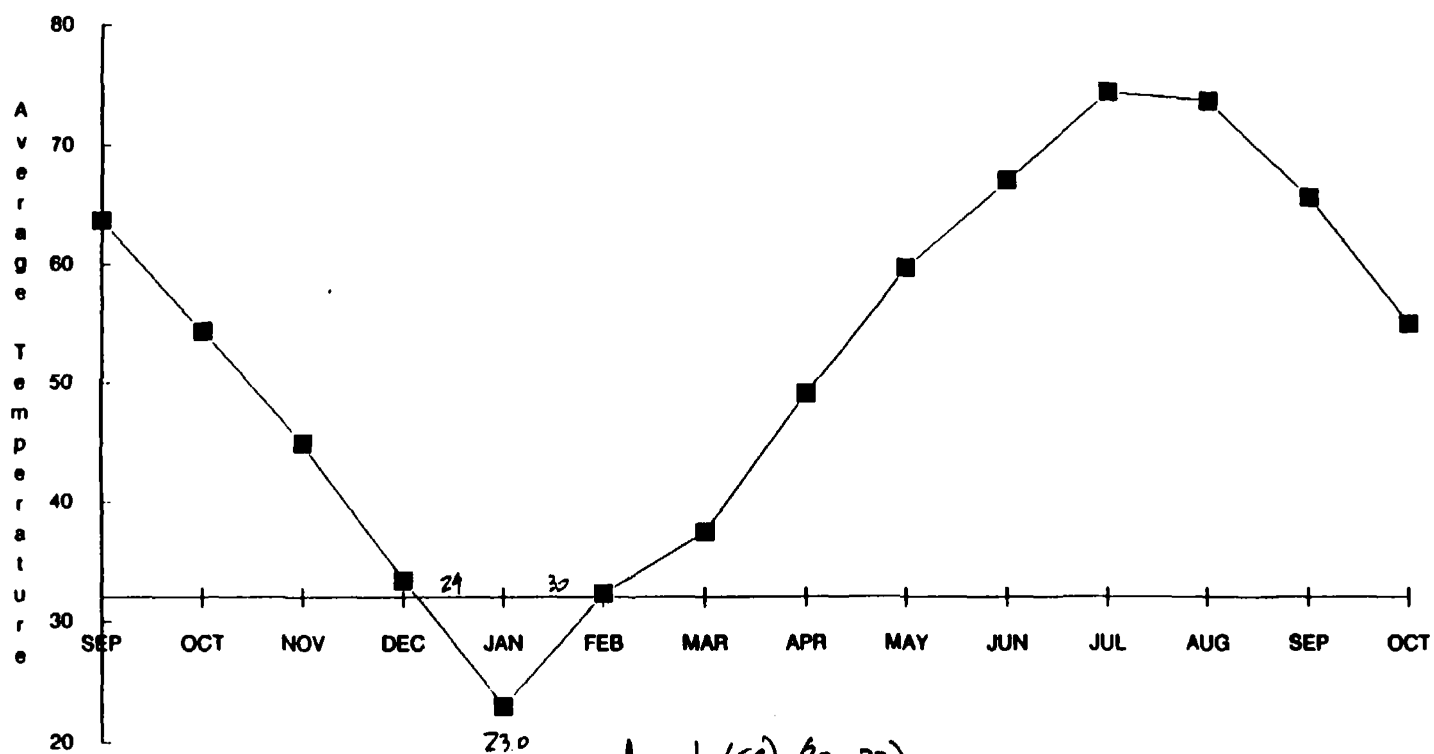
$A_1 = \frac{1}{2}(15)(32-29.5)$ $A_2 = 30(32-29.4)$ $A_3 = 33$ $A_4 = \frac{1}{2}(30)(30.9-29.3)$ $A_5 = \frac{1}{2}(5)(32-30.9)$

$A_1 = 19$ $A_2 = 78$ $A_4 = 24$ $A_5 = 3$

CanonieEnvironmental

266 Degree Days

1969-1970



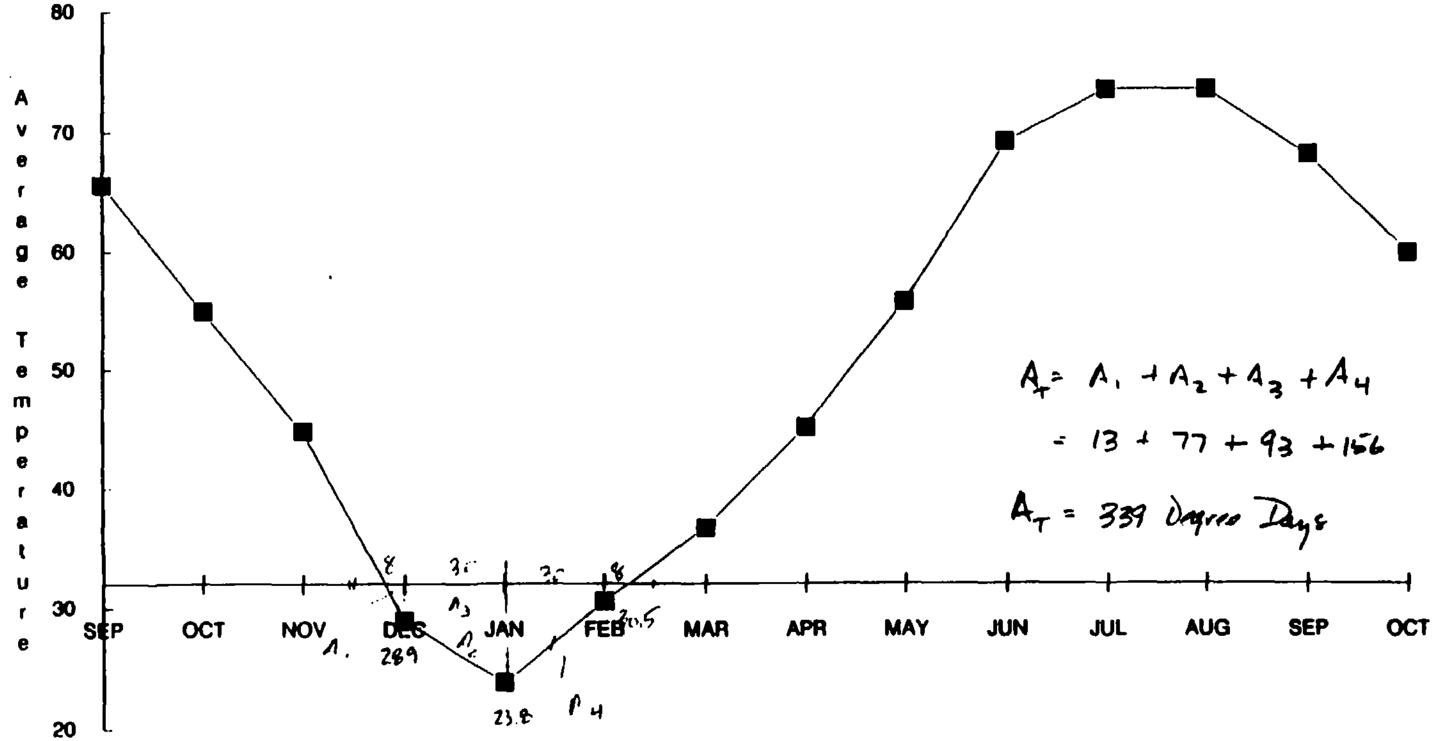
A: $\frac{1}{2}(59)(32-23)$

$= 266$

5-16

339 Degree Days

1970-1971



$A_T = A_1 + A_2 + A_3 + A_4$

$= 13 + 77 + 93 + 156$

$A_T = 339$ Degree Days

$A_1 = \frac{1}{2}(8)(32-28.9)$

$A_1 = 13$

$A_2 = \frac{1}{2}(30)(28.9-23.8)$

$A_2 = 77$

$A_3 = 30(32-28.9)$

$A_3 = 93$

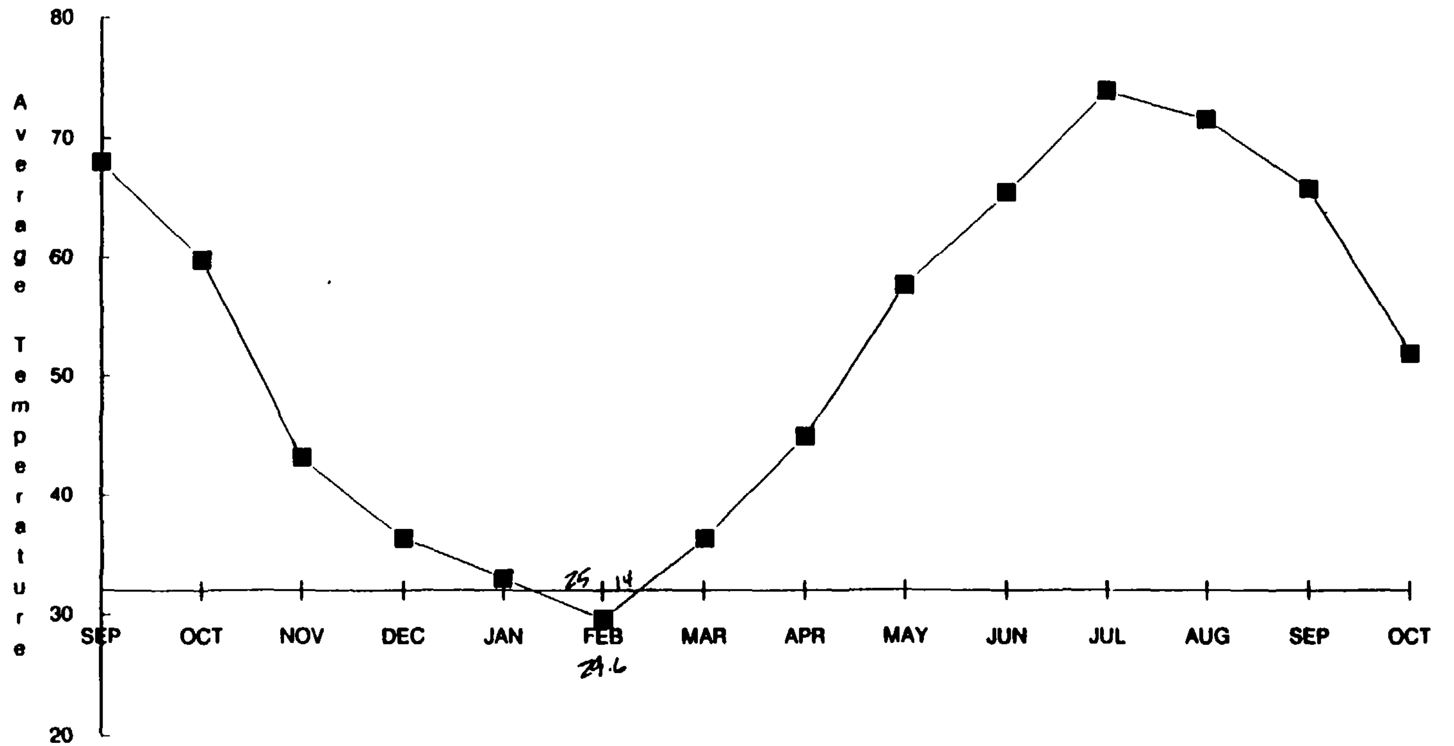
$A_4 = \frac{1}{2}(38)(32-23.8)$

$A_4 = 156$

5-17

CanonieEnvironmental

1971-1972 47 Degree Days

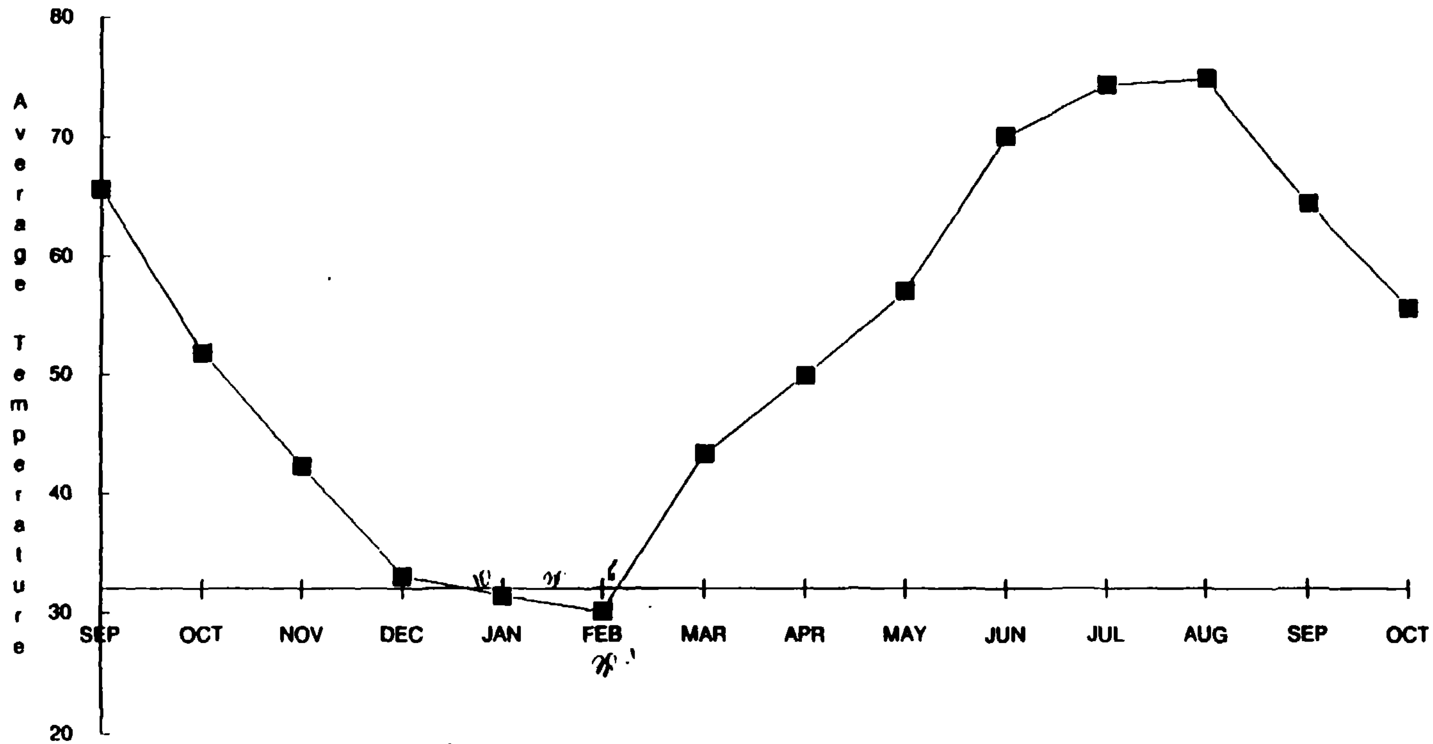


$A = \frac{1}{2}(39)(32-29.6)$

$= 47$

5-18

1972-1973 43 Degree Days

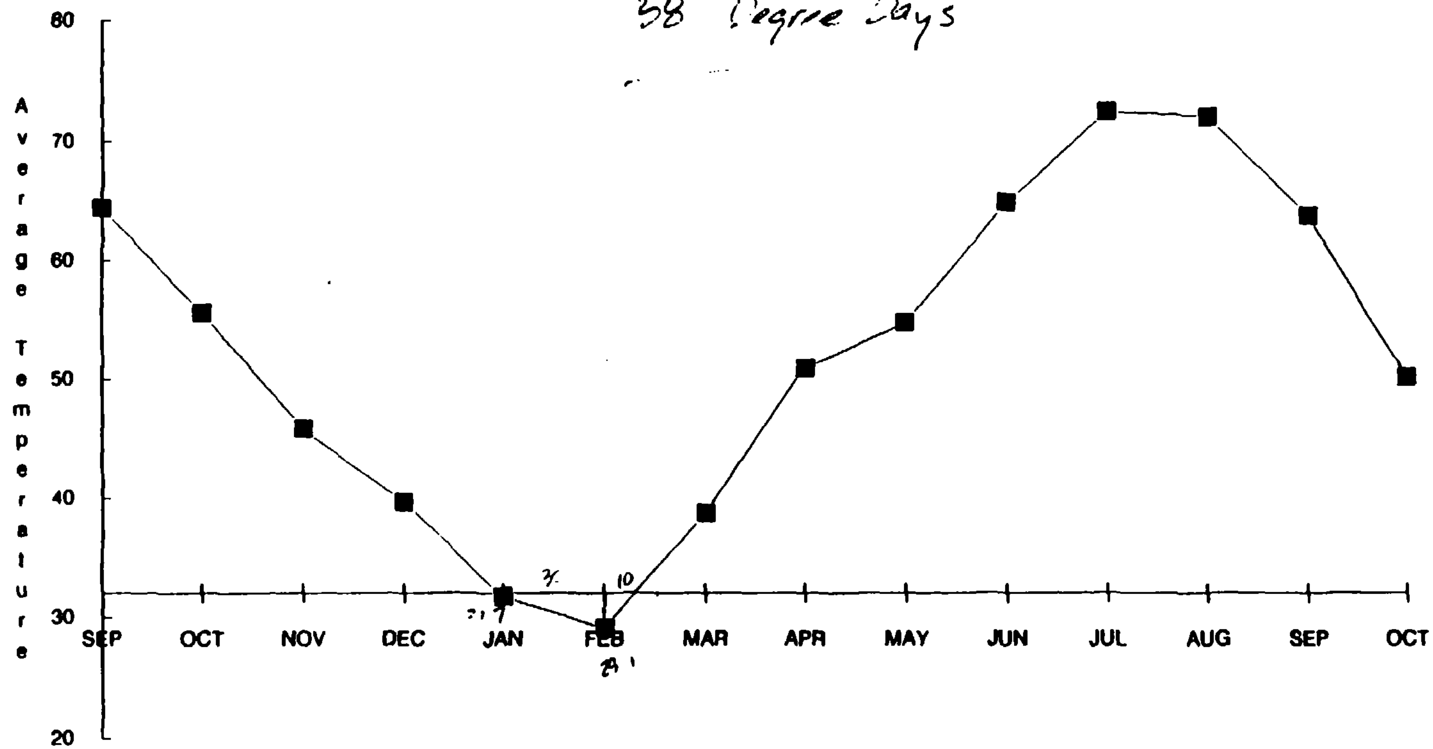


$A = \frac{1}{2}(45)(32-30.1)$

$A = 43$

1973-1974

58 Degree Days

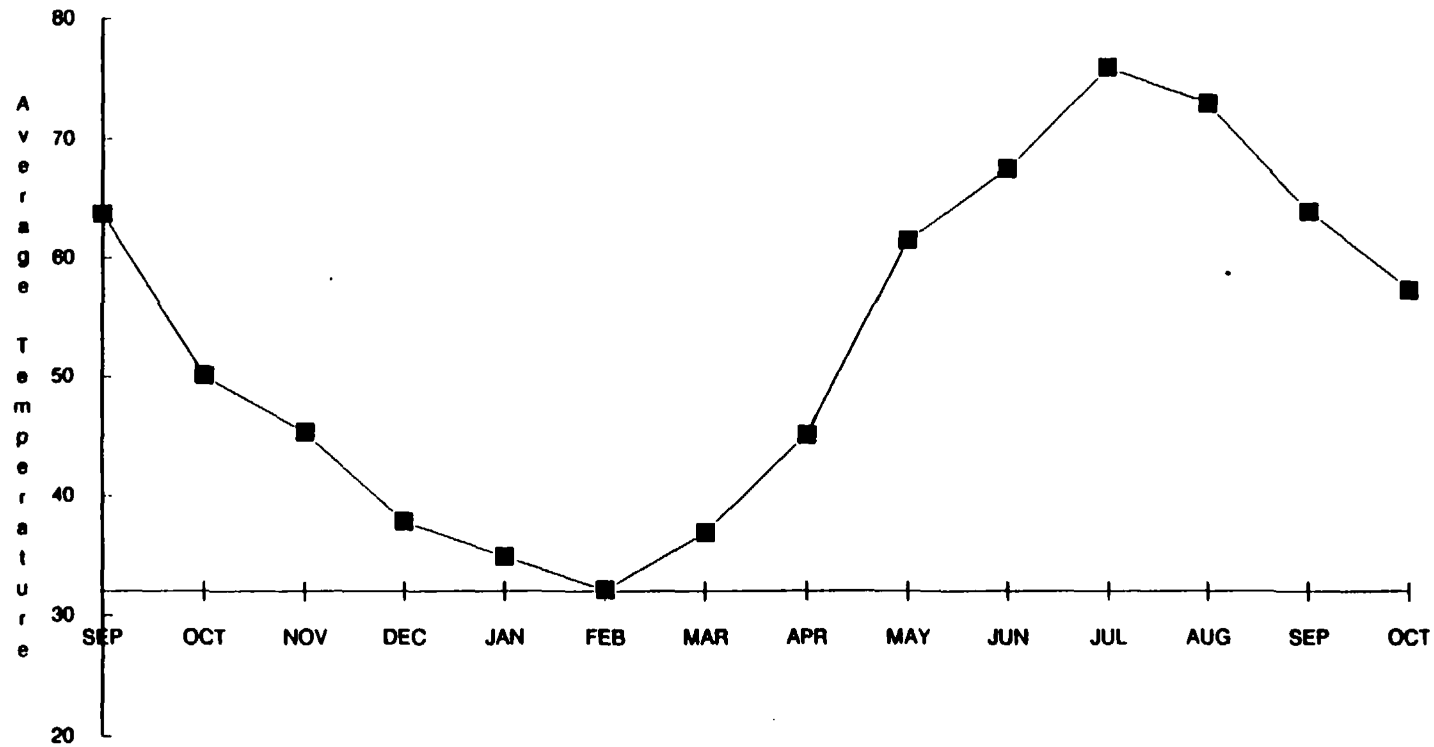


$A = \frac{1}{2}(40)(32 - 29.1)$

$A = 58$

5-26

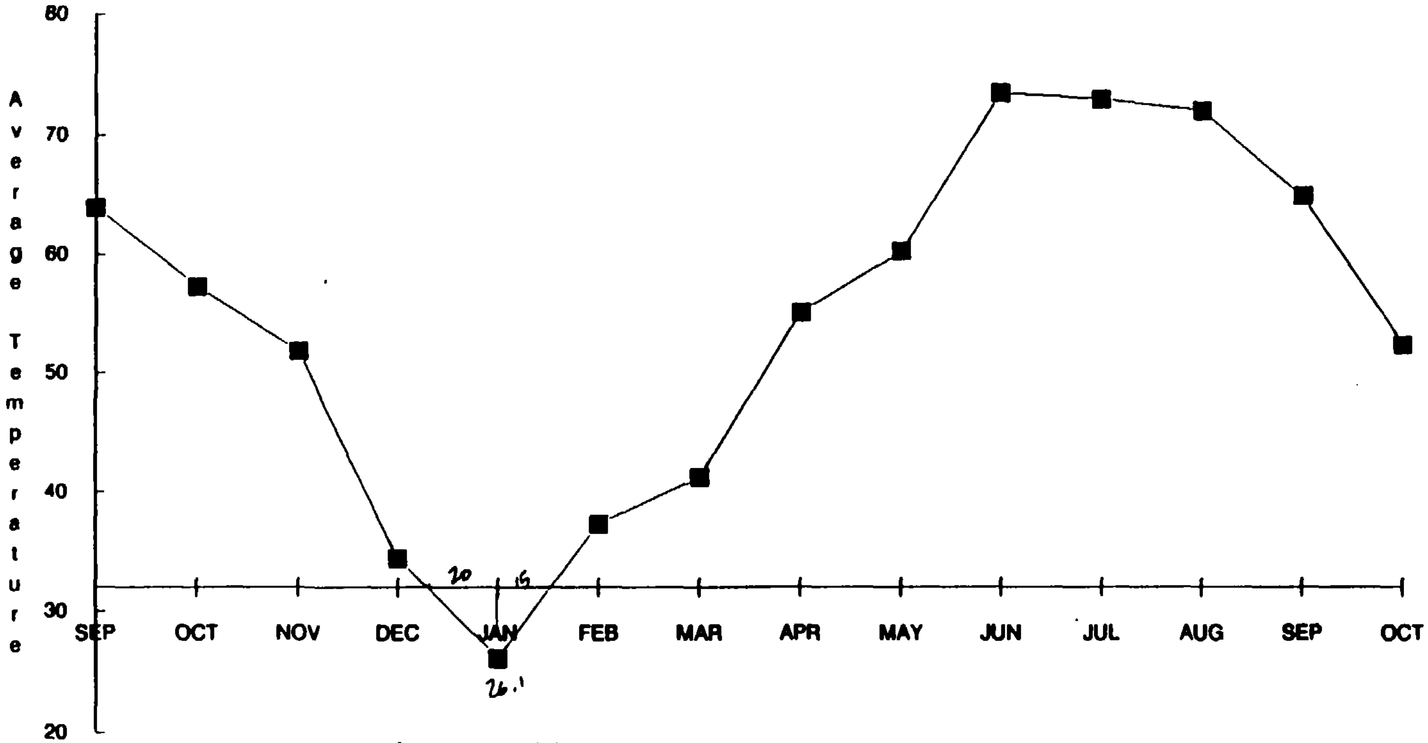
1974-1975 (1)



CanonieEnvironmental

104 Degree Days

1975-1976



$A = \frac{1}{2}(35)(32-26.1)$

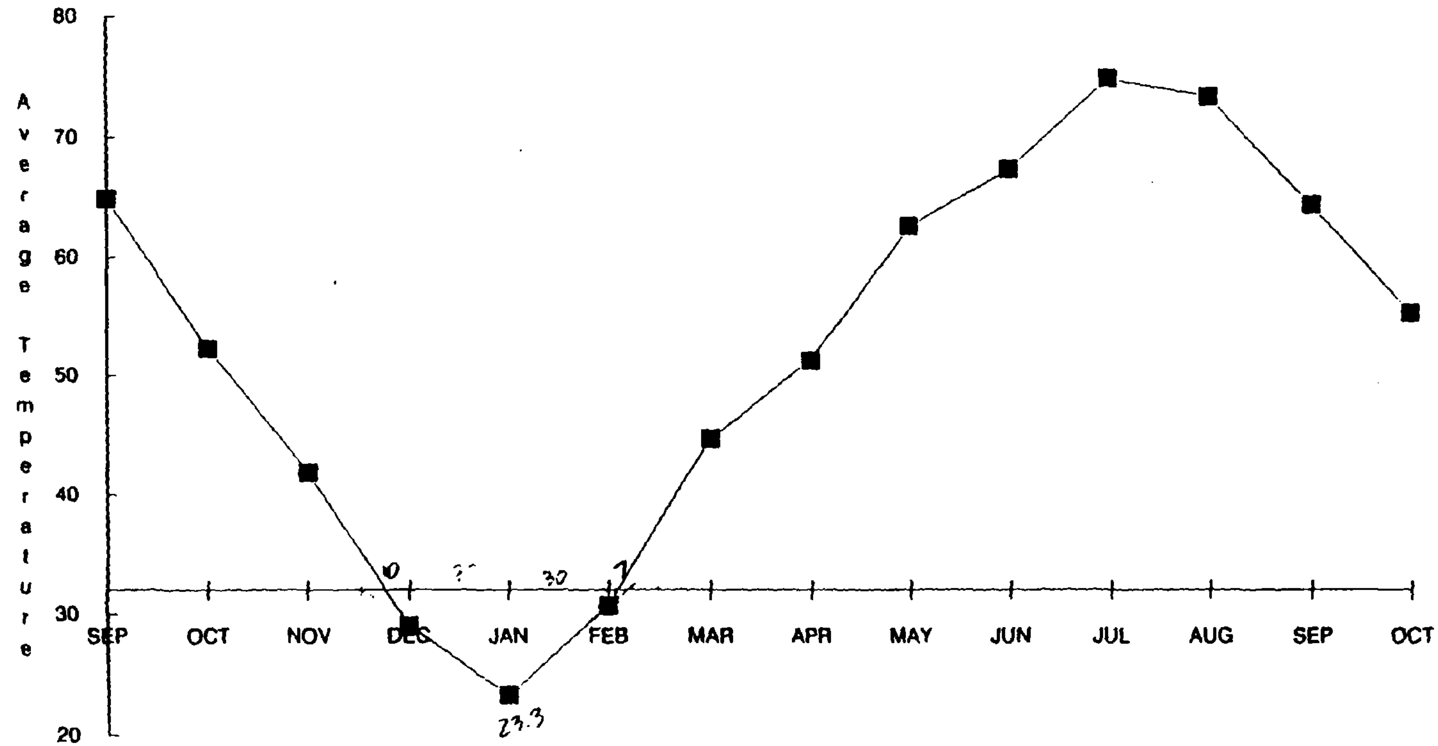
$A = 104$

5-22

CanonieEnvironmental

335 Degree Days

1976-1977



$A = \frac{1}{2}(77)(32-23.3)$

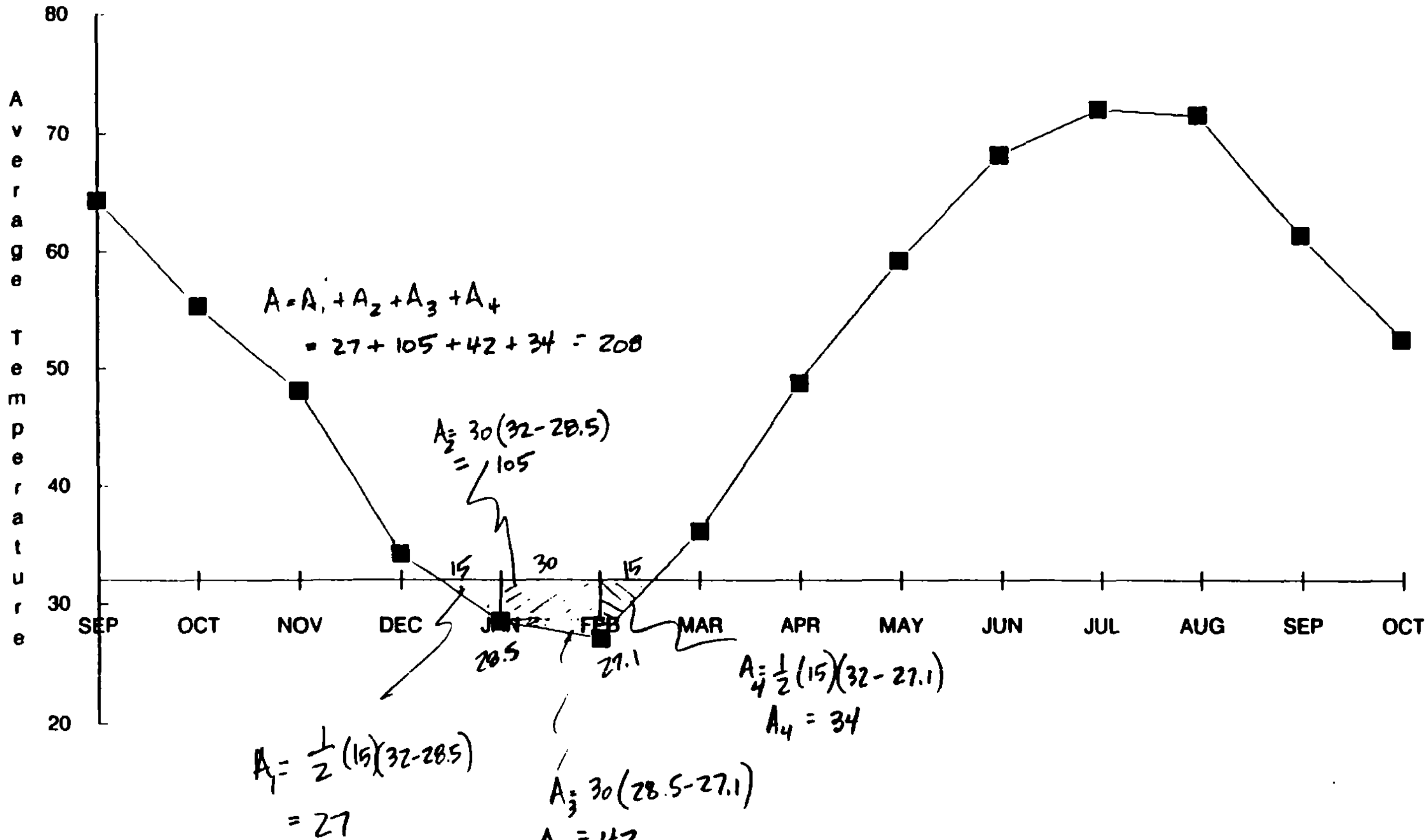
$A = 335$

5-23

Canonie Environmental

208 Degree Days

1977-1978

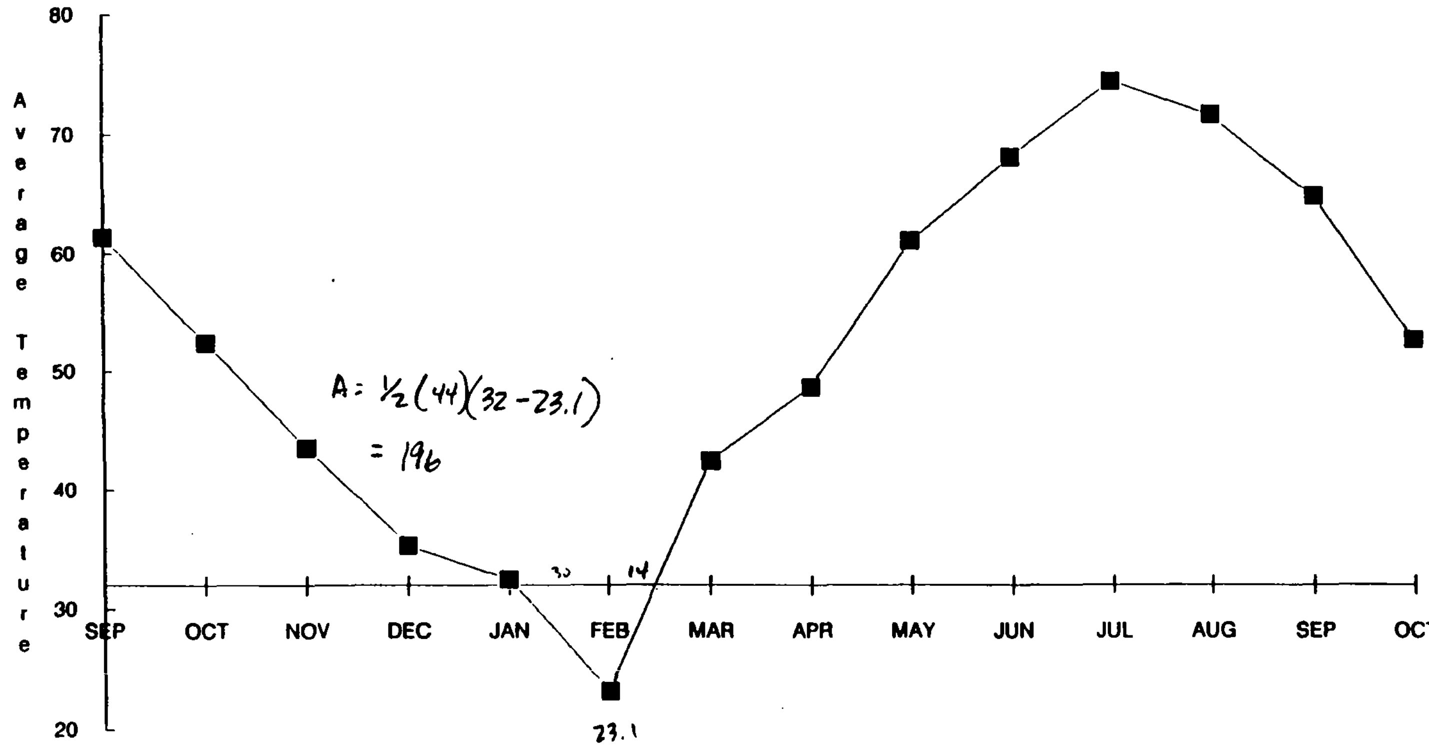


5-24

CanonieEnvironmental

196 Degree Days

1978-1979

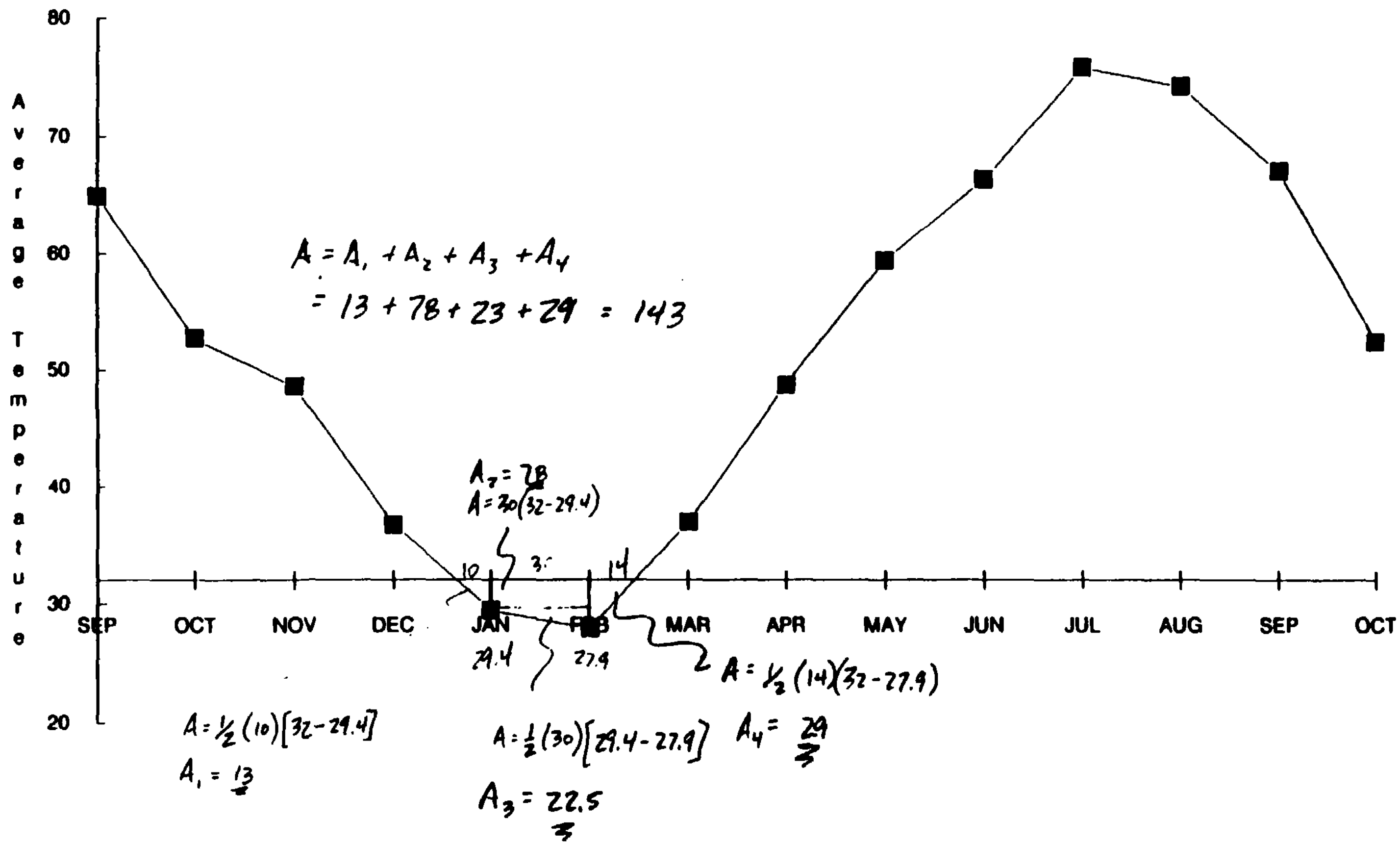


5-25

CanonieEnvironmental

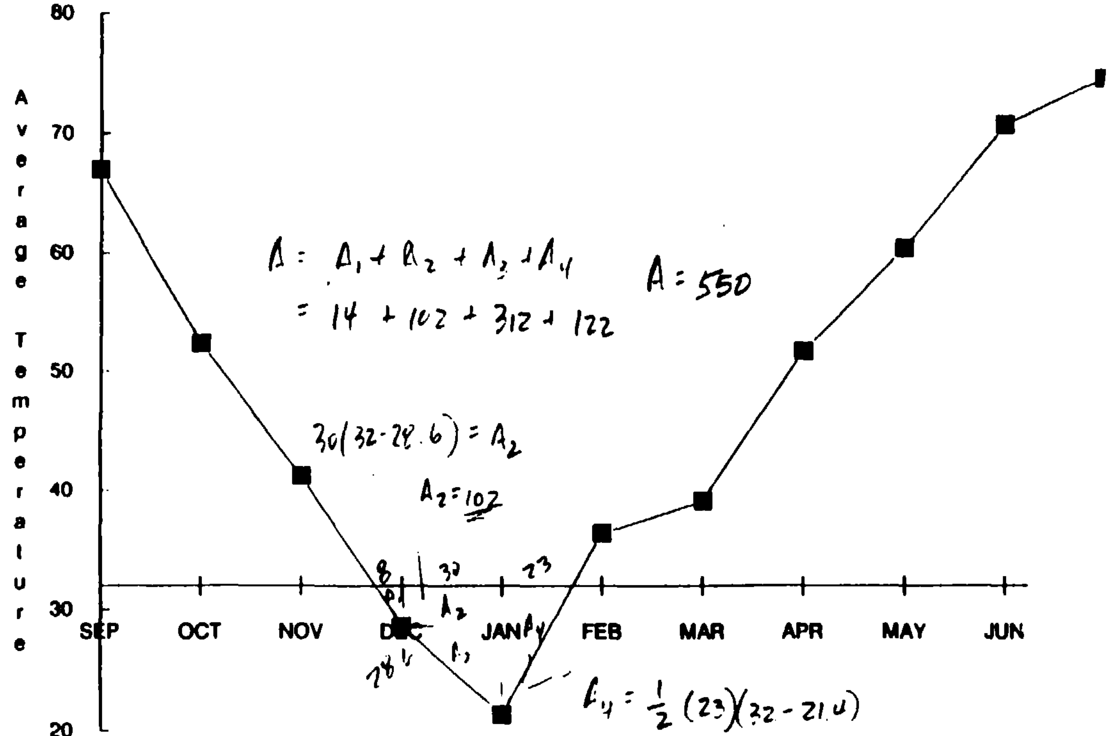
143 Degree Days

1979-1980



CanonieEnvironmental

1980-1981 550° days



$A = A_1 + A_2 + A_3 + A_4$
$= 14 + 102 + 312 + 122$

$A = 550$

$30(32 - 28.6) = A_2$

$A_2 = \underline{102}$

$A_1 = \ldots(32 - 28.6) = \underline{14}$

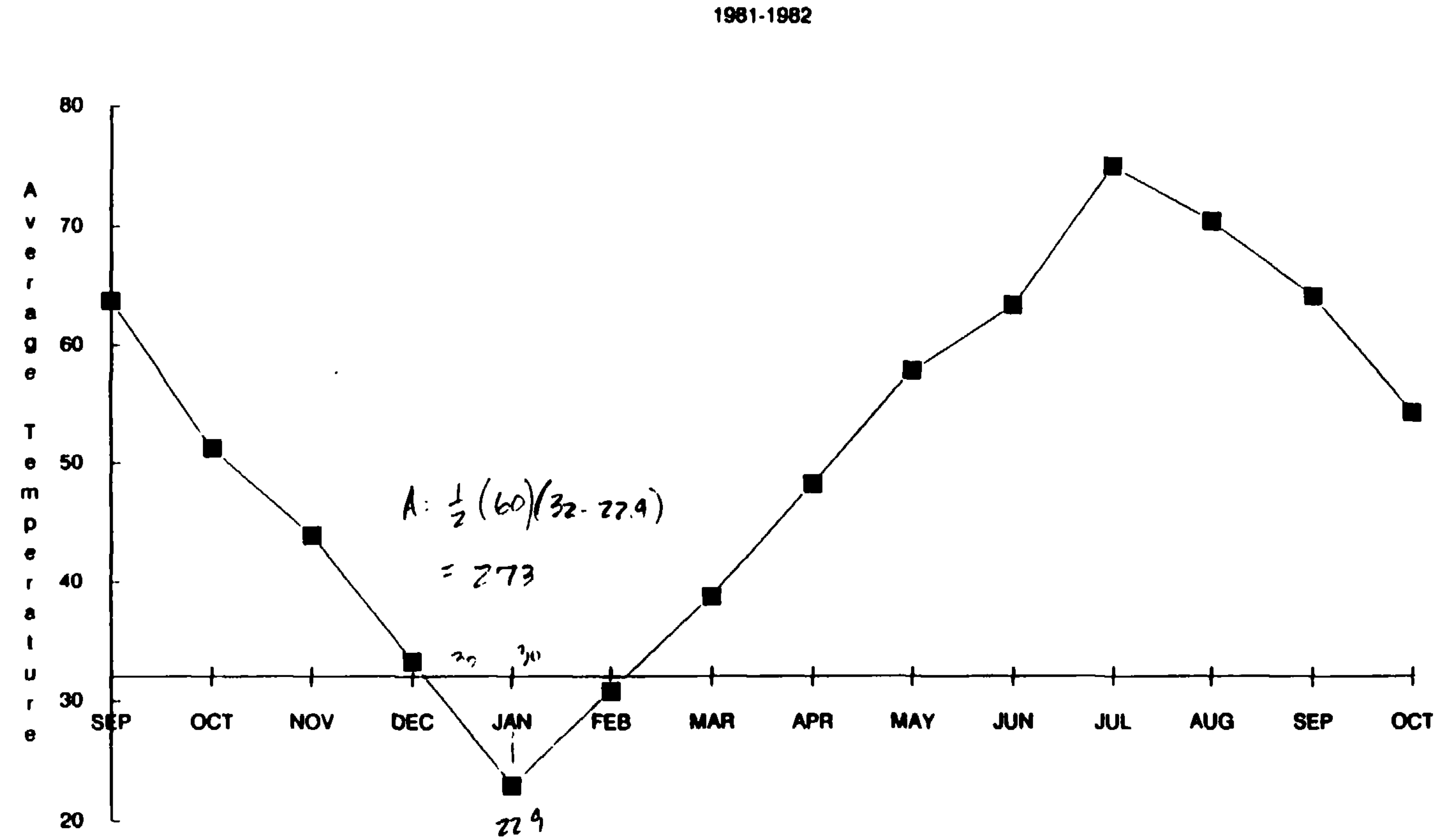
$A_4 = \frac{1}{2}(23)(32 - 21.4)$
$= \underline{122}$

$A_3 = \frac{1}{2}(28.6 - 21.4)(\ldots)$
$A_3 = \underline{312}$

Page 1

5-27

273 °days

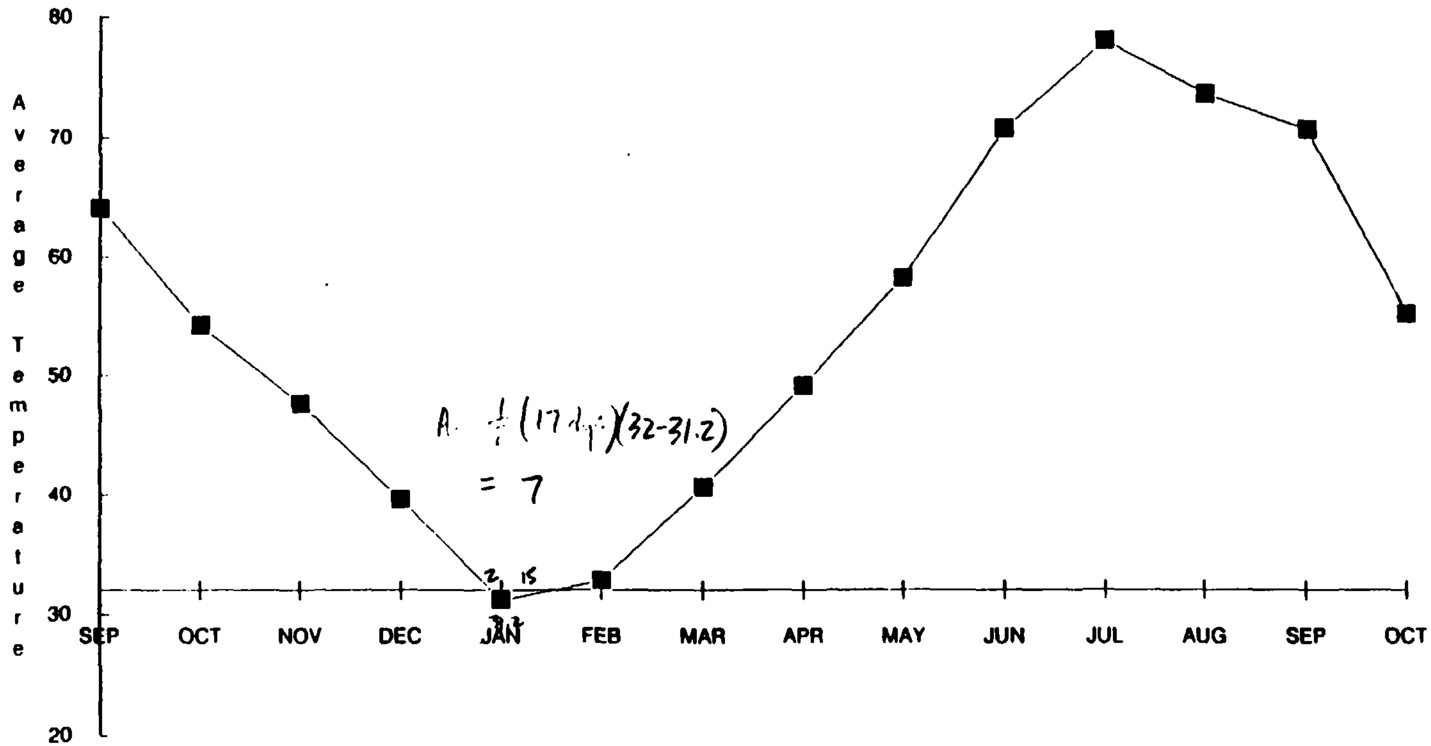


5-28

CanonieEnvironmental

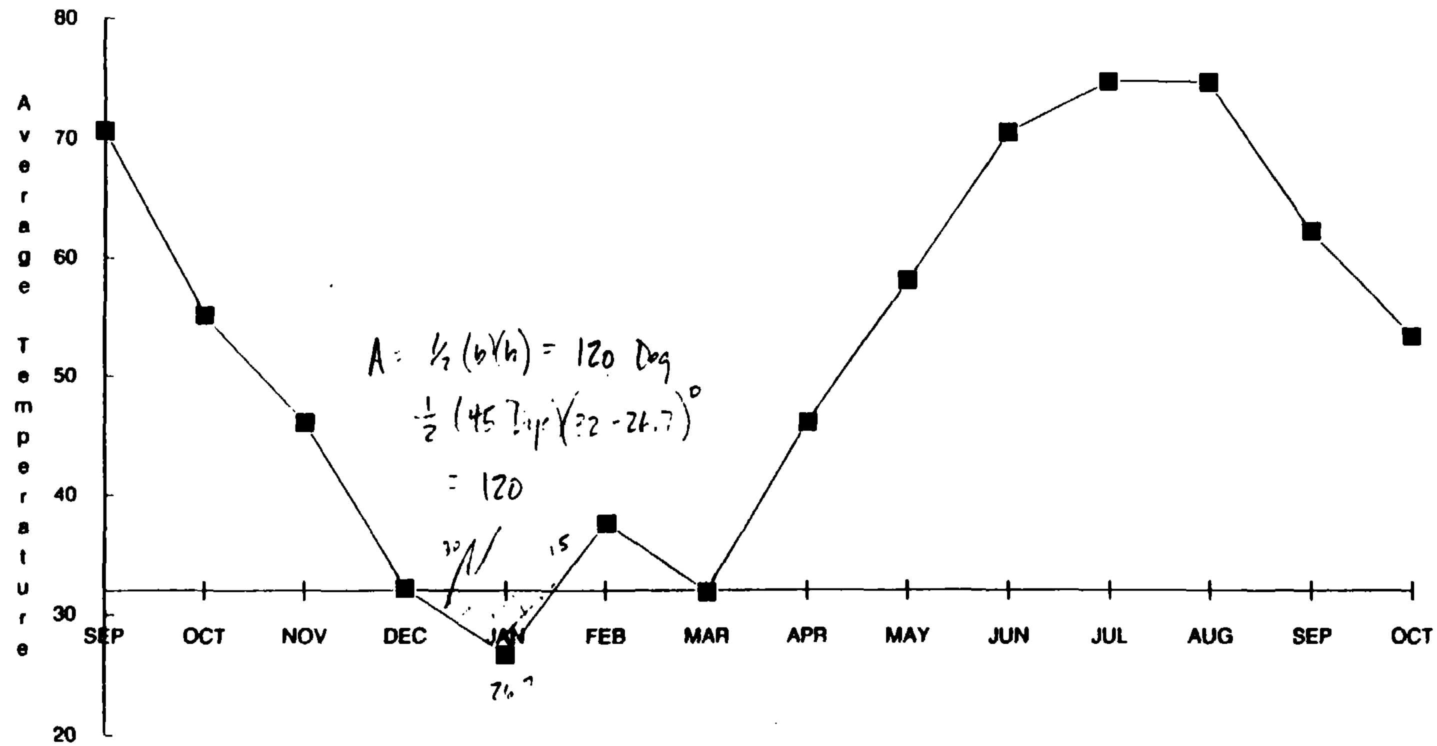
7 Degree Days

1982-1983



5-29

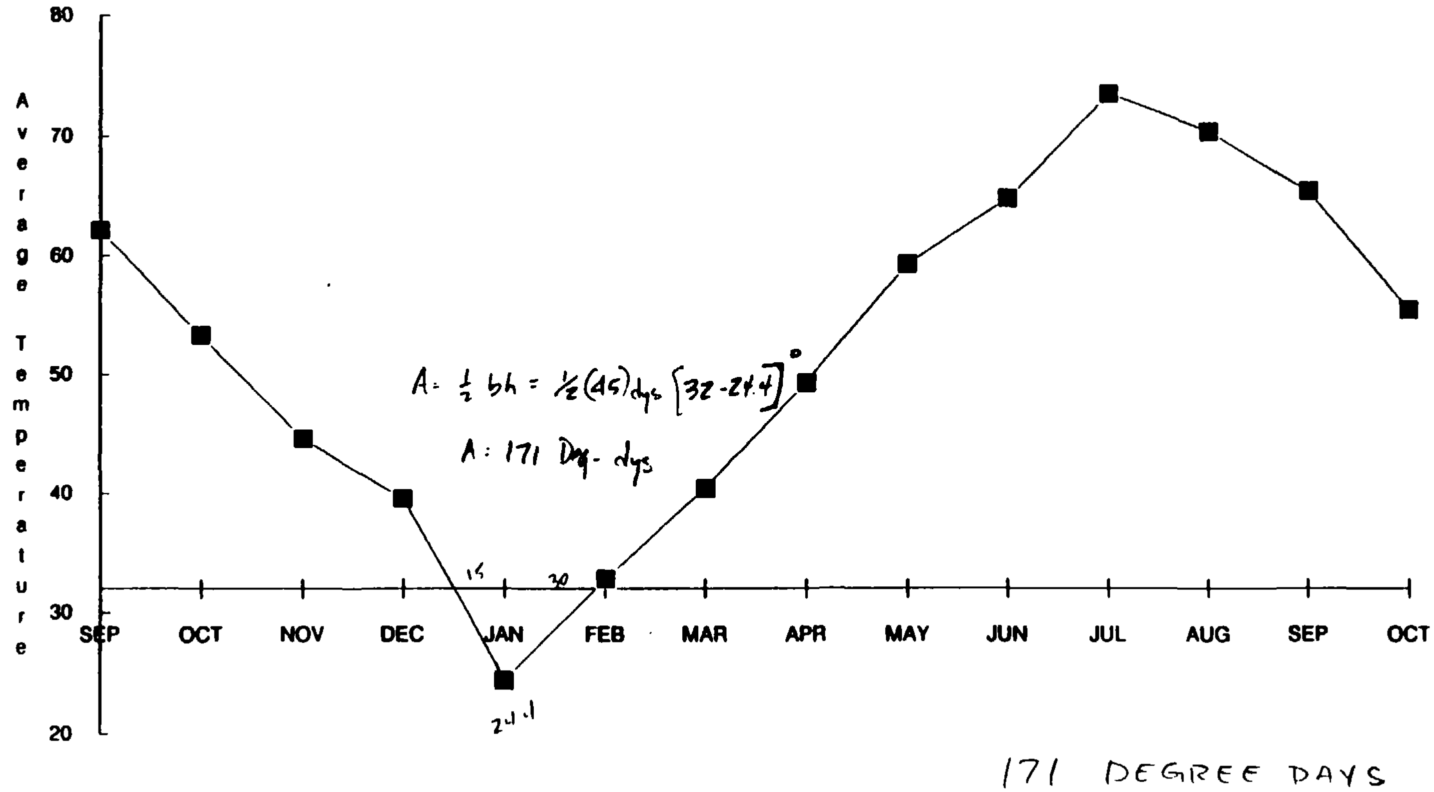
1983-1984



Page 1

5-30

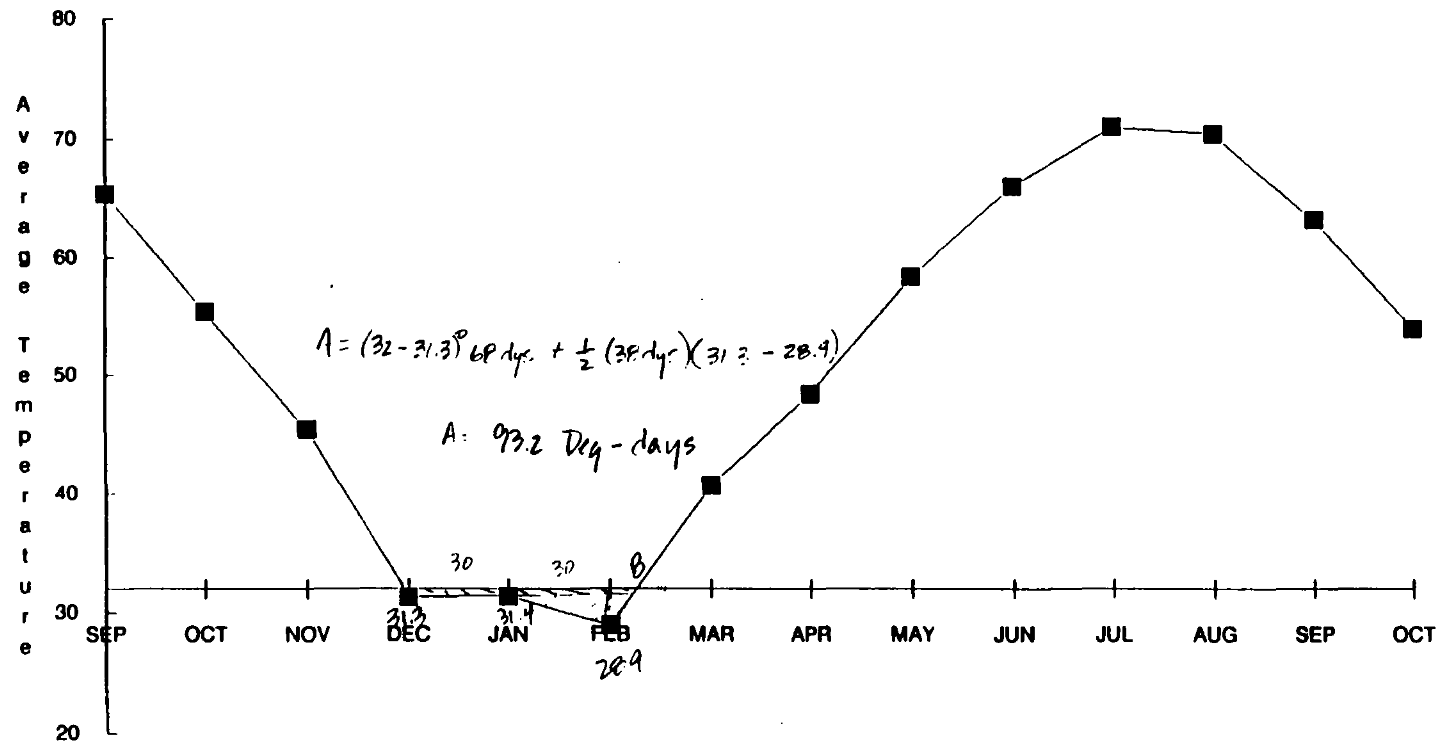
CanonieEnvironmental



5-31

CanonieEnvironmental

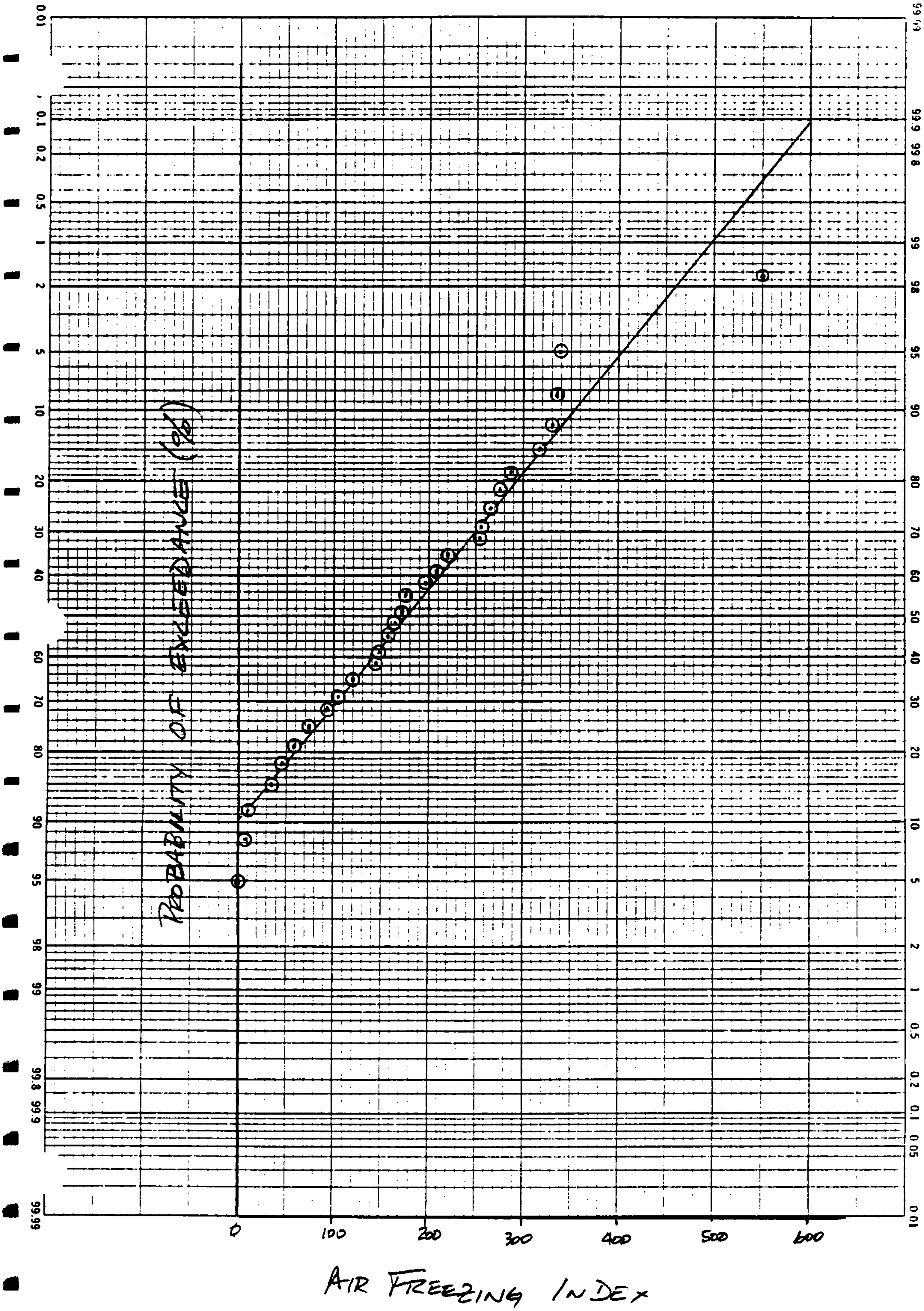
1985-1986



93.2 DEGREE DAYS

5-32

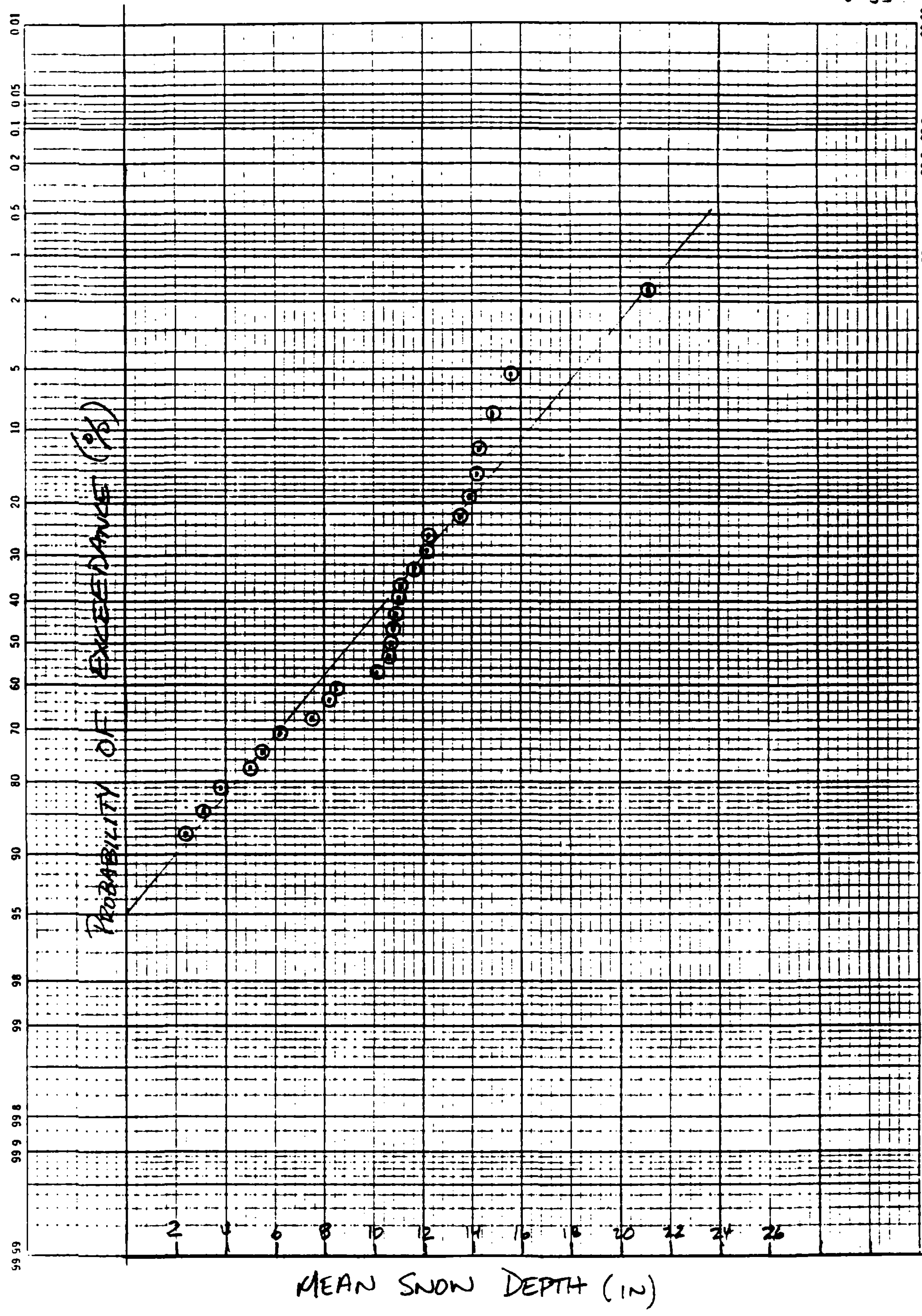
Canonie Environmental


PROBABILITY OF EXCEEDANCE (%)
0
100
200
300
400
500
600
AIR FREEZING INDEX

5-34

YEAR	DEC	JAN	FEB	MAR	AVG	RANK	PROBABILITY OF EXCEEDANCE (%)
1957-1958	0	6.6	23.9	12	10.6	16	53.4
1958-1959	4.6	4.1	10.7	14.6	8.5	18	60.3
1959-1960	5.5	10.2	2.3	22.3	10.1	17	56.9
1960-1961	16.9	18.7	14.9	9	14.9	3	8.6
1961-1962	11.4	2.5	28.7	1.1	10.9	13	43.1
1962-1963	5.3	6.5	4.6	13.6	7.5	20	67.2
1963-1964	17.7	14.4	23.2	7.7	15.8	2	5.2
1964-1965	12.2	22.2	4.7	9.7	12.2	8	25.9
1965-1966	2.3	26.4	12.1	3.3	11.0	12	39.7
1966-1967	9.9	0.5	23.5	22.9	14.2	5	15.5
1967-1968	14.7	17.7	3.4	6.8	10.7	15	50.0
1968-1969	5.1	0.9	41.3	6.1	13.4	7	22.4
1969-1970	12.6	7.4	10.5	18.2	12.2	8	25.9
1970-1971	27.9	12	8.1	7.4	13.9	6	19.0
1971-1972	7.9	7.8	16.5	12.1	11.1	11	36.2
1972-1973	3.3	3.6	2.5	0.3	2.4	26	87.9
1973-1974	0	16	17.8	0.1	8.5	18	60.3
1974-1975	3.6	2.2	17	1.8	6.2	21	70.7
1975-1976	19.3	15	1.4	10.8	11.6	10	32.8
1976-1977	17.2	23.2	5.9	10.7	14.3	4	12.1
1977-1978	5.2	35.9	27.2	16.1	21.1	1	1.7
1978-1979	5.8	10.5	6.6	0	5.7	22	74.1
1979-1980	2	0.4	6.5	3.6	3.1	25	84.5
1981-1981	5.6	11.9	1.9	0.5	5.0	23	77.6
1981-1982	17.6	18	7.6	5.3	12.1	9	29.3
1982-1983	5.5	4.7	22.3	0.2	8.2	19	63.8
1983-1984	2.6	21.1	0.3	19	10.8	14	46.6
1984-1985	3.7	7	10.2	3.7	6.2	21	70.7
1985-1986	1.3	0.8	10.4	2.6	3.8	24	81.0

25% ≅ 6 inches



Probability Scale x 90 Divisions

SNOWFALL (inches)

31 31 28 31

BOSTON, MASSACHUSETTS

AVG DEC-MAR

5-36

SEASON	JULY	AUG	SEP	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUNE	TOTAL
1957-58	0.0	0.0	0.0	T	0.1	T	6.6	23.9	12.0	2.1	T	0.0	44.7
1958-59	0.0	0.0	0.0	0.0	0.1	4.6	4.1	10.7	14.6	T	0.0	0.0	34.1
1959-60	0.0	0.0	0.0	0.0	0.6	5.5	10.2	2.3	22.3	T	0.0	0.0	40.9
1960-61	0.0	0.0	0.0	T	T	16.9	18.7	14.9	9.0	2.0	0.0	0.0	61.5
1961-62	0.0	0.0	0.0	T	0.9	11.4	2.5	28.7	1.1	0.1	0.0	0.0	44.7
1962-63	0.0	0.0	0.0	T	0.9	5.3	6.5	4.6	13.6	T	0.0	0.0	30.9
1963-64	0.0	0.0	0.0	T	0.0	17.7	14.4	23.2	7.7	T	0.0	0.0	63.0
1964-65	0.0	0.0	0.0	T	T	12.2	22.2	4.7	9.7	1.6	0.0	0.0	50.4
1965-66	0.0	0.0	0.0	0.0	T	2.3	26.4	12.1	3.3	T	T	0.0	44.1
1966-67	0.0	0.0	0.0	0.0	0.0	9.9	0.5	23.5	22.9	3.3	T	0.0	60.1
1967-68	0.0	0.0	0.0	0.0	2.2	14.7	17.7	3.4	6.8	0.0	0.0	0.0	44.8
1968-69	0.0	0.0	0.0	0.0	0.4	5.1	0.9	41.3	6.1	T	0.0	0.0	53.8
1969-70	0.0	0.0	0.0	T	T	12.6	7.4	10.5	18.2	0.1	0.0	0.0	48.8
1970-71	0.0	0.0	0.0	T	T	27.9	12.0	8.1	7.4	1.9	0.0	0.0	57.3
1971-72	0.0	0.0	0.0	0.0	2.8	7.9	7.8	16.5	12.1	0.4	0.0	0.0	47.5
1972-73	0.0	0.0	0.0	T	0.6	3.3	3.6	2.5	0.3	T	0.0	0.0	10.3
1973-74	0.0	0.0	0.0	0.0	0.0	T	16.0	17.8	0.1	3.0	0.0	0.0	36.9
1974-75	0.0	0.0	0.0	0.0	2.0	3.6	2.2	17.0	1.8	1.0	0.0	0.0	27.6
1975-76	0.0	0.0	0.0	T	0.1	19.3	15.0	1.4	10.8	T	0.0	0.0	46.6
1976-77	0.0	0.0	0.0	0.0	1.0	17.2	23.2	5.9	10.7	T	0.5	0.0	58.5
1977-78	0.0	0.0	0.0	0.0	0.7	5.2	35.9	27.2	16.1	T	0.0	0.0	85.1
1978-79	0.0	0.0	0.0	0.0	4.2	5.8	10.5	6.6	T	0.4	0.0	0.0	27.5
1979-80	0.0	0.0	0.0	0.2	T	2.0	0.4	6.5	3.6	T	0.0	0.0	12.7
1980-81	0.0	0.0	0.0	0.0	2.4	5.6	11.9	1.9	0.5	0.0	0.0	0.0	22.3
1981-82	0.0	0.0	0.0	0.0	T	17.6	18.0	7.6	5.3	13.3	0.0	0.0	61.8
1982-83	0.0	0.0	0.0	0.0	T	5.5	4.7	22.3	0.2	T	0.0	0.0	32.7
1983-84	0.0	0.0	0.0	0.0	T	2.6	21.1	0.3	19.0	T	0.0	0.0	43.0
1984-85	0.0	0.0	0.0	0.0	T	3.7	7.0	10.2	3.7	2.0	0.0	0.0	26.6
1985-86	0.0	0.0	0.0	0.0	3.0	1.3	0.8	10.4	2.6	T	0.0	0.0	18.1
1986-87	0.0	0.0	0.0	0.0	3.5	3.4							
Record Mean	0.0	0.0	0.0	T	1.2	[illegible]	12.2	11.5	7.6	0.9	T	0.0	41.1

See Reference Notes on Page 6B.

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REFERENCE NOTES

BOSTON, MASSACHUSETTS

GENERAL
T TRACE AMOUNT
BLANK ENTRIES DENOTE MISSING/UNREPORTED DATA
# INDICATES A STATION OR INSTRUMENT RELOCATION
SEE STATION LOCATION TABLE ON PAGE 9

SPECIFIC
PAGE 2
[illegible] INCLUDES LAST DAY OF PREVIOUS MONTH

PAGE [illegible]
[illegible] LENGTH OF RECORD IN YEARS, ALTHOUGH
INDIVIDUAL MONTHS MAY BE MISSING
[illegible] THAN [illegible]
NORMAL BASED ON THE [illegible] RECORD PERIOD
EXTREME [illegible] DATE ARE THE MOST RECENT OCCURRENCE
WIND DIR [illegible] NUMERALS SHOW TENS OF DEGREES CLOCKWISE
FROM TRUE NORTH [illegible] INDICATES CALM
RESULTANT DIRECTIONS ARE GIVEN TO WHOLE DEGREES

EXCEPTIONS

PAGES 4A, 4B, 6A
RECORD MEANS ARE THROUGH THE CURRENT YEAR,
BEGINNING IN 1872 FOR TEMPERATURE
1871 FOR PRECIPITATION
1936 FOR SNOWFALL

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 598 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Snow	10.0	0	15	9.50	0.18	0.0	0	0
2: Course-grained	6.0	18	95	28.62	0.75	2394.0	568	568
3: Course-grained	0.4	10	110	26.95	1.20	1584.0	26	594

End of Frost Penetration

TOTAL FROST PENETRATION = 16.4 inches

5-37

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 598 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Snow	8.0	0	15	9.50	0.18	0.0	0	0
2: Course-grained	6.0	18	95	28.62	0.75	2394.0	453	453
3: Course-grained	2.6	10	110	26.95	1.20	1584.0	145	598

End of Frost Penetration

TOTAL FROST PENETRATION = 16.6 inches

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 598 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Snow	12.0	0	15	9.50	0.18	0.0	0	0
2: Course-grained	5.0	18	95	28.62	0.75	2394.0	598	598
End of Frost Penetration								

TOTAL FROST PENETRATION = 17.0 inches

5-39

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 598 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Snow	6.0	0	15	9.50	0.18	0.0	0	0
2: Course-grained	6.0	18	95	28.62	0.75	2394.0	337	337
3: Course-grained	6.0	10	110	26.95	1.20	1584.0	262	599

End of Frost Penetration

TOTAL FROST PENETRATION = 18.0 inches

5-40

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 598 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Snow	6.0	0	15	9.50	0.18	0.0	0	0
2: Course-grained	6.0	18	95	28.60	0.75	2394.0	336	336
3: Course-grained	6.0	10	110	26.95	1.20	1584.0	265	601

End of Frost Penetration

TOTAL FROST PENETRATION = 18.0 inches

5-41

24" Cobb.

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 299 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	FREEZING INDEX DISTRIBUTION Accumulated
1: Course-grained	6.0	18	95	28.60	0.75	2394.0	39	39
2: Course-grained	15.2	10	110	26.95	1.20	1584.0	259	298

End of Frost Penetration

TOTAL FROST PENETRATION = 21.2 inches

18"

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 299 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Course-grained	6.0	18	95	28.60	0.75	2394.0	39	39
2: Course-grained	12.0	10	110	26.95	1.20	1584.0	182	221
3: Course-grained	3.4	10	110	26.95	1.20	1584.0	79	300

End of Frost Penetration

TOTAL FROST PENETRATION = 21.4 inches

5-43

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 598 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Snow	2.0	0	15	9.50	0.18	0.0	0	0
2: Course-grained	6.0	18	95	28.60	0.75	2394.0	127	127
3: Course-grained	18.0	10	110	26.95	1.20	1584.0	471	598

End of Frost Penetration

TOTAL FROST PENETRATION = 26.0 inches

5-44

30 Inch Cover

MODIFIED BERGGREN SOLUTION

--- Summary ---

Weather data is from: Boston, Massachusetts

Design Freezing Index (AIR) = 598 F-days
Design Freezing Index (SURFACE) = 299 F-days
Mean Annual Temperature = 50.4 Degrees F
Length of Freezing Season = 67 Days

LAYER #: Type	LAYER THICKNESS (inches)	WATER CONTENT (%)	DRY DENSITY (pcf)	HEAT CAPACITY	THERMAL CONDUCTIVITY	LATENT HEAT of FUSION	FREEZING INDEX DISTRIBUTION Each layer	Accumulated
1: Course-grained	6.0	18	95	28.62	0.75	2394.0	39	39
2: Course-grained	15.6	10	110	26.95	1.20	1584.0	266	305

End of Frost Penetration

TOTAL FROST PENETRATION = 21.6 inches

5-45

CanonieEnvironmental

Canonie

By KAP Date 11/19/91 Subject South St, Walpole, MA Sheet No. 1 of 1

Chkd. By CC Date 11-20-91 CONTAINMENT VOLUME Proj. No. 89-223-05

1/4" X 1/4"

5.2 Available Volume Calculation for Asbestos Contaminated Soil BackFill Above New Pipe Arch Culvert

In order to determine the available containment volume for the asbestos fill material the computer program DCA by Softdesk was used. A trial and error method consisting of drawing the site cap final contours and then computing the amount of fill volume that would be required to develop the contours was used.

The computer print out that follows was generated using the site cap contours that will be constructed in the field. These contours restrict the amount of fill matterial placed above the crown of the culvert to six feet. This value can be increased in the field to eleven feet in order to provide additional containment volume if needed.

îëëë Grid Based Earthworks | DCA DTM 386 Ver 11.0 | Project: CAD89223 ëëëëë

south street

5-47

Surface 1: EGRND		Surface 2: FGRND	
Min. X:	666807.9218	Min. X:	666807.9218
Min. Y:	414723.4631	Min. Y:	414723.4631
Max. X:	667339.7621	Max. X:	667338.0912
Max. Y:	415265.7581	Max. Y:	415265.7581
Random Points:	-	Random Points:	-
Fault Points:	-	Fault Points:	-
Contour Points:	4581	Contour Points:	3325
Total:	4581	Total:	3325
Min. Elev.:	145.0000	Min. Elev.:	145.0000
Max. Elev.:	171.0000	Max. Elev.:	173.0000
Mean Elev.:	156.9569	Mean Elev.:	159.8131

Grid Volume Status

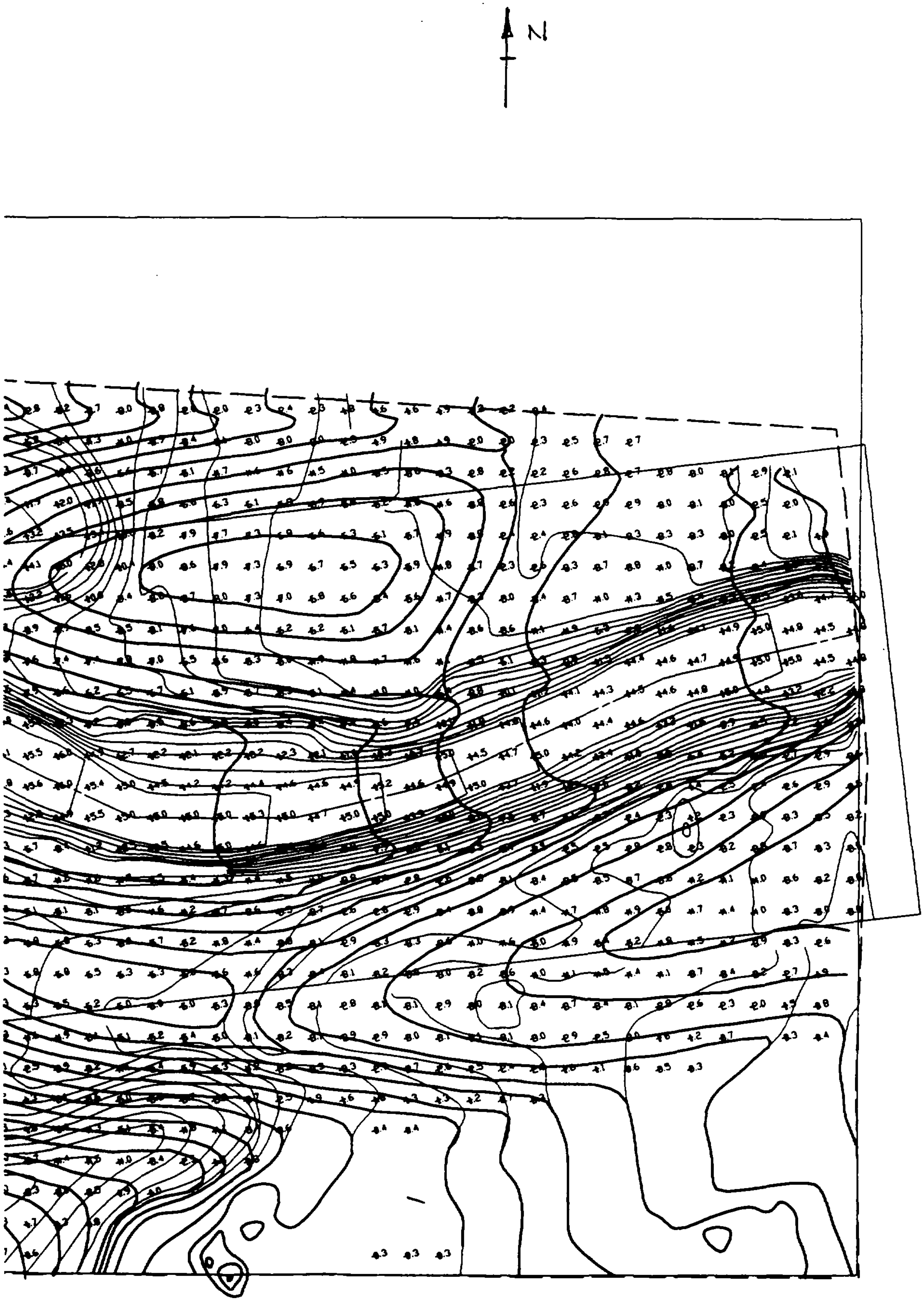
Cut: (cu.yds.):59.2

Fill: (cu.yds.):26007.3

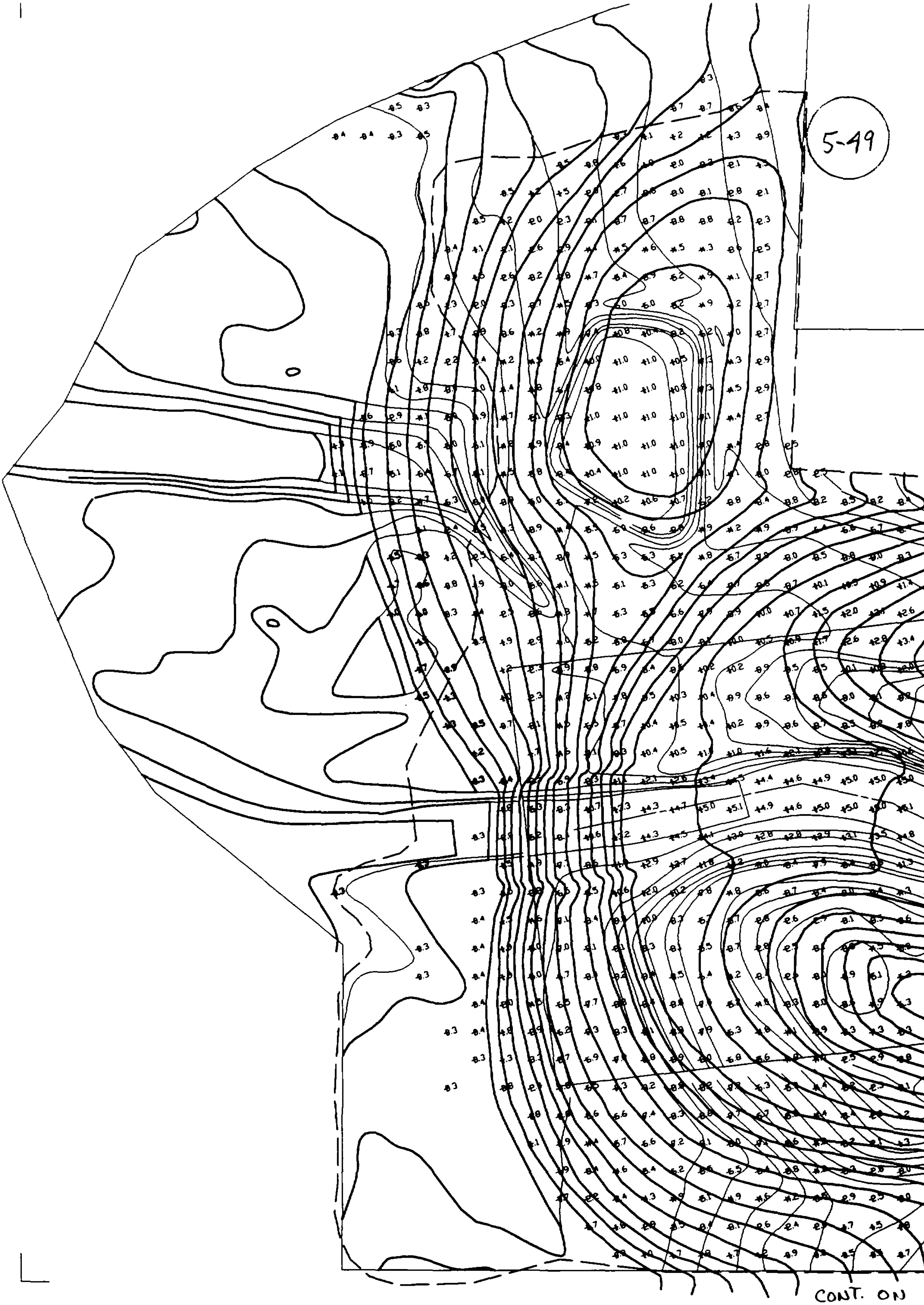
Net FILL: (cu.yds.):25948.0

Complete: 100 %

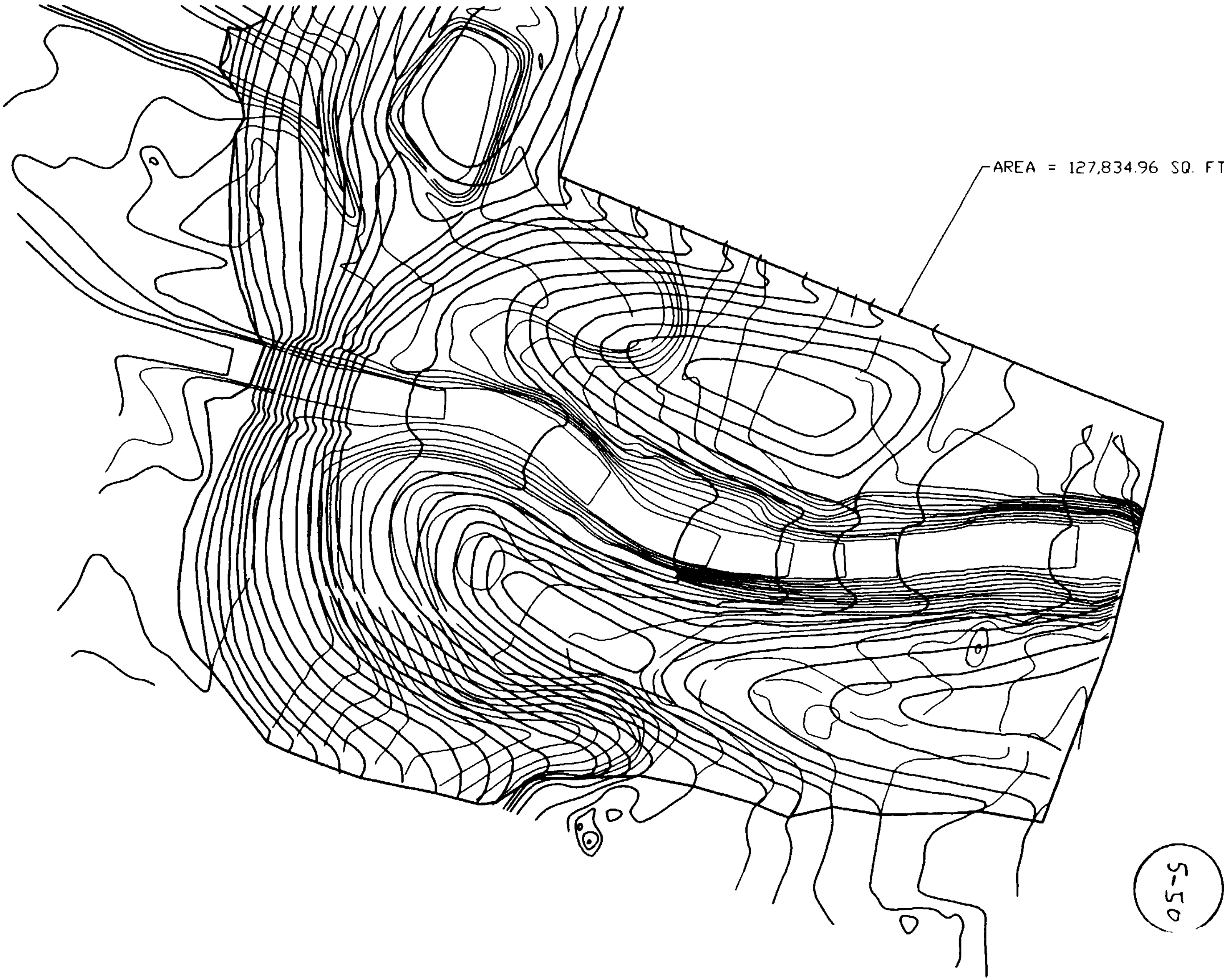
Volume calculations done. Press any key to continue...



CONT. ON PG. 5-49



CONT. ON
PG 5-50


AREA = 127,834.96 SQ. FT.

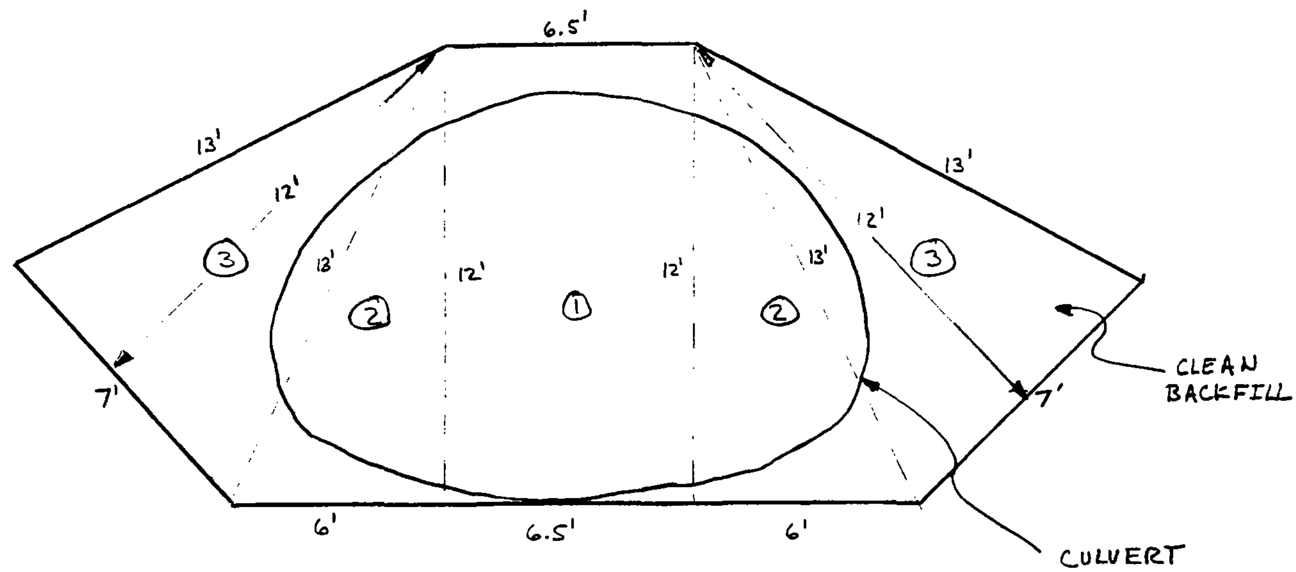
Canonie

By MJG Date 11/19/91 Subject Culvert Area Volume Calculations Sheet No. 1 of 2

Chkd. By CC Date 11-19-91 Proj. No. 89-223-05

1/4" X 1/4"

TYPICAL CROSS - SECTION



SCALE 1" = 5'

AREA CALCULATIONS:

AREA 1	11'-7" x 6.5'	=	75.3 ft² x 1	=	75.3 ft²
AREA 2	½ x 6' x 12'	=	36 ft² x 2	=	72 ft²
AREA 3	½ x 7' x 12'	=	42 ft² x 2	=	84 ft²
					231.3 ft²

VOLUME CALCULATIONS:

$231.3 ft^2 \times 400\ ft = \underline{\underline{92,520\ ft^3}}$

$92,520\ ft^3 \times 1\ yd^3/27\ ft^3 \simeq \underline{\underline{3,427\ yd^3}}$

Canonie

By MJG Date 11/20/91 Subject Volume Calculations Sheet No. 2 of 2

Chkd. By CC Date 11-20-91 Proj. No. 89-223-05

1/4" X 1/4"

NET CONTAINMENT VOLUME = COMPUTED VOLUME − CAP VOLUME − CULVERT VOLUME

TOTAL COMPUTED VOLUME = 25,948 yd^3

CULVERT VOLUME = 3,427 yd^3

CAP VOLUME = 127,835 ft^2 x 2.5 ft = 319,588 ft^3

= 319,588 ft^3 x 1 yd^3/27 ft^3

= 11,836 yd^3

NET CONTAINMENT VOLUME = 25,948 yd^3 − 11,836 yd^3 − 3,427 yd^3

= 10,685 yd^3

Canonie

By NAC Date 7/13/92 Subject SOIL CAP RUNOFF Sheet No. 1 of 1

Chkd. By SP Date 7/13/92 Proj. No. 89-223-18

1/4" X 1/4"

SECTION 5.3 SOIL CAP SURFACE RUNOFF

INSTALLING A CLOSED CULVERT IN PLACE OF OPEN RIVER WILL AFFECT THE AMOUNT OF RUNOFF IN THE AREA AROUND THE CULVERT. RAINFALL WHICH WOULD HAVE FALLEN DIRECTLY ON THE RIVER OR INFILTRATED THROUGH THE BANKS WILL NOW RUNOFF FROM THE SOIL CAP OR INFILTRATE INTO THE CAP. DRAINAGE SWALES WILL BE CONSTRUCTED TO DIRECT THIS RUNOFF INTO THE RIVER AT THE DOWN-STREAM END OF THE CULVERT.

FOLLOWING SECTION PRESENTS THE SURFACE RUNOFF CALCULATIONS AND THE NEED FOR EROSION PROTECTION.

Canonie

By KAP Date 6/1/92 Subject South Street Sheet No. 1 of 12
Chkd. By KJH Date 6/2/92 Final Cap Swale Erosion Proj. No. 89-223-17
1/4" X 1/4"

In order to determine if erosion protection is required the maximum flow velocity and depth for each swale will be computed for a rainfall equal to the 24 hr 100 year storm [Ref 2-9]. The procedure used will be the TR-55: Urban Hydrology for Small Watersheds, Technical Release No. 55 as presented in, "A Guide to Hydrologic Analysis Using SCS Methods," Ref 5-3

The above referenced method will be used to calculate the flow rate that would result from a twenty four hour 100 year rainfall. The velocity and depth will then be calculated using Manning's Equation for open channel flow.

TR-55 Method [Ref 5-3]

1) The curve number, CN, is estimated from Table 2 based on Group B Soil Classification (Sandy loam), open spaces, good condition, and grass cover on 75% or more

$\Rightarrow$ CN = 61 Table 2 p14 Correction for Type III soil condition $\Rightarrow$ CN=79

2) The potential maximum retention, S, is based on CN = 61

$$S = \frac{1000}{CN} - 10 = \frac{1000}{79} - 10 = 2.66 \qquad Eq\ 8\ p\ 11$$

CN and S are constant

The drainage area for each swale is calculated next

Area Draining Into Each Swale

Most Northern Swale:

$5.5\text{ in} \times 2.25\text{ in} = 12.375\text{ in}^2$

$3.5\text{ in} \times 5\text{ in} = 17.5\text{ in}^2$

29.875 in^2 on sheet 22

$$29.875\text{ in}^2 \times \left(\frac{30\text{ ft}}{\text{in}}\right)^2 = 26,887.5\text{ ft}^2$$

Next Northern Swale:

$2.3\text{ in} \times 5\text{ in} = 11.5\text{ in}^2$

$3\text{ in} \times 4.25\text{ in} = 12.75\text{ in}^2$

24.25 in^2 on sheet 22

$$24.25\text{ in}^2 \times \left(\frac{30\text{ ft}}{\text{in}}\right)^2 = 21,825.0\text{ ft}^2$$

South Bank Swale:

$2\text{ in} \times 9\text{ in} = 18\text{ in}^2$

$4\text{ in} \times 3\text{ in} = 12\text{ in}^2$

30.00 in^2 on sheet 22

$$30.00\text{ in}^2 \times \left(\frac{30\text{ ft}}{\text{in}}\right)^2 = 27,000.0\text{ ft}^2$$

Canonie

By KAP Date 6/2/92 Subject South Street Sheet No. 3 of 12
Chkd. By KJH Date 6/2/92 Final Cap Swale Erosion Proj. No. 89-223-17
1/4" X 1/4"

Most Southern Swale

$1.5'' \times 8'' = 12\ in^2$
$4'' \times 3.5'' = 14\ in^2$
$1'' \times 5'' = 5\ in^2$
$2'' \times 3.5'' = \underline{7\ in^2}$
$38\ in^2 \times (30\ ft/in)^2 = 34,200\ ft^2$

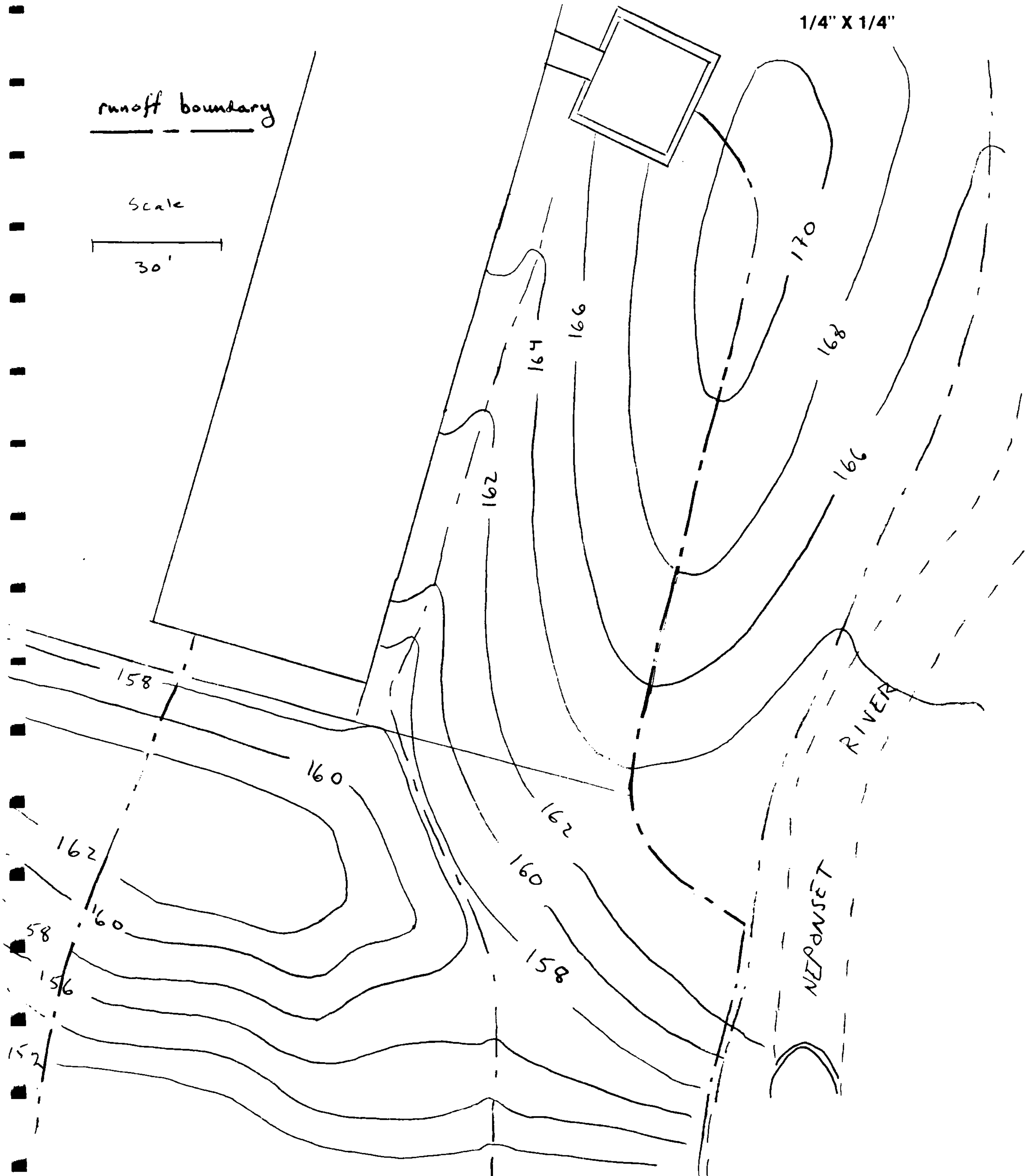
The calculated drainage area for each swale will be used in the TR-55 method.

Canonie

By NAS Date 5/20/92 Subject South Street Sheet No. 4 of 12

Chkd. By KAP Date 5/21/92 Final Cap Swale Erosion Proj. No. 89-223-17

1/4" X 1/4"

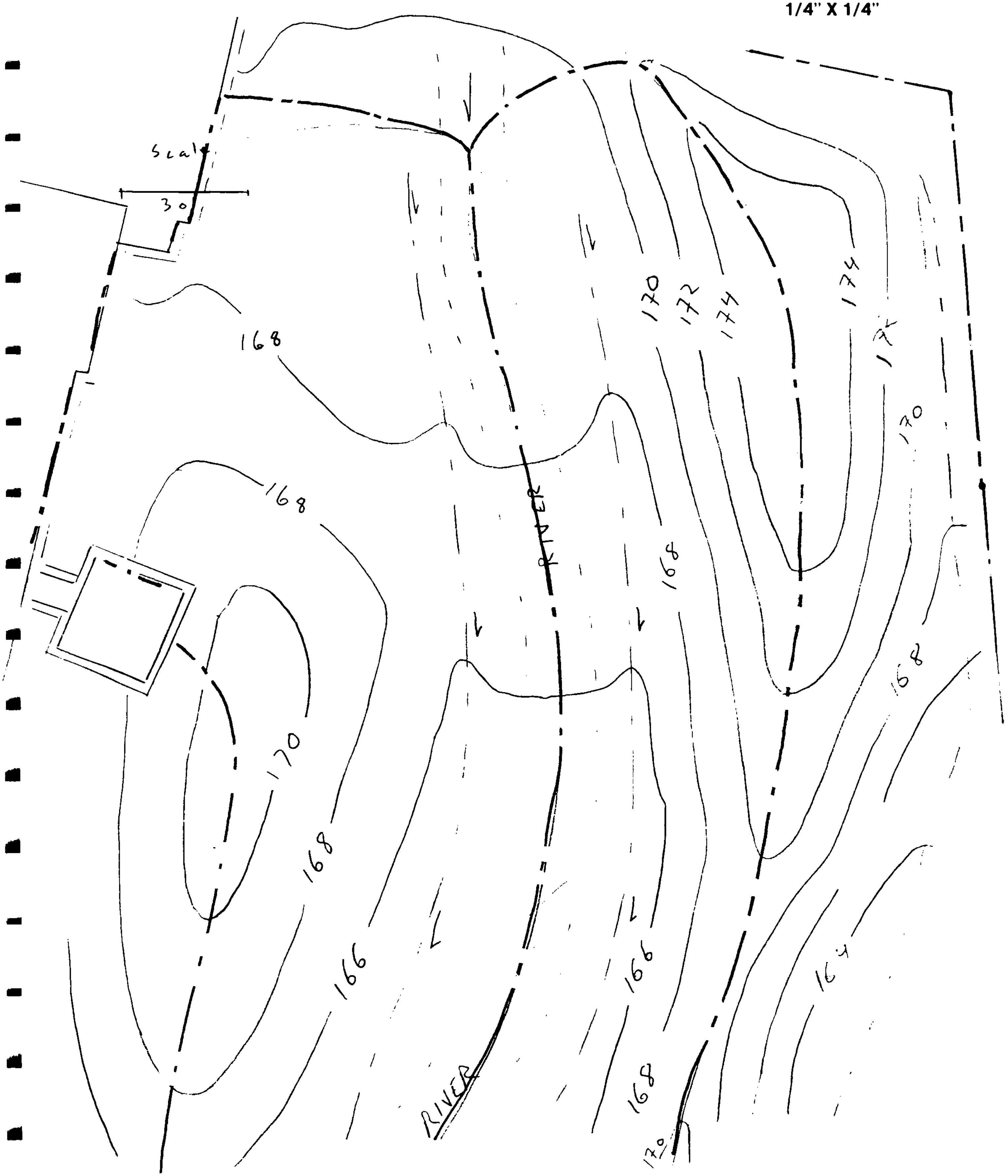


Canonie

By NAS Date 5/20/92 Subject South Street Sheet No. 5 of 12

Chkd. By KAP Date 5/21/92 Final Cap Swale Erosion Proj. No. 89-223-17

1/4" X 1/4"

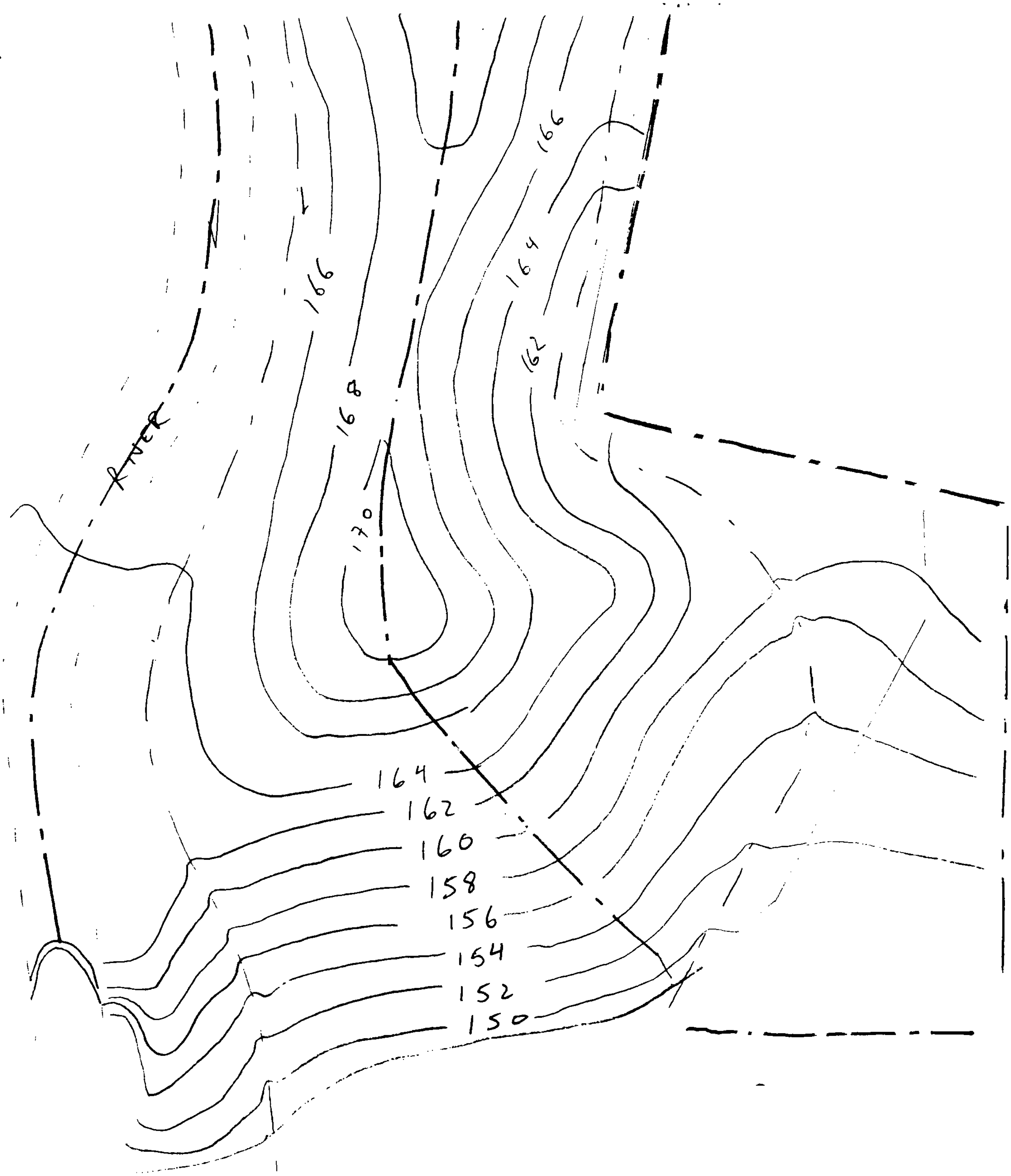


Canonie

By NAS Date 5/20/92 Subject South Street Sheet No. 6 of 12

Chkd. By KAP Date 5/21/92 Final Cap Swale Erosion Proj. No. 89-223-17

1/4" X 1/4"



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Canonie

By KAP Date 6/1/92 Subject South Street Sheet No. 7 of 12
Chkd. By KJH Date 6/2/92 Final Cap Swale Erosion Proj. No. 89-223-17
1/4" X 1/4"

1) Most Northern Swale

1. Time Lag, L.

$$L = \frac{l^{.8}(S+1)^{.7}}{1900\ Y^{.5}} \qquad \text{Eq 9} \quad \text{p 19}$$

l = length of swale = 250 ft
$S = 2.66$
$Y = \text{slope} = \frac{166' - 152'}{250'} = 5.6\%$

$$L = \frac{(250')^{.8}(2.66+1)^{.7}}{1900\ (5.6)^{1/2}} = 0.046 \text{ hours}$$

2. Time-of-concentration, t_c.

$$t_c = \frac{5}{3} L = \frac{5}{3}(.046) = 0.076 \text{ hrs} \qquad \text{Eq 10} \quad \text{p 19}$$

3. Direct Runoff

$$Q = \frac{(P - .2S)^2}{P + .8S} \qquad \text{Eq 7} \quad \text{p 11}$$

P = 24 hr 100 yr precipition = 8.4 in

$$Q = \frac{(8.4 - .2(2.66))^2}{8.4 + .8(2.66)} = 5.88 \text{ in}$$

4. Area = 27,000 ft^2 = 0.001 mi^2

5. unit peak discharge q'_p from Fig 9 p 25

$\Rightarrow q'_p = 1000\ cfs/mi^2/in$

Canonie

By KAP Date 6/1/92 Subject South Street Sheet No. 8 of 12
Chkd. By KJH Date 6/2/92 Final Cap Erosion Proj. No. 89-223-17
1/4" X 1/4"

6. Peak discharge, q_p

$$q_p = q_p' \, A \, Q = (1000 \text{ cfs}/\text{mi}^2/\text{in})(.001 \text{ mi}^2)(5.88 \text{ in})$$ Step 5 p 24

$$q_p = 5.88 \text{ cfs}$$

7. Resulting Velocity, V.

$$q_p = \frac{1.49}{n} A R^{2/3} S^{1/2}$$ Manning's Equation [Ref 1-4]

The swale channels will be constructed with 1:6 side slopes

$\Rightarrow$ $A = 6y^2$ (wetted area corresponding to water depth y)

$P_{wetted} = 2\sqrt{y^2 + (6y)^2} = 2y\sqrt{37}$ (wetted perimeter with wate depth y)

$R = \frac{A}{P_{wetted}} = \frac{6y^2}{2y\sqrt{37}} = \frac{3y}{\sqrt{37}}$ (hydraulic radius)

Substituting into Mannings Equation and solving for Y

$$Y = \left[\frac{0.179 \, q_p \, n}{S^{1/2}}\right]^{3/8}$$ $n = 0.03$ for grass $S_{max} = 0.1$

$$Y = \left[\frac{0.179(5.88)(0.03)}{\sqrt{0.1}}\right]^{3/8} = 0.43 \text{ ft}$$

$$A = 6y^2 = 6(.43 \text{ ft})^2 = 1.11 \text{ ft}^2$$

$$V = \frac{q_p}{A} = \frac{5.88 \text{ cfs}}{1.11 \text{ ft}^2} = 5.3 \text{ ft/s}$$

Canonie

By KAP Date 6/1/92 Subject South Street Sheet No. 9 of 12

Chkd. By KJH Date 6/2/92 Final Cap Swale Erosion Proj. No. 39-223-17

1/4" X 1/4"

B) Northern Bank Swale $Y = \frac{170' - 152'}{360'} = 0.05 = 5\%$

1) Time Lag $L = \frac{(360\,ft)^{.8}(2.66+1)^{.7}}{1900\ (5.0)^{1/2}} = 0.065$ hrs

2. $t_c = 5/3\,L = 5/3(0.065) = 0.11$ hrs

3. $Q = 5.88$ in

4. Area $= 22{,}000\,ft^2 = 0.00079\ mi^2$

5. $q_p' = 950\ cfs/mi^2/in$

6. $q_p = q_p' QA = 950(5.88)(0.00079) = 4.41\ cfs$

7. Velocity at steepest section (near outlet) $S_{max} = 0.4$

$$Y = \left[\frac{0.179(4.41)(0.03)}{\sqrt{0.4}}\right]^{3/8} = 0.29\ ft$$

$$A = 6Y^2 = 0.51\ ft^2$$

$$V = \frac{4.41\ cfs}{0.51\ ft^2} = 8.6\ ft/s$$

Velocity before outlet area $S = 0.036$ (Elevation 170' to 162')

$$Y = \left[\frac{0.179(4.41)(0.03)}{\sqrt{0.036}}\right]^{3/8} = 0.46\ ft$$

$$A = 6Y^2 = 1.26\ ft^2$$

$$V = \frac{4.41\ cfs}{1.26\ ft^2} = 3.5\ ft/s$$

Canonie

By KAP Date 6/1/92 Subject South Street Sheet No. 10 of 12

Chkd. By KJH Date 6/2/92 Final Cap Swale Erosion Proj. No. 89-223-17

1/4" X 1/4"

c) Southern Bank Swale $Y = \frac{170' - 152'}{360'} = 0.05 = 5\%$

1. $L = \frac{(360 ft)^{.8}(2.66+1)^{.7}}{1900\ (5.0)^{1/2}} = 0.065$ hrs

2. $t_c = \frac{5}{3}(.065) = 0.11$ hrs

3. $Q = 5.88$ in

4. $Area = 27{,}000\ ft^2 = 0.001\ mi^2$

5. $q_p' = 950\ cfs/mi^2/in$

6. $q_p = q_p' A Q = 950(0.001)(5.88) = 5.59$ cfs

7. Velocity at steepest section (near outlet) $S_{max} = 0.2$

$$Y = \left[\frac{0.179\ (5.59)(0.03)}{\sqrt{0.2}}\right]^{3/8} = 0.36\ ft$$

$$A = 6Y^2 = 0.79\ ft^2$$

$$V = \frac{q_p}{A} = \frac{5.59\ cfs}{0.79\ ft^2} = 7.1\ ft/s$$

Velocity before outlet area $S = 0.031$ (Elevation 170' to 162')

$Y = 0.51$ ft

$A = 6Y^2 = 1.6\ ft^2$

$V = \frac{q_p}{A} = 3.51$ ft/s

Canonie

By KAP Date 6/2/92 Subject South Street Sheet No. 11 of 12

Chkd. By KJH Date 6/2/92 Final Cap Swale Erosion Proj. No. 89-223-17

1/4" X 1/4"

D) Most Southern Swale $\quad Y = \frac{170' - 150'}{380} = 0.053 \quad 5.3\%$

1. $L = \frac{(380 ft)^{.8}(2.66+1)^{.7}}{1900\ (5.3)^{1/2}} = 0.066$ hrs

2 $t_c = \frac{5}{3}(0.066 hrs) = 0.11$ hrs

3. $Q = 5.88$ in

4. Area $= 34{,}200\ ft^2 = 0.00122\ mi^2$

5. $q'_p = 950$ cfs/mi²/in

6. $q_p = q'_p\, Q\, A = (950\ cfs/mi^2/in)(5.88\ in)(0.00122\ mi^2) = 6.81$ cfs

7. Resulting Velocity for max slope $S = 0.22$ (Elevation 158' to 156')

$$Y = \left[\frac{0.179(6.81 cfs)(0.03)}{\sqrt{.22}}\right]^{3/8} = 0.38\ ft$$

$A = 6y^2 = 0.87$

$$V = \frac{q_p}{A} = \frac{6.81\ cfs}{0.87\ ft^2} = 7.8\ ft/s$$

Resulting Velocity for Slope $S = 0.13$ (max slope above elevation 162')

$$Y = \left[\frac{0.179(6.81 cfs)(0.03)}{\sqrt{0.13}}\right]^{3/8} = 0.42\ ft$$

$A = 1.08\ ft^2$

$$V = \frac{6.81\ cfs}{1.08\ ft^2} = 6.31\ ft/s$$

Change contours to achieve a max slope of $S = 0.05$ at locations above elevation 162'

Table 2-5 "Suggested Maximum Permissible Mean Channel Velocities" in EM 1110-2-1601 gives the following [Ref 2-21]

Material	Mean Channel Velocity (ft/s)
Grass-Lined Earth (Slopes less than 5%)	5 - 8 depending on type of grass and soil

Conclusion

The slopes of the swales above elevation 162 ft will be adjusted to yield a maximum slope of 5%

The computed velocities in the swales above elevation 162 ft are in or below the allowable range of 5 to 8 ft/s. Therefore, no erosion protection is required.

The slopes of the swales below elevation 162 ft are greater than 5% and do not qualify for the Table 2-5 listed above. Therefore, Rip-Rap will be placed in the swales from elevation 162 ft to 150 ft were the slope is greater than 5%.

The rip-rap used will be of the same type that will be placed at the culvert entrance and outlet.

SECTION 5 REFERENCES

5-66

REFERENCE 5-1

Contract Report to EPA, Region I by U.S. Army Cold Regions Research and Engineering Laboratory (CRREL), Lexington, Massachusetts, Engineering Design, Classification and On-Site Technical Assistance for Waste Asbestos Site, Lowell Road, Hudson, New Hampshire

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of Engineers

New England Division

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Chief, GES & CRREL
Mr. Bill Quinn
603-646-4471

5-68

APPENDIX

Rationale for Determining Soil Depth Cover for Waste Asbestos in Nashua and Hudson, New Hampshire

Introduction

This rationale is intended to provide preliminary guidance and relates specifically to the Shady Lane and Lowell Road asbestos burial sites in Nashua and Hudson, New Hampshire, respectively. It is based primarily on laboratory and field experience with freezing and thawing of soils containing inclusions.

The primary objective of this design is to determine the amount of soil cover needed to significantly reduce the number of times that the frost zone penetrates the soil ladened asbestos material. The main concern is that if the soil with asbestos or simply asbestos is allowed to freeze and thaw, a process called sorting may bring the asbestos pieces to the ground surface and expose the environment to their hazardous effects.

When a moist soil freezes, water within the soil is drawn toward the freezing front, i.e. where freezing is taking place. A mass of ice is formed within the voids of the soil which tends to push the particles apart. Normally, the direction in which the particles of soil are most free to move is toward the surface, resulting in frost heave.

If a source of free water is available, such as a water table within 10 ft. of the surface, the ice masses will continue to draw water from below, eventually forming what are typically termed ice lenses. Significant volume changes occur in a soil when ice lenses form. Not only will the ground surface rise (i.e., heave) anywhere from 1 inch to 12 inches

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during a winter season, but individual particles will be moved closer to the surface (Washburn, 1973; Corte, 1969; Kaplar, 1965).

During the subsequent spring thawing period much of the frost heave will be recovered: the ground normally subsides more or less back to its former volume. In order for this to happen, excess water stored in the ice lenses must somehow drain away; but until it does, the soil will be soft and easily deformable.

Because the soil is temporarily soft, the finer (smaller) particles of soil tend to move down and under the larger particles, holding them in the positions to which they moved when they were frozen. When the soil finally becomes more stable in the summer, the larger particles end up closer to the surface than they were the previous summer. Because this sorting process takes place year after year, stones and other particles may sometimes move significant distances toward the ground surface. The process is likely to be accelerated when the particles approach the ground surface, because this zone experiences multiple freeze/thaw cycles during a single winter season.

It is assumed that asbestos chunks and particles are likely to be affected by the seasonal processes of freezing and thawing in much the same manner as stones and other natural inclusions. Freeze/thaw cycles may have two types of long-term effects on the permanent containment of waste asbestos as a result of:

a. The differential movement of foreign matter toward the surface with each cycle of freezing and thawing may eventually bring chunks and pieces of asbestos to the ground surface and thereby return them to the external environment; and

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b. Asbestos scraps (broken boards, chunks, even cemented pellets) may be broken into smaller pieces as moisture held within the scraps freezes, so that when they reach the surface they are sufficiently small to be environmentally hazardous.

A prospective long-term means of preventing asbestos particles from moving toward the surface through the freezing and thawing process is to make sure that they are never frozen. However, because economic considerations must be taken into account, the design presented here is based on an attempt to contain the waste asbestos below the ground surface for a minimum period of 100 years. The actual safe lifetime is probably well in excess of 100 years, provided the removal of covering material by erosion or human activities is effectively controlled. This design assumes a turf cover with a well established root zone. It does not include a safety factor to account for construction or garden cultivation.

The procedure used here to select an appropriate depth of soil cover to contain asbestos material below the surface in southern New Hampshire is a modification of a procedure developed by the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) for designing pavements against failure caused by freezing and thawing (Dept. of the Army and Air Force, 1985). It employs the use of the modified Berggren equation for layered systems, which is also discussed in Lunardini (1981).

This procedure has been validated with more than 30 years of experience by the Corps of Engineers in airfield and highway design. The technique has been shown to yield excellent results, when applied to in situ phase changes in soils (Dept. of the Army and Air Force, 1966; McRoberts, 1975).

5-7

Basically the ground freezes because of the removal of heat. It is therefore clear that the depth of frost penetration below the surface is related to the meteorological variables that affect heat loss and the physical properties of the soil. The major variables are (1) the severity of the winter air temperatures, and (2) the amount of snow cover. The severity (coldness) of the air temperature is expressed in terms of an air freezing index for each winter. The efficiency of thermal transfer between the air and the ground or snow is expressed in terms of an n-factor, where n is a fraction between 0 and 1. In combination, the two parameters result in a surface freezing index, which is a measure of the amount of heat leaving the ground over an entire winter. The manner in which these values are utilized to calculate the suggested depth of cover for safe burial of waste asbestos is described below.

Air Temperature

The air freezing index for a given area is the total number of Fahrenheit degree-days below freezing (32°F) for a winter season. It is calculated as discussed in the Department of the Army Technical Manual 5-852-6 (1966). The calculation procedure is similar to that used for the more familiar "heating degree-days," except that for heating purposes the winter temperatures are subtracted from 65°F.

The average daily, or mean monthly, air temperatures used for these calculations are those measured by the nearest official station of the National Weather Service. The Weather Service publishes a monthly record of these temperatures (U.S. Dept. of Commerce, 1950-1980). At 10 year intervals, they also publish 30-year normals of various climatological parameters (U.S. Dept. of Commerce, 1985).

Nashua, New Hampshire, is one of the weather stations for which an extended record of air temperatures is available. An accurate record of many years duration is desirable because it shows not only the average temperatures likely to occur in a winter, but the warmer and colder years as well and the frequency with which they occur. The temperatures in a region are normally considered to be those of the central station with adjustments for exposure; the temperatures at Nashua are assumed to be representative of those at Hudson, New Hampshire.

For the Nashua/Hudson area the air freezing index, averaged over the 30-year period from 1951 to 1980, is calculated from the temperature normals to be 665 degF-days. This value is termed the mean air freezing index for the area. Distribution of mean air freezing index values for the continental U.S. is given in Figure A1. In the absence of additional information, it is assumed for this study that the mean air freezing index in the future will be similar.

The surface freezing index represents a quantity of heat removed from the ground over a winter season; the loss of this heat causes some of the moisture in the ground to freeze. For moist, sandy soils such as those normally selected to cover the waste materials (see properties, Table A3), calculation based on the procedures described in Aitken and Berg (1968) indicates that the average frost depth for a freezing index of 665 will be about 24 inches when the ground is kept free of snow. A cover of snow reduces the average frost depth, because air held within the snow particles insulates the ground surface. An analysis of the Nashua weather records (described later) indicates that an average of 6 inches of snow remains on the ground in a typical winter. The actual average frost depth in the

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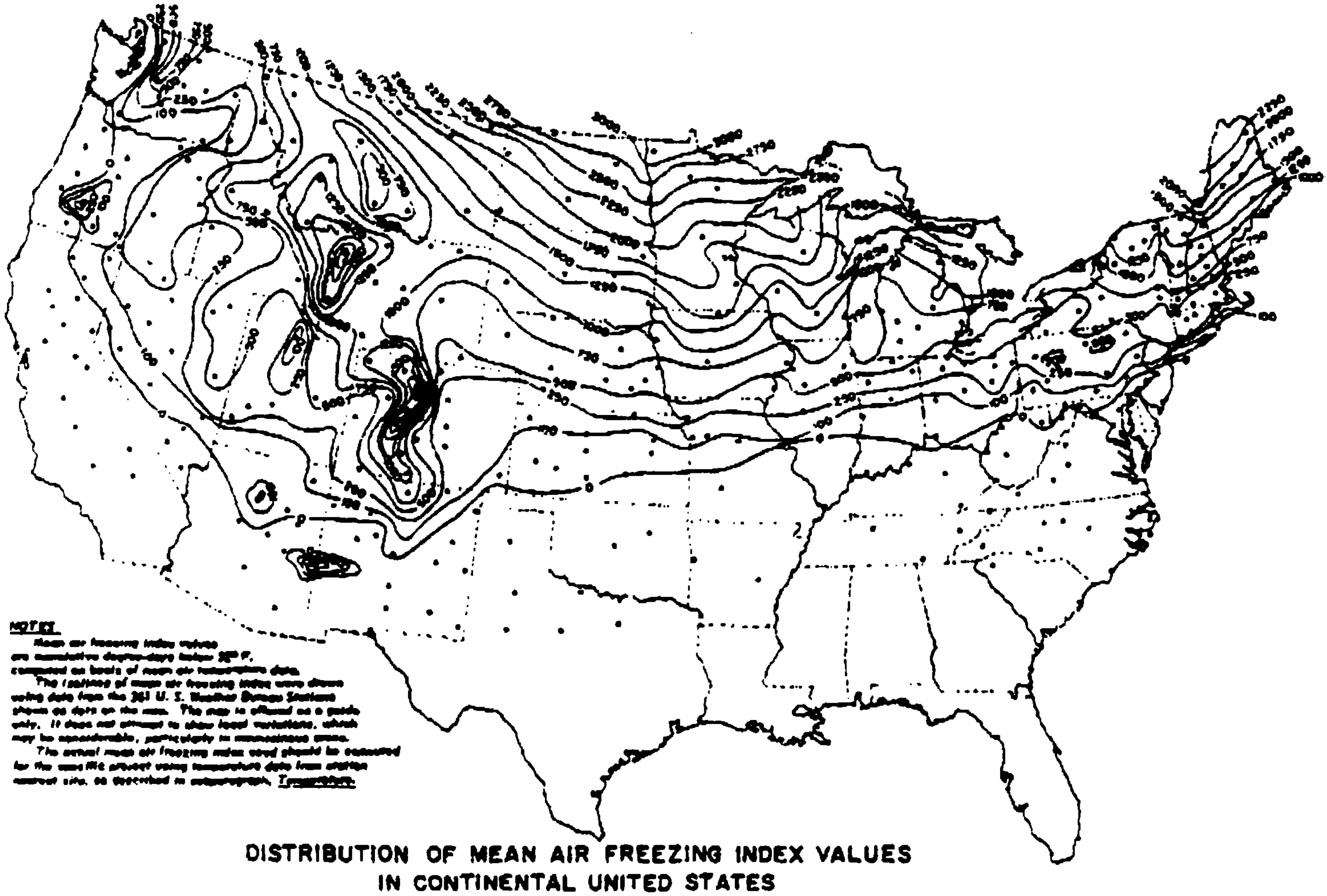

Figure A1. Distribution of mean air freezing index values in the continental U.S. (Dept. of Army and Air Force, 1985).

Nashua/Hudson area during a winter with a freezing index of 665 and a snow depth of 6 inches would be estimated to be about 12 inches.

It might seem that a soil cover of 12 in. would consequently protect the waste asbestos from freezing. However, 12 in. is not sufficient for the simple reason that average values are normally exceeded for approximately one-half the total time. Thus in about 50 years out of 100 the frost depth will be greater than 12 inches, with the frost zone extending into the asbestos deposit. This is not a safe condition for long-term protection, because the asbestos is likely to be moved by freeze-thaw effects in each of those 50 years.

A reasonable criterion for safe burial is taken to be an air freezing index exceedance level of no more than 10%, representing an average of 10 freezings into the asbestos per century. This criterion is based on the standard used for the design of pavements in seasonal frost areas. Air freezing indices selected for pavement design are usually based upon the coldest year in 10 years or the average for the three coldest years in 30.

In order to determine an air freezing index to be used for asbestos cover design in the Nashua/Hudson area, Nashua air temperature records from the 30-year period 1951 to 1980 were analyzed (U.S. Dept. of Commerce, 1950-1980). To get an idea of the variation of the air freezing index during this period, yearly indices calculated from the mean monthly temperatures were plotted versus a normal probability scale (Fig. A2). Using this plotting method, the 10% exceedance level is about 990 deg F-days.

To determine more accurate values for design purposes, air freezing indices for the coldest winters were calculated from the mean daily temperatures. Table A1 shows the mean freezing indices of the three cold-

5-75

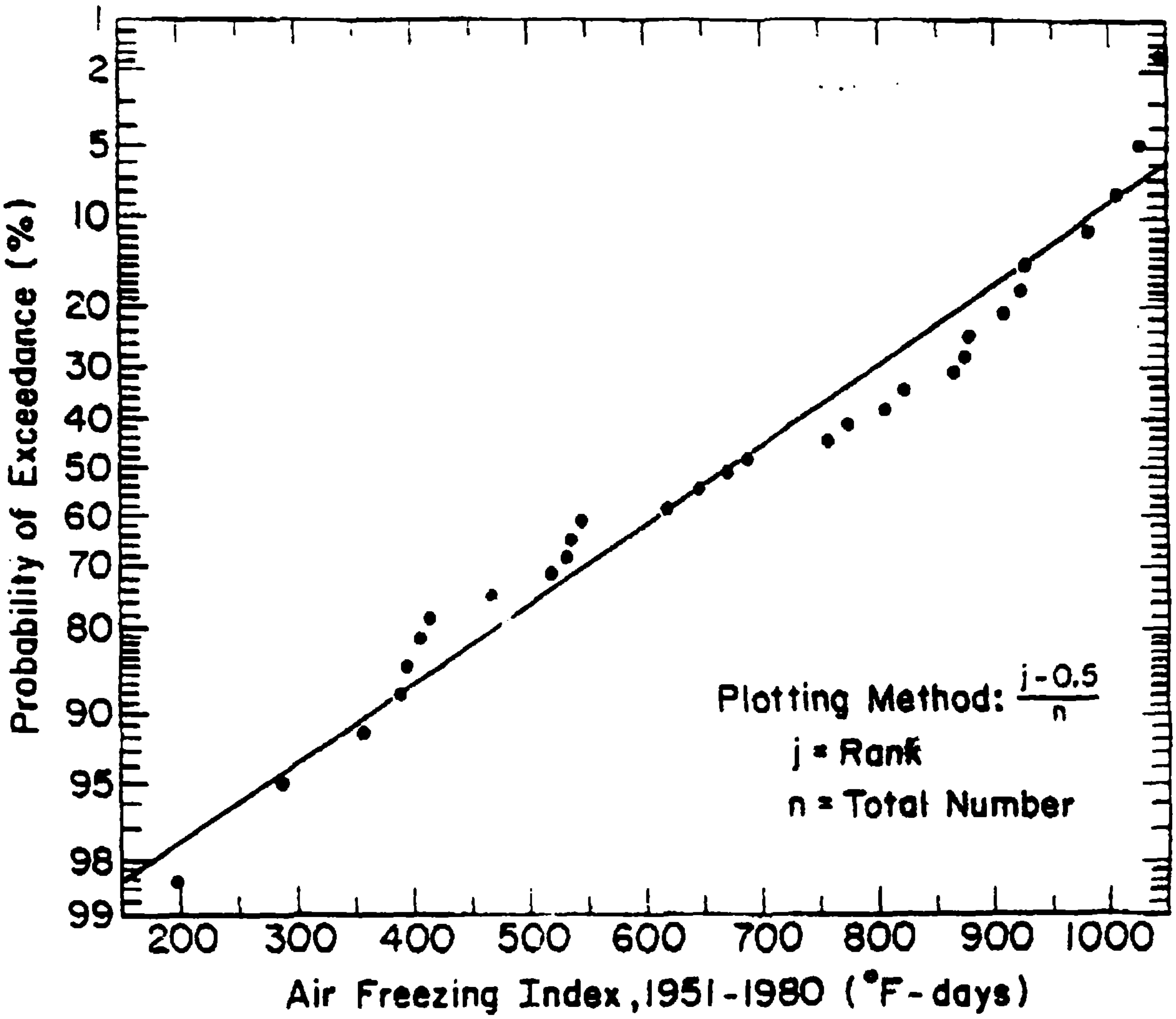

Figure A2. Air freezing indices from Nashua plotted on normal probability paper. Individual points are yearly data calculated from mean monthly temperatures. Solid line indicates cumulative probability of occurrence based on normal distribution.

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Table A1. Mean air freezing index for Nashua, New Hampshire (1951-1980).

	Mean Air Freezing Index (deg F-days)	
Condition	Cal. by Daily Mean	Cal. by Monthly Mean
3 coldest in 30	1097	1023
1 coldest in 10	1075	1041

Table A2. Ratios (n-factors) to convert from air freezing index to surface freezing index.

Surface Type	n-Factor
Snow Surface	1.0
Portland Cement Concrete	0.75
Bituminous Pavement	0.7
Bare Soil	0.7
Turf	0.5

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est winters in 30 and the coldest winter in 10 as calculated by daily temperatures. These are compared with the equivalent freezing indices as calculated by mean monthly temperatures.

To summarize, design air freezing indices for the Nashua area fall in the general range from 990 to 1100 deg F-days, depending on the procedure used for calculation. Indices determined from daily temperatures, which are higher, are likely to be more representative of actual values. For this reason, the design air freezing index selected for use in determining asbestos cover depths was 1097 deg F-days, the average of the three coldest years in thirty.

To determine depth of frost penetration, one other aspect of winter air temperature data must be examined; that is the length of the freezing season. During calculations of the air freezing index from the mean daily temperatures, this was determined to be about 100 days for the Nashua area.

<u>Surface freezing index</u>

To determine the <u>frost penetration</u> resulting from an air freezing index, one must first estimate the corresponding <u>surface freezing index.</u> The difference between air and surface temperatures at any specific time is influenced by latitude, cloud cover, time of year, time of day, atmospheric conditions, wind speed, surface characteristics, and subsurface thermal properties. However, over an entire freezing season, a single ratio, or "n-factor," can generally be used for various surface types to convert air index to surface index. Table A2 lists the n-factors recommended by Linell and Lobacz (1980).

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For simplicity in developing the rationale for this study, an n-factor of 0.5, simulating turf conditions, was assumed for the snow-free case and an n-factor of 1.0 was used when snow was modeled as the cover condition.

Frost Penetration Calculation Procedures

The procedure used to calculate the predicted depth of frost penetration was a multilayered solution of the modified Berggren equation. Detailed discussion of this procedure is found in previous references and only a cursory explanation will be included here (Departments of the Army and Air Force, 1966; Aitken and Berg, 1968; Aldrich and Paynter, 1953). The modified Berggren equation is expressed as:

$$X = \lambda \sqrt{\frac{48\ K\ nF}{L}} \qquad (1)$$

where

X = depth of freeze, ft.

K = thermal conductivity of the soil, B.t.u./ft. hr. °F.

L = volumetric latent heat of fusion, B.t.u./cu.ft.

n = conversion factor for air index to surface index, dimensionless

F = air-freezing index, degree-days

λ = the Lambda coefficient which takes into consideration the effect of temperature changes in the soil mass.

The Lambda coefficient (λ) is a function of the freezing index, the mean annual temperature of the site, the length of the freeze season, and the thermal properties of the soil. In order to determine a value for the Lambda coefficient, one must solve a transcendental equation by iteration using the error function (Aitken and Berg, 1968; Aldrich and Paynter, 1953).

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In modeling frost penetration with the Berggren equation, several assumptions are made. The model assumes one-dimensional heat flow with the entire soil mass at its mean annual temperature prior to the start of the freeze season. In Nashua, the mean annual temperature is 46.4°F (U.S. Department of Commerce, 1983). It is also assumed that when the freezing season starts, the surface temperature changes suddenly (as a step function) from the mean annual temperature to a temperature v_s degrees below freezing and remains at this new temperature throughout the freeze season. The value v_s is the average surface temperatuare differential and is calculated by dividing the surface freezing index by the length of the freeze season, t, in days (v_s = nF/t). These calculations also consider latent heat to be a heat sink at the moving frost line, and assume that soil freezes at a temperature of 32.0°F.

When expressed as shown in equation 1, the modified Berggren equation can be used to model the frost penetration depth in one layer of homogeneous soil. When several layers with different soil types are to be modeled, a multilayer solution to the equation is used. The process involves determining that portion of the surface freezing index required to penetrate each layer. The sum of the thicknesses of all the frozen layers is the total depth of freeze. The partial freezing index required to penetrate the top layer is given by:

$$F_1 = \frac{L_1 d_1}{24\lambda_1^2}\left(\frac{R_1}{2}\right)$$

(5-80)

where

d_i = thickness of soil layer, ft.

$R_i = \frac{d_i}{K_i}$ = thermal resistance of layer .

The partial freezing index required to penetrate the second layer is

$$F_2 = \frac{L_2 d_2}{24\lambda_2^2}\left(R_1 + \frac{R_2}{2}\right) .$$

The partial index required to penetrate the n^{th} layer is

$$F_n = \frac{L_n d_n}{24\lambda_n^2}\left(\Sigma R + \frac{R_n}{2}\right) ,$$

where ΣR is the total thermal resistance above the n^{th} layer and equals

$$R_1 + R_2 + R_3 \ldots + R_{n-1} .$$

The summation of the partial indexes, $F_1 + F_2 + F_3 \ldots + F_n$, is equal to the surface freezing index.

Cover specifications

At the Nashua/Hudson asbestos sites, the majority of the restored area was slated to be revegetated. Therefore, the initial consideration in designing the cover materials was the necessity to support vegetation, specifically grasses.

Because the asbestos waste is extremely alkaline, having a pH of 11 or 12, it is highly toxic to plant life. Therefore, the minimum depth of cover required is that which will be deep enough to keep the grass roots from penetrating the asbestos. Studies have shown that grass roots extend down to 18 inches beneath the surface (Iskandar et al., 1979), so a cover

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at least that deep is required to assure the long-term survival of the vegetation.

Nutrients are also needed for vegetation survival. It is therefore recommended that the upper 6 inches of cover include topsoil to provide nutrients for the grasses. A 2:1 ratio of topsoil to sand would provide nutrients and allow drainage. Beneath the topsoil, the cover material should consist of a 6-in. layer of fine sand, underlain by an adequate amount of sandy gravel to reduce problems with frost penetration.

In order to determine the amount of gravel to recommend, frost penetration was modeled using a multilayer solution of the modified Berggren equation with a soil profile consisting of 6" topsoil, 6" fine sand, and an indefinite amount of sandy gravel. Table A3 shows the physical and thermal characteristics of the modeled materials for both a wet and a dry case. Properties were chosen so as to be generally representative of the materials at the site. The left-hand portion of Table A4 shows the predicted frost penetration depths at the design freezing index using the soil profile with a variable thickness of gravel.

In the most extreme case, that with no snow, frost penetration is on the order of 34-35 inches. When a snow layer at the surface is included, frost penetration decreases as the snow thickness increases. The next aspect to be considered, then, is what depth of snow cover should be used for the design. The duration of snow cover is another important aspect. However, for the purposes of this design, it was assumed that the duration of snow cover was similar to the length of the freeze season, or about 100 days.

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Table A3. Characteristics of Materials Modeled for Frost Penetration.

Layer Type	Moisture Content (%)	Dry Density (pcf)	Heat cap. (Btu/cuft°F)	Thermal Cond. (Btu/fthr°F)	Latent Heat of Fusion (Btu/cuft)
a) Wet Soil					
Topsoil	25	90	32.18	0.85	3240
Sand	15	100	28.25	1.15	2160
Gravel	10	120	29.40	1.59	1728
Asbestos	15	95	26.84	0.68	2052
b) Dry Soil					
Topsoil	18	95	28.62	0.75	2394
Sand	10	110	26.95	1.20	1584
Gravel	10	120	29.40	1.59	1728
Asbestos	15	95	26.84	0.68	2052
(c) Snow	0	15	9.50	0.18	0

Table A4. Frost penetration depths predicted by a multilayer solution of the modified Berggren equation using climate parameters typical for Nashua (freezing index = 1097°F-days; mean annual temp = 46.4°F; length of freeze season = 100 days).

Snow Depth (in.)	Frost penetration (in.) Variable cover thickness[1] Soil condition Dry	Wet	30-Inch cover thickness[2] Soil condition Dry	Wet
0"[3]	34.4	35.4	33.4	34.0
4"[4]	31.7	30.7	31.3	30.4
6"	24.8	23.0	24.8	23.0
8"	19.8	17.7	19.8	17.7
10"	16.3	13.3	16.3	13.3

[1]soil profile: 6" topsoil
6" fine sand
indef. gravel

[2]soil profile: 6" topsoil
6" fine sand
18" gravel
30" total

[3]n-factor = 0.5 (turf)

[4]n-factor = 1.0 (snow)

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Depth of snow measurements were recorded at the Nashua weather station between 1952 and 1976, with some periods of missing data (U.S. Dept. of Commerce, 1950-1980). Complete daily records are available for 15 winter seasons. To get a mean snow depth value for each winter season, the daily depth data between 1 December and 16 March were summed and divided by the number of days. The distribution of these mean seasonal depths are shown in Figure A3. The individual points indicate the yearly data and the solid line shows the cumulative probabilities based on a normal distribution.

The mean snow depth for the 15 years of data is 6.2 inches and the probability that Nashua will have less than this mean amount is 50 percent. As shown in Figure A3, the probability that the mean snow depth will be less than 4 inches is about 25 percent. These two snow depths, then, bracket the amount of snow that might be considered for design purposes.

When the 15-year mean snow depth, 6 inches, is modeled in the Berggren solution, a frost penetration of about 24 inches is predicted (Table A4). Using a 4-inch snow depth, a frost penetration of 30 inches is predicted. For the Nashua/Hudson sites, it was decided to use the 4-inch mean snow depth and the resulting 30 inch minimum cover depth as a final recommendation. This conservative decision resulted from the lack of knowledge about actual rates that asbestos might move through a soil column subjected to cyclic freeze-thaw.

Frost penetration depths were modeled using the Berggren equation with a 30-in. cover (right-hand side, Table A4). Again, material properties used were those shown in Table A3. Because of the lack of actual data, the asbestos properties had to be estimated. The values used were based on the assumption that the top layer of asbestos would have partially mixed with the existing thin topsoil cover. The frost depths predicted by the model

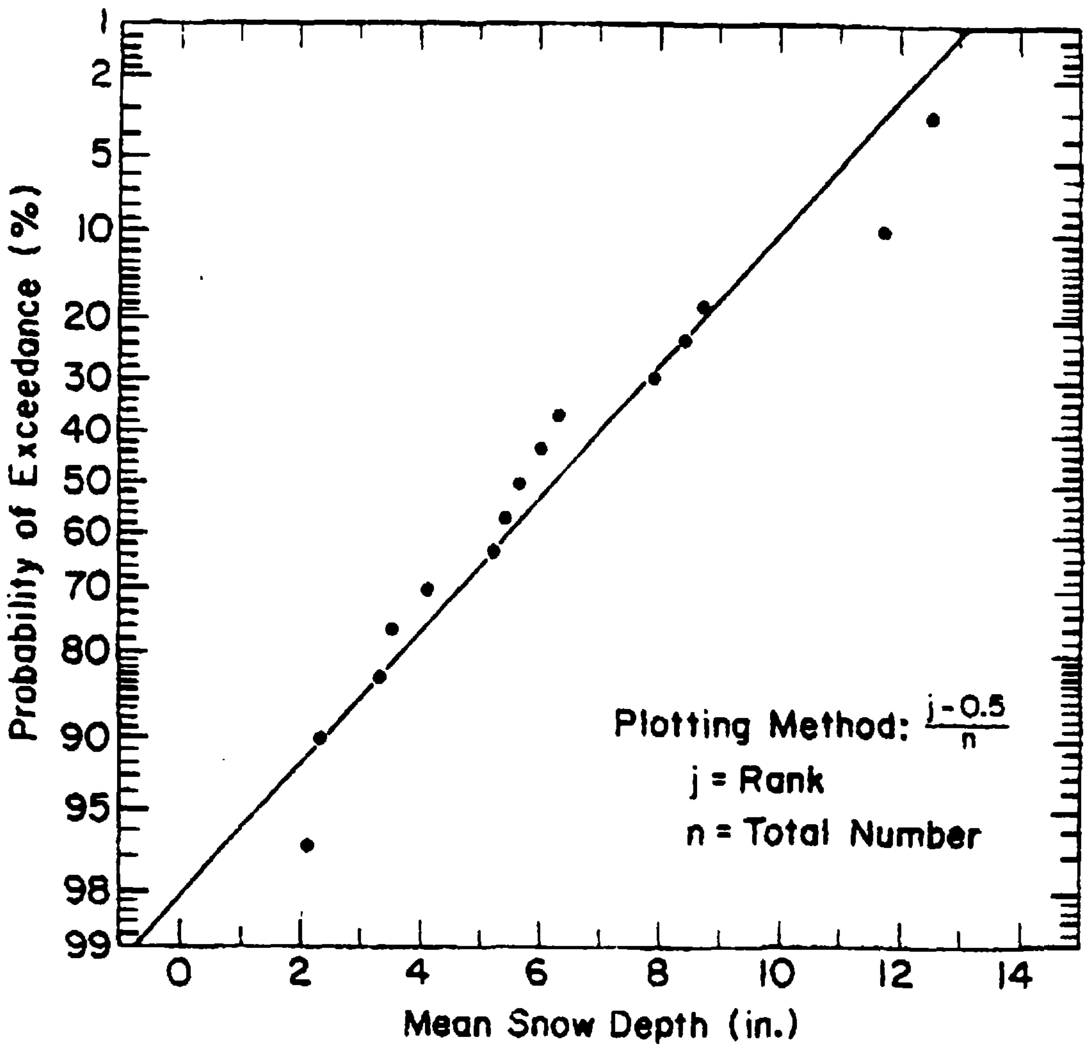

Figure A3. Mean snow depth data from Nashua plotted on normal probability paper. Individual points are the mean snow depth for the winter season 1 December to 16 March. Solid line indicates the cumulative probability of occurrence based on normal distribution.

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would penetrate 4 inches of the asbestos in a no-snow year with the design freezing index. In years with a mean snow depth of 4 inches or greater, the frost would be contained in the cover material.

In summary, the thickness of cover recommended at the present time for the restoration of gently sloping terrain at waste asbestos sites in Nashua and Hudson, New Hampshire is 30 inches. This cover should consist of the following three layers from top to bottom: 6 inches of topsoil and sand mixed in a 2:1 ratio, 6 inches of fine sand, and 18 inches of sandy gravel.

Verification

The 30-in. cover determined through modeling with the Berggren solution correlates well with an independent investigation of observed penetration depths compared with freezing degree day accumulations (Haugen and King, in prep.). Figure A4 shows a total of 282 data points observed over a 10-year period at 37 sites within a five-state area (N. Dakota, S. Dakota, Minnesota, Wisconsin and Upper Michigan). The surface conditions at the sites were primarily low vegetative cover or crop rubble. The soil types were quite variable, although most sites were in agricultural fields or in cemeteries, and thus were primarily fine-grained in nature. The snow depth category for these data (based on 3 observations per year) ranged from 0 to 4 inches. An empirical equation for these data was determined to be

$$Y = -6.46228 + 1.02471 \sqrt{X}$$

where Y is frost depth in inches, and X is accumulated freezing degree days (°F) for the season. Its standard error of estimate is 7.5 inches.

When the Nashua design freezing index of 1097 is used in the empirical equation, it predicts a maximum frost penetration of 27.5" ± 7.5". Con-

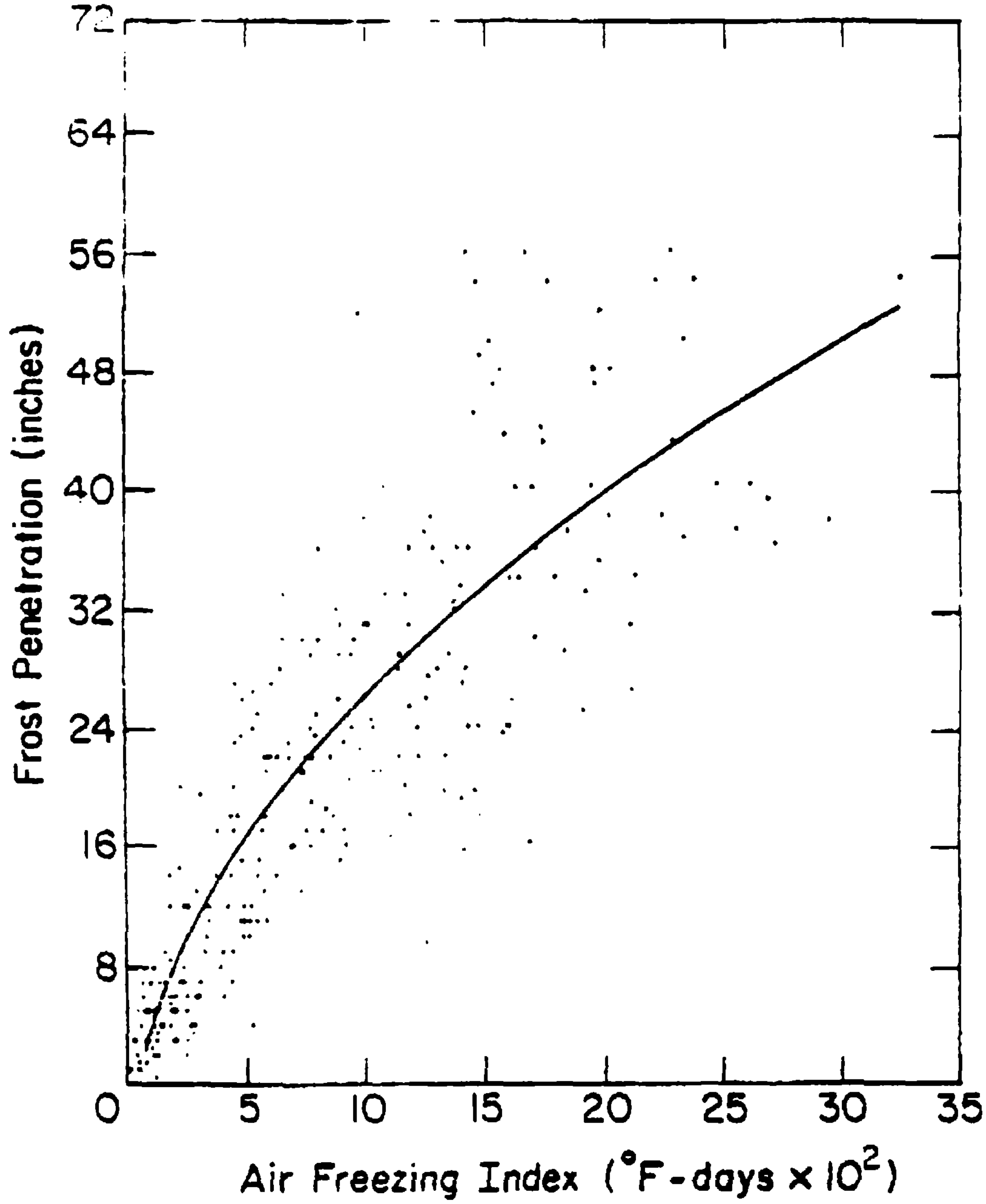

Figure A4. Observed frost penetration depth versus air freezing index (from Haugen and King, in prep.).

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sidering that the equation was derived from sites with mean snow depths ranging from 0 to 4 inches, the Berggren solution prediction of 30 inches penetration based on 4 inches of snow is somewhat high. This discrepancy may perhaps be accounted for by the relatively coarse-grained soils, with high thermal conductivities, that were modeled in the Berggren solution for the Nashua site.

<u>Closure</u>

A final, major point to be made relative to the containment of asbestos waste in a natural environment of freezing temperatures is that <u>the minimum depth of cover is not a single value but varies with the climate, the type of soil cover, and the soil moisture conditions</u>. The design referred to here is site specific. The actual thickness of a required cover in the northern areas of the U.S. may be 3-5 feet while in southern areas perhaps only one foot would be needed from the standpoint of frost penetration.

CRREL is not aware of any laboratory or field test information providing data on freeze-thaw induced movement of asbestos material in soils used to cap asbestos waste. It is suggested that the EPA view the covered sites in southern New Hampshire as a case study opportunities for obtaining actual field performance data. These data, along with data from laboratory tests are needed to adequately document this rationale. Once this information has been obtained, a comprehensive document providing sound design criteria for asbestos disposal in seasonal frost areas can be developed.

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REFERENCE 5-2

Contract Report to EPA, Region I by U.S. Army Corps of Engineers, New England Division, Engineering Division, Geotechnical Engineering Branch, <u>EPA Asbestos Abatement Work, New England Division, Engineering Division, Position Paper</u>.

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EPA ASBESTOS ABATEMENT WORK

NEW ENGLAND DIVISION

ENGINEERING DIVISION

POSITION PAPER

Prepared by: U. S. Army Corps of Engineers
New England Division
Engineering Division
Geotechnical Engineering Branch

For: U. S. Environmental Protection Agency
Region 1 (New England)
Lexington, MA

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION. CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254-9149

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REPLY TO ATTENTION OF

December 15, 1987

Engineering - Geotechnical

Mr. Paul Groulx
U. S. Environmental Protection Agency
Region 1
60 Westview Street
Lexington, Massachusetts 02173

Dear Mr. Groulx:

The attached report entitled "EPA Asbestos Abatement Work, New England Division, Engineering Division, Position Paper" is furnished for your information and use. This report contains the rationale developed by the Geotechnical Engineering Branch, including backup documentation, for the cleanup of asbestos waste sites.

Sincerely,

Richard D. Reardon
Chief, Engineering Division

Enclosure

Copy furnished:

Mr. Ferenz - EPA, Lexington, MA

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EPA Asbestos Abatement Work - Geotechnical Engineering Branch Position Paper

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List of Tables

Attachment

Lowell Road Site, Hudson, NH
Engineering Design, Specifications, and On-Site Technical Assistance for Covering Waste Asbestos
U.S. Army CRREL, Hanover, NH
March 1987
For USEPA - Region I (New England), Lexington, MA

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EPA Asbestos Abatement Work - Geotechnical
Engineering Branch Position Paper

NED Policy For Handling and Disposal of Asbestos Waste

1. Background:
   a. CRREL Involvement

(1) 1982: EPA - Region I requested assistance from CRREL for Asbestos Abatement Work at six sites in Hudson, NH. CRREL's Geotechnical Engineering Branch inspected these sites and observed significant evidence of asbestos particle movement. CRREL's report to EPA on the Lowell Road, Hudson, NH site was for the purpose of presenting technical material bearing on the possible effects of annual freezing and thawing. In particular, the determination of the required depth of soil cover to prevent asbestos material from reappearing at the surface was addressed in this report (Encl. A).

(2) August 1983: Messrs. McGaw and Iskandar of CRREL gave expert testimony for EPA in Federal District Court concerning the depth of cover required for these six sites. They recommended an expedient cover of 30 inches of sandy gravel or its equivalent as being sufficient to provide 50 to 100 years of protection. They also testified on the depth of organic topsoil required to sustain a permanent grass cover over the asbestos material, which is very alkaline (pH of 11 or 12). They recommended at least 18 inches of topsoil, because it had been determined that the roots will extend to that depth and would be burned by the high pH levels in the underlying soil. The remaining 12 inches of cover would be a sandy gravel, coarse enough to be of low frost susceptibility, and fine enough to contain sufficient moisture to support the vegetation at the surface.

(3) October 1983 - January 1984: CRREL-GEB advised EPA on the cover specifications for several sites at Hudson, NH.

(4) May - November 1985: On site inspections, guidance and a technical report prepared for Lowell Rd, Hudson, NH site by CRREL.

b. NED/GEB Involvement:

(1) 16 March 1987: Joint Geotechnical Engr. Branch meeting with CRREL. NED solicited background information on past CRREL involvement and the latest technical material bearing on the possible effects of annual freezing and thawing on waste disposal sites in Hudson, NH.

(2) March 1987: GEB telecon with Noel Urban, OCE, General Engineering Branch, Engineering & Construction Directorate on the latest technology for non-frost criteria for capping hazardous material asbestos.

(3) March 1987: GEB telecon with Mr. Paul Schroeder at USCE Waterways Experiment Station.

2. Design Rationale For Determining Depth of Soil Cover For Asbestos.

a. Frost Susceptible Sites

This rationale is intended to provide guidance and relates specifically to the asbestos burial sites in Nashua and Hudson, New Hampshire. It is based primarily on laboratory and field experience with freezing and thawing of soils containing inclusions.

The primary objective of this design is to determine the amount of soil cover needed to significantly reduce the number of times that the frost zone penetrates the asbestos material. The main concern is that if the soil with asbestos or simply asbestos is allowed to freeze and thaw, a process called sorting may bring the asbestos pieces to the ground surface and expose the environment to their hazardous effects.

When a moist soil freezes, water within the soil is drawn toward the freezing front, i.e. where freezing is taking place. A mass of ice is formed within the voids of the soil which tends to push the particles apart. Normally the direction in which the particles of soil are most free to move is toward the surface, resulting in frost heave.

If a source of free water is available, such as a water table within 10 ft. of the surface, the ice masses will continue to draw water from below, eventually forming what are typically termed ice lenses. Significant volume changes occur in a soil when ice lenses form. Not only will the ground surface rise (i.e. heave) anywhere from 1 inch to 12 inches during a winter season, but individual particles will be moved closer to the surface.

During the subsequent spring thawing period much of the frost heave will be recovered: the ground normally subsides more or less back to its former volume. In order for this to happen, excess water stored in the ice lenses must somehow drain away; but until it does, the soil will be soft and easily deformable.

Because the soil is temporarily soft, the finer (smaller) particles of soil tend to move down and under the larger particles, holding them in the positions to which they moved when they were frozen. When the soil finally becomes more stable in the summer, the larger particles end up closer to the surface than they were in the previous summer. Because this sorting process takes place year after year, stones and other particles may sometimes move significant distances toward the ground surface. The process is likely to be accelerated when the particles approach the ground surface, because this zone experiences multiple freeze/thaw cycles during a single winter season.

It is assumed that asbestos chunks and particles are likely to be affected by the seasonal process of freezing and thawing in much the same manner as stones and other natural inclusions. Freeze/thaw cycles may have two types of long-term effects on the permanent containment of waste asbestos as a result of:

(a) The differential movement of foreign matter toward the surface with each cycle of freezing and thawing may eventually bring chunks and pieces of asbestos to the ground surface and thereby return them to the external environment; and

(b) Asbestos scraps (broken boards, chunks, even cemented pellets) may be broken into smaller pieces as moisture held within the scraps freezes, so that when they reach the surface they are sufficiently small to be environmentally hazardous.

A prospective long-term means of preventing asbestos particles from moving toward the surface through the freezing and thawing process is to make sure that they are never frozen. However, because economic considerations must be taken into account, the design presented here is based on an attempt to contain the waste asbestos below the ground surface for a minimum period of 100 years. The actual safe lifetime is probably well in excess of 100 years, provided the removal of covering material by erosion or human activities is effectively controlled. This design assumes a turf cover with a well established root zone. It does not include a safety factor to account for construction or garden cultivation.

The procedure used here to select an appropriate depth of soil cover to contain asbestos material below the surface in southern New Hampshire is a modification of a procedure developed by the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) for designing pavements against failure caused by freezing and thawing (Dept. of the Army and Air Force, 1985). It employs the use of the modified Berggren equation for layered systems.

This procedure has been validated with more than 30 years of experience by the Corps of Engineers in airfield and highway design. The technique has been shown to yield excellent results, when applied to in situ phase changes in soils.

Basically, the ground freezes because of the removal of heat. It is therefore clear that the depth of frost penetration below the surface is related to the meteorological variables that affect heat loss and the physical properties of the soil. The major variables are (1) the severity of the winter air temperatures, and (2) the amount of snow cover. The severity (coldness) of the air temperature is expressed in terms of an air freezing index for each winter. The efficiency of thermal transfer between the air and the ground or snow is expressed in terms of an n-factor, where n is a fraction between 0 and 1. In combination, the two parameters result in a surface freezing index, which is a measure of the amount of heat leaving the ground over an entire winter. The manner in which these values are utilized to calculate the suggested depth of cover for safe burial of waste asbestos is described in a report entitled, "Lowell Road Site, Hudson, NH; Engineering Design, Specification, and On-Site Technical Assistance for Covering Waste Asbestos" (Attachment No. 1).

b. Non-Frost Susceptible Sites

This rationale is intended to provide guidance for design of asbestos cover systems in areas where frost is not a primary concern. It is based on experience with erosion control structures, research studies performed by the USCE Waterways Experiment Station (WES) and the USCE Cold Regions Research and Engineering Laboratory (CRREL), and EPA report No. EPA/540/2-85/002 "Covers for Uncontrolled Hazardous Waste Sites", dated September 1985.

The EPA has identified five requirements which a cover system for certain hazardous wastes must meet, as follows:

(1) Provide long-term minimization of migration of liquids.

(2) Function with minimum maintenance.

(3) Promote drainage and minimize erosion or abrasion of the cover.

(4) Accommodate settling and subsidence so that the cover's integrity is maintained.

(5) Have a permeability less than or equal to the permeability of any bottom liner system or natural subsoils present.

Since asbestos waste material is not known to migrate through soil with the flow of groundwater and is not considered to be a water pollution hazard, items (1) and (2) above can be deleted for asbestos cover systems. EPA guidance relative to meeting requirements (3), (4), and (5) states that a cover system should consist of a vegetated top cover underlain by a drainage layer. Detailed guidance from EPA on these two layers is as follows:

A) Vegetated Top Cover

(1) minimum 24 in. thick.
(2) should support vegetation that minimizes erosion without continued maintenance
(3) planted with persistent species - no roots that will penetrate beyond the vegetative and drainage layers
(4) top slope, after settling and subsidence, of between 3-5%
(5) surface drainage system capable of conducting runoff across cap with no erosion problems.

B) Drainage Layer

(1) minimum 12 in. thick
(2) saturated conductivity not less than $1x10^{-3}$ cm/sec
(3) bottom slope of at least two percent
(4) designed to prevent clogging - overlain by a graded granular or synthetic fabric filter
(5) discharge flows freely

The granular or fabric filter is used to prevent plugging of the porous media with fine particles carried down from the vegetated layer.

To prevent fluid from backing up into the drainage layer, the discharge at the site should flow freely (the edge of the unit should drain freely, e.g., into surface runoff ditch).

An additional design consideration in non-frost areas is potential damage to a cover system by burrowing animals. To prevent this from occurring WES recommends placement of an "armor" layer beneath the vegetative cover layer.

The EPA criteria is intended for use on relatively flat slopes and are not universally applicable. WES recommends a 10% (1V on 10H) "practical maximum" slope for a vegetated cover. For slopes between 10% and 20% some type of soil reinforcement should be used to hold the topsoil until sod cover is established. Slopes greater than 20% (1V on 5H) should be provided with slope protection.

c. Cover Specifications

(1) Frost Susceptible Sites

At the Nashua/Hudson asbestos site, the majority of the restored area was slated to be revegetated. Therefore, the initial consideration in designing the cover materials was the necessity to support vegetation, specifically grasses.

Because the asbestos waste is extremely alkaline, having a pH of 11 or 12, it is highly toxic to plant life. Therefore, the minimum depth of cover required is that which will be deep enough to keep the grass roots from penetrating the asbestos. Studies have shown that grass roots extend down to 18 inches beneath the surface so a cover at least that deep is required to assure the long-term survival of the vegetation.

Nutrients are also needed for vegetation survival. It is therefore recommended that the upper 6 inches (compacted layer thickness) of cover include topsoil to provide nutrients for the grasses. A 2:1 ratio of topsoil to sand would provide nutrients and allow drainage. Beneath the topsoil, the cover material should consist of a 6-inch layer of fine sand, underlain by an adequate amount of sandy gravel to reduce problems with frost penetration. The sand layer acts as a filter between the topsoil and gravel, retains moisture and will eventually form a subsoil containing organics.

In order to determine the amount of gravel to recommend, frost penetration was modeled by CRREL using a multilayer solution of the modified Berggren equation with a soil profile consisting of 6" topsoil, 6" fine sand, and an indefinite amount of sandy gravel. Table A (from Enclosure A Appendix, Table A3) shows the physical and thermal characteristics of the modeled materials for both a wet and dry case. Properties were chosen so as to be generally representative of the materials at the site. The left-hand portion of Table B shows the predicted frost penetration depths at the design freezing index using the soil profile with a variable thickness of gravel.

In the most extreme case, that with no snow, frost penetration is on the order of 34-35 inches. When a snow layer at the surface is included, frost penetration decreases as the snow thickness increases. The next aspect to be considered, then, is what depth of snow cover should be used for the design. The duration of snow cover is another important aspect. However, for the purposes of this design, it was assumed that the duration of snow cover was similar to the length of the freeze season, or about 100 days.

Table A. Characteristics of Materials Modeled for Frost Penetration.

Layer Type	Moisture Content (%)	Dry Density (pcf)	Heat cap. (Btu/cuft °F)	Thermal Cond. (Btu/fthr °F)	Latent Heat of Fusion (Btu/cuft)
a) Wet Soil					
Topsoil	25	90	32.18	0.85	3240
Sand	15	100	28.25	1.15	2160
Gravel	10	120	29.40	1.59	1728
Asbestos	15	95	26.84	0.68	2052
b) Dry Soil					
Topsoil	18	95	28.62	0.75	2394
Sand	10	110	26.95	1.20	1584
Gravel	10	120	29.40	1.59	1728
Asbestos	15	95	26.84	0.68	2052
c) Snow	0	15	9.50	0.18	0

Table B. Frost penetration depths predicted by a multilayer solution of the modified Berggren equation using climate parameters typical for Nashua (freezing index = 1097°F-days; mean annual temp = 46.4°F; length of freeze season = 100 days).

	FROST PENETRATION (IN.)			
	Variable cover Thickness[1]		30-Inch cover Thickness[2]	
	Soil Condition		Soil Condition	
Snow Depth (in.)	Dry	Wet	Dry	Wet
0"[3]	34.4	35.4	33.4	34.0
4"[4]	31.7	30.7	31.3	30.4
6"	24.8	23.0	24.8	23.0
8"	19.8	17.7	19.8	17.7
10"	16.3	13.3	16.3	13.3

[1]soil profile: 6" topsoil
6" fine sand
indef. gravel

[2]soil profile: 6" topsoil
6" fine sand
18" gravel
30" total

[3]n-factor = 0.5 (turf)
[4]n-factor = 1.0 (snow)

Depth of snow measurements were recorded at the Nashua weather station between 1952 and 1976, with some periods of missing data (U.S. Dept. of Commerce, 1950-1980). Complete daily records are available for 15 winter seasons. To get a mean snow depth value for each winter season, the daily depth data between 1 December and 16 March were summed and divided by the number of days. The distribution of these mean seasonal depths are shown in Enclosure A, Figure A3.

The mean snow depth for the 15 years of data is 6.2 inches and the probability that Nashua will have less than this mean amount is 50-percent. The probability that the mean snow depth will be less than 4 inches is about 25 percent. These two snow depths, then, bracket the amount of snow that might be considered for design purposes.

When the 15-year mean snow depth, 6 inches, is modeled in the Berggren solution, a frost penetration of about 24 inches is predicted (see Table B). Using a 4-inch snow depth, a frost penetration of 30 inches is predicted. For the Nashua/Hudson sites, it was decided to use the 4-inch mean snow depth and the resulting 30 inch minimum cover depth as a final recommendation. This conservative decision resulted from the lack of knowledge about actual rates that asbestos might move through a soil column subjected to cyclic freeze-thaw.

Frost penetration depths were modeled using the Berggren equation with a 30-in. cover (right-hand side, Table B). Again, material properties used were those shown in Table A. Because of the lack of actual data, the asbestos properties had to be estimated. The values used were based on the assumption that the top layer of asbestos would have partially mixed with the existing thin topsoil cover. The frost depths predicted by the model would penetrate 4 inches of the asbestos in a no-snow year with the design freezing index. In years with a mean snow depth of 4 inches or greater, the frost would be contained in the cover material.

In summary, the thickness of cover recommended at the present time by CRREL for the restoration of gently sloping terrain at waste asbestos sites in Nashua and Hudson, New Hampshire is 30 inches. This cover should consist of the following three layers from top to bottom: 6 inches of topsoil and sand mixed in a 2:1 ratio, 6 inches of fine sand, and 18 inches of sandy gravel.

(2) Non-Frost Susceptible Sites

In a non-frost susceptible site, the criteria for the vegetative cover layer discussed above for a frost susceptible site is applicable. Therefore, in order to provide for long term survival of the vegetative cover, a minimum of 18 inches of soil must be placed over the asbestos waste material. The top six inches should be a 2:1 mix of topsoil and sand and the underlying six inches should be fine sand.

The requirements for a drainage layer and an armor layer beneath the vegetative cover layer can be satisfied by a single layer of sandy gravel containing stones up to three inches in size. A 6-inch to 12-inch thick layer of this material when properly graded will provide adequate drainage, and will be difficult for burrowing animals to penetrate. The fine sand layer will function as a filter between the topsoil and the gravel layers, and will prevent clogging of the drainage layer.

In summary, a cover system for asbestos waste in gently sloping terrain should consist of a vegetative cover layer and a drainage/armor layer. These two layers are further defined from top to bottom as follows: 6 inches of topsoil and sand mixed in a 2:1 ratio, 6 inches of fine sand, and 6 to 12 inches of sandy gravel. For slopes between 10% (1V on 10 H) and 20% (1V on 5H) soil reinforcement should be used to hold the topsoil until sod is established, and for slopes steeper than 20%, the vegetative cover layer should be replaced with slope protection.

(3) Closure

A final, major point to be made relative to the containment of asbestos waste in a natural environment of freezing temperatures is that the minimum depth of cover is not a single value but varies with the climate, the type of soil cover, and the soil moisture conditions. The design referred to here is site specific. The actual thickness of a required cover in the northern areas of the U.S. may be 3-5 feet while in southern areas perhaps only one foot would be needed from the standpoint of frost penetration.

CRREL is not aware of any laboratory or field test information providing data on freeze-thaw caused movement of asbestos material in soils used to cap asbestos waste. EPA will be requested by NED to view the covered sites in southern New Hampshire as a case study with opportunities for obtaining actual field performance data. These data, along with data from laboratory tests, are needed to adequately document this rationale. Once this information has been obtained, a comprehensive document providing sound design criteria for asbestos disposal in seasonal frost areas can be developed by CRREL.

3. Cover Design

a. Design Concept

(1) Frost Susceptible Sites: As with other waste disposal sites in the town of Hudson and Nashua, the primary design principal is to contain the waste asbestos by means of long-term burial with appropriate soil layers. A soil cover and surface treatment should be selected that would restrain the asbestos particles from moving toward the surface for a 50 to 100 year period under the effects of freezing and thawing. It is also very important that the surface treatment be such as to eliminate erosion of the burial layers through the actions of wind, stream flow, surface runoff, and subsurface flows.

(2) Non-Frost Susceptible Sites: In a non-frost susceptible site the primary design principal is to encapsulate the waste asbestos and prevent its exposure to the environment for a 50 to 100 year period. The cover layers must be selected to protect against the erosive effects of wind, stream flow, surface runoff, subsurface flow, and burrowing animals.

b. Air and Subsurface Investigation. Surface and subsurface soil samples were taken by EPA to investigate the areal extent and depth of contamination at and around the site. Wipe samples were also taken from buildings and equipment on-site. Air sampling was not taken.

c. Characteristics of Foundation, Asbestos, and Cover Materials

(1) Foundation Soils. Foundation soils are site specific and vary from site to site. The soils in Hudson and Nashua, NH include glacial till (SM), man-made fills (other than asbestos) and outwash sands and gravels (SP-SM, GP-GM).

(2) Clearing and Grubbing. Foundation areas for fills shall be cleared and grubbed prior to placement of any fill material. This work shall consist of clearing and grubbing (only where specifically

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designated), removing and disposing of all vegetation and debris within the limits shown on the plans. In general, all trees and vegetation should be cut at or near the ground surface and disposed of except those trees specifically designated on the plans and in the specifications to remain. Unless otherwise shown on the plans, clearing and grubbing shall extend 10 feet beyond fill slopes. Stumps within the limits of the fills shall be cut flush with the existing ground surface. All stumps and large roots shall be completely covered to a depth of 2.5 feet with fill material in order to prevent regrowth of the species.

(3) Asbestos. Asbestos is a commercial term applied to a group of highly fibrous silicate minerals that readily separate into long, thin, strong fibers. There are two properties of asbestos, which make it particularly suitable for disposal by sanitary landfill:

(a) The mineral fibers resist degradation and are inert and insoluble in water. As such, they do not represent a threat to ground-water supplies as the result of leaching.

(b) Because of its fibrous nature asbestos tends to lodge in the voids between individual grains of sand and gravel, unless the material at the point of land disposal is exceptionally coarse or the area is subject to flooding.

The various asbestos waste sites in the Hudson and Nashua, NH contain buried, fully exposed, and partially exposed asbestos waste deposits. This material is generally comprised of aged, deteriorating plate scrap, intermixed with friable baghouse material, including white, grey, black, and red varieties.

Friable asbestos material is defined as any material that contains more than one percent asbestos by weight and that can be crumbled, pulverized, or reduced to a powder, when dry, by hand pressure.

(4) Topsoil is defined as surface soils containing at least 3% by weight of organic material. Municipal sludge (no industrial sludge allowed) containing 2 to 9% solids can be substituted for native topsoil. Local ordinances prohibit stripping of topsoil. Two parts of topsoil could be mixed with one part fine sand (by volume) to provide nutrients and allow drainage. The minimum compacted layer thickness is 6 inches (8 to 9 inches loose thickness).

(5) Fine Sand should act as a filter between the overlying topsoil and underlying non-frost susceptible sandy gravel fill. In general, non-uniform soils containing less than 3% of grains finer than 0.02 mm and uniform soils containing less than 10% of grains finer than 0.02 mm are suitable. Gradation criteria will be specified, however, because the time to conduct hydrometer and specific gravity tests in the field to determine the percent finer than 0.02 mm is considered to be too long. Fine sand shall meet the gradation specifications for fine concrete

aggregate (State of NH Standard Specifications) or shall consist of bank-run medium fine or fine sand meeting the following gradation requirements:

U.S. Standard Sieve Size	Percent Passing by Dry Weight
3 inch	100
No. 4	80-100
No. 200	0-15

(6) Sandy Gravel Fill should be a non-frost susceptible sandy gravel or gravelly sand with a maximum particle size of 3 inches. Gradation criteria will be specified because the time to conduct hydrometer and specific gravity tests to determine the percent finer than 0.02 mm is considered too long. Sandy gravel fill material shall consist of well-graded sandy gravel composed of tough, durable particles of natural sand and gravel except that particles larger than 3/8 inch may be crushed stone. The material shall meet the following gradation limits (State of NH Standard Specs for Granular Backfill - Sand/Gravel):

U.S. Standard Sieve Size	Percent Passing By Dry Weight
3-inch	95-100
No. 4	25-70
No. 200	0-8*

* In no case shall more than 10% by dry weight of the component passing the No. 4 sieve pass the No. 200 sieve. The maximum size stone particles shall not exceed 3/4 of the compacted thickness of the layer being placed.

(7) Slope Protection. Slope protection can be constructed from a variety of materials, including stone and concrete which are the most durable in water. Revetment types will include riprap, gabions, and concrete armor units. The type and layer thickness will be site specific, based on hydraulic design criteria.

(8) Crushed Stone Fill for retaining and protecting specifically designated trees and shrubs shall consist of quarried rock fragments varying in size from 3 inches to 5 inches.

d. Slopes

(1) Riverside slopes shall be designed for embankment stability. Fully compacted fills generally enable the use of steeper slopes than those constructed of semi-compacted dumped fills. A 1V on 2H slope is generally accepted in industry standards as the steepest slope that will permit machine placement of riprap and also the steepest slope that will ensure stability of the riprap blanket. Riverside slopes will be designed to provide protection from damage by wave and ice action, stream flow (for a 100 yr. frequency flood) and surface and subsurface erosion (see Sketch No. 1). Slopes are assumed to be stable for 50 to 100 years with minimal maintenance.

(2) Other slopes shall be designed to provide protection from frost action, and surface and subsurface erosion. All slopes steeper than 10 degrees (IV on 6 H) shall be protected by slope protection consisting of stone protection, gabions, or concrete armor units (see Sketch No. 2). All trees and vegetation shall be cut flush with the existing ground surface, removed from the site, and the slope covered with a gravel fill blanket to a minimum thickness of 18 inches. The total depth of cover in all cases in Nashua, NH shall be a minimum of 30 inches excluding any slope protection. Slopes flatter than IV on 6H shall be protected similar to level or gently sloping ground.

e. Level or Gently Sloping Ground. Gently sloping ground is defined as flatter than 10 degrees (approx. 1V on 6H). The effective cover should be at least 30 inches (see Sketch No. 3) to prevent the asbestos from being returned to the surface through the yearly process of freezing and thawing, and to provide 50 to 100 years of protection with minimal maintenance. The restored areas should be vegetated to prevent surface erosion. Paved surface drainage features should be utilized to collect and channel runoff in order to prevent surface erosion which could eventually lead to exposure of the underlying asbestos.

f. Maintenance. EPA is not authorized to fund maintenance of completed asbestos waste clean up projects. Land owners are responsible *for maintenance of their own properties.* At sites where maintenance will be performed, mowing of grasses should be done two or three times each growing season to stimulate growth. In the Spring, the distribution of grass growth should be monitored closely, so that reseeding can be done early in those areas showing less than adequate coverage with grass. Table C contains a maintenance procedure for grasses after the first year of soil cover.

g. Monitoring of Sites. EPA will be requested to view the covered sites in southern New Hampshire as a case study opportunity for obtaining actual field performance data. Data from laboratory tests are needed to adequately document this rationale. Just a few of the sites should be selected for monitoring. Tests should include visual observations, sampling (hand auger), and chemical testing. Instrumentation (thermocouples) should be installed during construction to measure temperature and frost penetration.

h. Settlement. Most of the asbestos materials were deposited over 20 years ago. Only minor settlements are expected due to the addition of the new fills. Depressions which develop in grassed areas should be regraded utilizing additional topsoil and then reseeded by the owners. In areas protected by slope protection, settlement caused by decaying stumps or consolidation of the underlying asbestos should be refilled with appropriate slope protection materials.

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TABLE C

Management of Asbestos Disposal Sites After the First Year of Soil Cover

1. In mid-April, inspect the germination of the grass seed visually.

2. Spot reseed the area where seeds did not germinate (no color change observed).

3. Soil samples should be taken and tested for N, P and K, and recommendations on fertilizer application should be followed. At minimum, if no tests are done or will be done, 40 pounds of N fertilizer should be applied per 1,000 sq. ft.

4. If hydroseeder will be used to reseed a large area, 100-150 lbs of solids per 100 gallons of water is the maximum mixture. Solids include fertilizer, seeds and mulch materials.

5. Grasses to be used are Perennial Ryegrass, Tall Fescue, Kentucky Bluegrass, Reed Canarygrass or Birdfoot Trefoil. The following are the recommended combinations and amounts of each:

a.	Switchgrass	5	(PLS)*
	Bluestem (big or little)	5	(PLS)*
	Perennial ryegrass	5	
	Birdfoot trefoil**	5	
	Total		20lb/AC
b.	Tall fescue	20	
	Flat pea	30	
	Total		50lb/AC
c.	Deer tongue	10	(PLS)*
	Birdfoot trefoil	8	
	Perennial ryegrass	3	
	Total		21lb/AC
d.	Deer tongue	10	
	Crownvetch**	15	
	Perennial ryegrass	3	
	Total		28lb/AC

* PLS pure line seed = $\frac{\text{\% germination} \times \text{\% purity}}{100}$

Actual lbs of commercial seed to be used = $\frac{100 \times \text{lbs of 100\% PLS}}{\text{\% PLS of commercial seed lot}}$

**Inoculate legume seeds; use four times the recommended amounts when hydroseeding.

6. Recommended Variety
   Tall fescure (Kentucky 31)
   Birdfoot trefoil (Empire)
   Switchgrass (Blackwell)
   Perennial ryegrass (Norlea, Manhattan)
   Flatpea (Latheo)

i. Methods of Placement and Compaction

(1) Spreading - Material for fills shall be spread with bulldozers or other approved equipment or by hand to form uniform loose layers of the following thicknesses:

Material	Maximum Loose Layer Thickness in Inches	
	General	Restricted Areas
Topsoil	8	6
Comp. Sand Fill	8	6
Comp. Gravel Fill	10	6

(2) Compaction - Materials for compacted fills shall be compacted as follows according to its fill type:

Fill Type	Compaction
Topsoil	Roll surface with standard landscaping rollers to create smooth surface.
Comp. Sand	At least 2 coverages of the tread of the heavy tractor or at least 4 coverages of tread of the light tractor.
Comp. Gravel	At least 4 coverages of the tread of the heavy tractor or at least 6 coverages of the tread of the light tractor.

(3) Compaction in Restricted Areas - Sand and gravel fill in a restricted area shall be compacted by the plate vibrator.

(4) Equipment - Compaction equipment shall conform to the following requirements and shall be used as prescribed in subsequent paragraphs.

(a) Heavy Tractor - A "heavy tractor" to be used for compacting fill material shall be a standard commercial make crawler type tractor weighing not less than 35,000 pounds and exerting a tread pressure of not less than 9 pounds per square inch. The tractor shall be equipped with standard width treads.

(b) Light Tractor - a "light tractor" to be used for compacting fill material shall be a standard commercial make crawler type tractor weighing between 7,500 and 12,000 pounds and having a width of 5-1/2 feet or less, measured between the outside edges of the crawler tracks.

(c) Plate Vibrator - A plate vibrator shall be an approved plate surface vibrator designed for the compaction of soils by vibration and the product of a manufacturer nationally recognized as a specialist in the design and manufacturer of such equipment. The surface contact plate shall be between 15 and 18 inches in width.

(5) Dumped Fills. Dumped gravel fill shall be placed by end dumping the material on a working surface and then pushing it into position by crawler dozer or similar equipment or by bucket lowering into the bottom and releasing the load on the bottom. Dumped fill material may be placed without dewatering of the foundation area.

APPENDIX A

EPA ENFORCEMENT APPROACH TO ASBESTOS SITE CLEANUP

APPENDIX A

EPA Enforcement Approach to Asbestos Site Cleanup

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APPENDIX A

EPA Enforcement Approach to Asbestos Site Cleanup

1. General: The following paragraphs were abstracted from a paper entitled, "U.S. EPA Enforcement Approach To Asbestos Site Cleanup" by Deborah S. Dalton (date unknown), U.S. Environmental Protection Agency, Office of Water Programs Enforcement, Washington, D.C.

2. Introduction: Superfund has been used to accomplish cleanup at a number of National Priorities List (NPL) and non-NPL sites, where the primary or only contaminant of concern is asbestos. National Priorities List sites include: Mountain View Mobile Home Estates, Globe, Arizona; Coalinga Asbestos Mine, Coalinga, California; Atlas Asbestos Mine, Fresno County, California; Ambler Asbestos Piles, Ambler, Pennsylvania. Non-NPL sites include ten sites in and around Hudson, New Hampshire; the Jaquays Mill site, Globe Arizona; and the Lloyd Hodges site, East Chicago, Indiana. Many of these were or are involved in enforcement action under RCRA '7003 or CERCLA '106.

In this paper the author will describe the approach taken in selecting remedial action at sites which are/were subject of intense enforcement involvement. Considerations used in selecting or approving the appropriate remedies at these sites will be outlined and the final cleanup actions will be described.

3. Asbestos:

a. Scientific Status

The definition of asbestos listed in the Glossary of Geology is:

(1) A commercial term applied to a group of highly fibrous silicate minerals that readily separate into long, thin, strong fibers of sufficient flexibility to be woven, are heat resistant and chemically inert, and suitable for uses (as in yarn, cloth, paper, paint, brake linings, tiles, insulation cement, fillers and filters), where incombustible, nonconducting, or chemically resistant material is required.

(2) A mineral of the asbestos group, principally chrysotile (best adapted for spinning) and certain fibrous varieties of amphibole (example: tremolite, actinolite, and crocidolite).

Inhalation of asbestos fibers is known to cause cancer in humans. Specifically, exposure to asbestos can cause bronchogenic carcinomas in the lung and pleural and peritoneal mesotheliomas after a latency period of up to 30 years. Asbestos is also known to lead to respiratory asbestosis, characterized by fibrosis calcification and fibrosis of the pleura. There is very limited information from which to infer the danger of cancer from ingestion of asbestos fibers in food or drinking water.

b. Legal Status

Asbestos is listed as a hazardous air pollutant under the Clean Air Act (CAA), Section 112. Asbestos air emissions are regulated by the Nation Emissions Standards for Hazardous Air Pollutants (NESHAPS) at 40 CFR Part 61, Subpart M. Asbestos is listed as a toxic pollutant under Section 307(a)(1) of the Federal Water Pollution Control Act (FWPCA). Asbestos is regulated in work places by OSHA (29 CFR Part 1910) and in schools by the U.S. EPA under the Toxic Substances Control Act (TSCA).

Although asbestos is not a hazardous waste listed under the RCRA regulations (40 CFR Part 261), its disease-causing properties meet the standards of the statutory definition of RCRA 11004 (s). This toxic property of asbestos allows use of the substantial hazard standard of RCRA '3013 and the imminent and substantial endangerment standard of '7003 for enforcement purposes. Because of its listing in CAA and FWPCA, asbestos is, by definition, a hazardous substance under CERCLA '101(14), enabling the U.S. EPA to take removal or remedial action with the Superfund or to take enforcement action for cleanup through administrative orders or judicial action under '106, and for cost recovery under '107.

Several enforcement actions have tested the U.S. EPA's response and enforcement authorities. The U.S. EPA has prevailed in these actions. In 1983, after oral argument and testimony in U.S. v. Johns Manville et al., the U.S. District Court in New Hampshire found "that there has been a release or there exists a substantial threat of a release in the environment of a hazardous substance as contemplated by '104(a)(1) of CERCLA" and ordered two defendants to allow the U.S. EPA access to their property to conduct a removal action (installation of a cap). In 1984, in U.S. v. Metate Asbestos et al., the U.S. District Court of Arizona found, on a partial summary judgement motion, that asbestos is a hazardous substance under the definition at '101(14) by virtue of the fact that asbestos is regulated under Section 307(a) of FWPCA and under Section 112 of Clean Air Act. The court ruled against the defendants' interpretation of the RCRA exclusion of mining wastes.

4. Site Investigation/Characterization:

a. General

The development and selection of remedies at asbestos sites varies little in its process from the process used at any other hazardous waste site. On one hand, decisions are made easier because there is no information indicating subsurface lateral or downward movement of asbestos in a landfill and asbestos in not a regulated hazardous waste under RCRA subtitle C. However, site investigations are made more difficult because analysis and quantification of asbestos is both complex and difficult to interpret.

b. Sampling

As with any other hazardous waste site, it is important to determine the scope of the contamination and possible movement or transport of the hazardous substance off-site. Asbestos sites dealt with under Superfund have primarily been the result of waste disposal from mining, milling or manufacturing facilities in the immediate area of the site. Asbestos contamination of soils is the result of either on-site waste disposal activities or the result of off-site deposition of asbestos particles through soil erosion from surface water or wind. There is no evidence to date of significant subsurface downward or lateral migration in soils. However, there may be upward movement of asbestos particles or products due to freeze-thaw effects common to rock migration in northern and New England soils. There is no documentation of groundwater transport of asbestos particles.

Subsurface soil samples and soil cores should be taken to investigate the areal extent and depth of contamination at and around the site. Site vegetation can be sampled after a wind or rain storm to investigate whether asbestos may have been transported from the soil, into the air and resettled on vegetation. Wipe samples should be taken from buildings or equipment on-site. If the site contains buildings or equipment which have air filters, these filters can be sampled.

c. Analysis

There are a number of uncertainties resulting from the difficulties of selecting a method of analysis, performing the analysis and interpreting and applying the results. A number of methods are used for the identification and quantitation of asbestos in air, water and soils. Optical polarized light microscopy, transmission electron microscopy, scanning electron microscopy and x-ray diffraction are useful but limited methods.

In general, it can be logically argued that asbestos documented on the surface of a site can be, and is, transported off-site by wind and by surface water runoff, to later be available for re-entrainment and subsequent exposure.

5. Feasibility Studies/Remedial Design:

a. General

At this point in time, there is only one option for permanent disposal of asbestos; that option is burial. There are several ways to accomplish this result depending on the size of the site and the volume of asbestos-contaminated soil: (1) excavation, transport, off-site landfilling; (2) burial in an on-site pit or landfill; (3) cap in place.

Because asbestos is neither a waste listed nor regulated under RCRA, disposal sites do not have to conform to Subtitle C standards. Off-site disposal of asbestos wastes from a Superfund site may require a justification to be exempt from the U.S. EPA/OSWER Off-site Disposal Policy which requires Superfund wastes to be disposed of only at sites with RCRA permits and a good compliance record. At the time of this paper, the issue had not been raised on a site-specific basis. However, adequate arguments that asbestos wastes do not require the groundwater protections inherent in the Subtitle C landfill permitting process can be made. General requirements for solid waste disposal under RCRA do apply (40 CFR Part 257).

Under the Clean Air Act, NESHAPS requires closure of an asbestos site by covering the asbestos material with at least 6 inches of compacted clean fill material and vegetation or 24 inches of compacted clean fill material (no vegetation) or a resinous or petroleum based duct suppression agent. The drawback of the latter method is that the duct suppression agent must be reapplied at least yearly to maintain maximum effectiveness.

A general discussion of asbestos waste management is given in U.S. EPA publication number 530-SW-85-007, May 1985, entitled "Asbestos Waste Management Guidance." Choice of a remedial option should be based upon a number of factors in addition to RCRA and NESHAPS. In some cases, the minimum required standards may be inadequate for a long-term remedial response. As mentioned above, dust suppression agents have a finite short life, 6 inches of fill may not be an adequate cap in difficult climates or steep topography and vegetation may be difficult to establish or maintain. In its enforcement actions, the U.S. EPA has focused on obtaining a remedy adequate for 30 to 50 years.

The following considerations have been used in selecting, recommending and/or approving remedies at RCRA and Superfund enforcement sites:

- Present site use, extant buildings and structures
- Site accessibility to the public
- Concentration of asbestos in the soil or wastes
- Volume of asbestos contaminated soil or wastes
- Areal extent of surface contamination
- Depth of contamination
- Site safety procedures during remedial work
- Topography
- Climate - temperature, rainfall, storm events
- Vegetation establishment and maintenance
- Future maintenance requirements
- Future use

b. Decision #1: On-Site or Off-Site

The first decision to be made is whether the remedial action should take place on-site or off-site. The primary considerations in making this decision are site use, site accessibility and the concentration and volume of asbestos contaminated soil on the site. Future use may also be a consideration if the site is zoned for residential or industrial use.

The extent and volume of contamination contribute directly to the decision as to the practicality and cost of excavation and off-site transport. On several sites in New Hampshire, asbestos manufacturing bag wastes were used as fill in marshy areas or ravines. Because of the depth of possible excavation and the large amount of waste to be transported, it was deemed more cost effective to cap the wastes in place. The concentration of asbestos in the soil and the accessibility of the site also contribute to the decision to excavate or cap in place.

On several smaller sites in New England, there was only a small surface area a foot or so in depth of contaminated soil. In these cases, it was more practical and more protective of health to remove the contamination and transport it to the local landfill for proper burial.

Residential sites should be looked at carefully to analyze the types and locations of activity and the locations of asbestos contamination. Certain typical suburban activities such as gardening and landscaping may preclude on-site disposal or capping.

As a result of a scope of contamination study performed at Mountain View Estates, Globe, Arizona, it was found that there was a fairly uniform distribution of asbestos over each residential lot. To adequately protect residents continuing to reside in the subdivision, there were three options other than permanent relocation: (1) installation of a cap in excess of 5 feet, (2) installation of a lesser cap with restrictions on any gardening, or heavy use of the lots for recreational uses or (3) complete removal of all asbestos-contaminated soil. These options were rejected upon consideration of these and other factors discussed below. At one site in New Hampshire, a pocket of asbestos contaminated fill was removed from a residential lot because it was deemed more protective to health and was feasible and cost effective to excavate.

A decision to transport off-site necessitates excavation of the wastes; and consequently, the health and safety of workers and nearby residents during the excavation and transport of the wastes, whether to an off-site or on-site landfill, is of concern. The scope of intensity of protective measures will affect the cost and feasibility of the job and the oversight required.

c. Residential Site Considerations

If there are residences on or adjacent to the site, sampling of settled dust should be done in those residences to determine whether asbestos has been transported from the site into them. Where there is information that asbestos attributed to the site is present in any building (aside from asbestos that may have been installed as insulation, siding or flooring) a decision must be made as to the feasibility of cleaning the building and its contents. A company skilled in asbestos cleanup in buildings should be consulted in the early stages of design to determine the best procedures and timing for cleaning.

It may be necessary to temporarily relocate nearby residents during times of intensive sitework and building cleanup activities. The U.S. EPA temporarily relocated several families adjacent to sites in New Hampshire based on recommendations of the Centers for Disease Control and the judgement of the On-Scene Coordinator assisted by an industrial hygienist.

Permanent relocation is an option that should be considered in the same time period as the decision for off-site or on-site disposal. The U.S. EPA permanently located more than 20 families from mobile homes at the Mountain View Estates in Globe, Arizona, after deciding that the mobile homes could not be cleaned adequately unless the interior walls were removed and the air spaces between the walls were cleaned. Interior walls in mobile homes are not air tight; air and dust can infiltrate the spaces between the walls. One option was to purchase new trailers for installation on site. However, because of the uniform distribution of the asbestos contamination of the site, it was felt that a cap in excess of 5 feet might be needed to allow for normal suburban residential activities on the property. This was deemed to be not cost effective. The residents were brought out; title to the property was assumed by the State of Arizona.

d. Decision #2: Cap in Place or Create On-Site Landfill

If it is decided to complete the remedy on-site, the second decision is whether to cap the contaminated area in place or to excavate and place the material in a burial pit or an on-site landfill. Again, the volume of the waste is a primary consideration.

Topography or the physical characteristics of the disposal site are also considerations because of wind and water erosion. If the wastes are in a large tailings pile or in a steep slope, it may be more secure for the-long term to remove the wastes to a burial pit or an area where they can be leveled off to the surrounding topography. A soil cover over a large steep sided pile may be a measure requiring a high degree of future maintenance because of the increased possibility of erosion. Health and safety considerations for workers and the surrounding population may play a part in the choice of capping in place or excavation to a new landfill site.

e. Cap Design

The design of a cap need not be strictly in accordance with RCRA regulations because, in the case of an asbestos closure, the cap serves a more limited purpose than for normal hazardous wastes; for asbestos, the purpose of the cap is to prevent remergence of the wastes on the surface of the site through the processes of wind and water erosion, freeze/thaw cycles and site use. At U.S. EPA enforcement sites, the nominal depth of the soil cap has varied from 6 inches to 5 feet, depending on topographical features, rainfall, winter temperature extremes, vegetation requirements, future maintenance requirements and future uses. Caps have been finished off with gravel, rip rap and/or vegetation depending on the foreseeable maintenance requirements, climate and aesthetics. In most enforcement actions, the U.S. EPA has been reluctant to accept the 6 inches plus vegetation minimum under NESHAP because of doubts about how long the cap would last due to erosion and continued site use.

The Corps of Engineers at the Cold Regions Research Laboratory in Hanover, New Hampshire has recommended a minimum of 2.5 feet of soil as a cap for New England sites because of research which found that there is an annual upward movement of pebbles, rocks and presumably asbestos particles through the action of freezing and thawing. They recommend that the top of the asbestos layer be lower than the mean freeze line in the soil after the cap is installed.

The Arizona-Nevada Area Office of the Corps recommended a minimum of 2 feet of cover fill material at the Mountain View Estates site because of the desert climate and the potential for heavy storm erosion. The State of Arizona will assume maintenance responsibilities for this site after construction. However, a 5 foot layer of soil was chosen at the adjacent Jaquays site because of the higher concentration of asbestos in the tailings and the need to design a remedy with minimal future maintenance by the owner/operator.

Liners have been used at several sites, primarily to stabilize excavations or to indicate extensive erosion. The liners have been both PVC and woven filter fabric and have been used on top of rather than under the asbestos contamination. In Ambler, Pennsylvania, a matting layer of paper fibers in polypropylene was used on top of the clean fill to stabilize the steep slopes of the asbestos piles during the time it took for the vegetation to become established. Because there is no information that asbestos migrates downward or laterally, a bottom liner is not needed.

The depth of the cover or cap is relevant also to the ability to establish and maintain vegetation. Some asbestos wastes are highly alkaline and may be very high in magnesium. The New Hampshire and Pennsylvania sites have had pHs of 12 or more. Too little soil on top of the asbestos wastes could result in vegetation being unable to become established or dying after several seasons of growth. In addition, asbestos tailings are lacking in nutrients. If a lesser depth of soil is used for a cap, the maintenance requirements should require frequent fertilization and pH adjustments to maintain a healthy mat of vegetation.

Vegetation is recommended to stabilize the cover when adequate rainfall is available to maintain growth without irrigation. The Corps of Engineers and the Soil Conservation Service have been very helpful in selecting vegetation types, mostly grasses and ground covers such as crown vetch, that are adapted to specific climate regions and particular soil types. In areas with little natural rainfall or on steep slopes a gravel or rip rap finishing layer should be used in place of vegetation. Asphalt or concrete paving is another option for a cap, especially on sites which may be designated for industrial uses, parking lots or driveways.

6. Site Construction Work:

Some general recommendations have been made to guide responsible parties in the drafting of health and safety plans. OSHA approved respirators should be required. Work clothing need not be impermeable but should be disposable or able to be cleaned on site. Under no circumstances should workers take contaminated clothing off the site for cleaning. A number of epidemiology studies have suggested that contaminated clothing can be a significant source of exposure to families of asbestos workers.

Visible emissions are a violation of NESHAPS. They also endanger site personnel and contribute to off-site air transport. Special consideration must be given to dust control with water and a dust suppressant. If the asbestos is in large tailings piles, as it has been at several sites, the interior of the piles may or may not have a moisture content sufficient to prevent entrainment under light wind conditions during removal activities. A moisture content of 10% in addition to constant soaking during excavation to prevent moisture losses through evaporation is recommended in these situations.

Buildings on or adjacent to the site should be sealed to prevent dust infiltration. Air circulating equipment should be shut down, and intake vents should be covered with sheets of plastic. Doors, windows and foundation and roof vents should be sealed with plastic, too. After the site work has been completed, buildings should be hosed off. Equipment used on the site should be cleaned prior to installation of clean cover material. Equipment air filters should be replaced prior to use on any other site.

7. Maintenance Provisions:

Selection of cap design for either a burial pit or an above ground landfill should take into consideration the intensity of maintenance requirements and the presence of some private party, company or governmental entity to continue oversight, maintenance, and repair of the cap.

The less likely a party is to be able to continue intensive maintenance, the more important the depth of the cap and the choice of vegetation or finishing layer of rocks becomes. Consent decrees or orders should contain specific requirements for maintaining the integrity of the cap through regular fertilization, pH adjustment, mowing, reseeding of vegetation, and regular checks and repairs of erosion damage or subsidence.

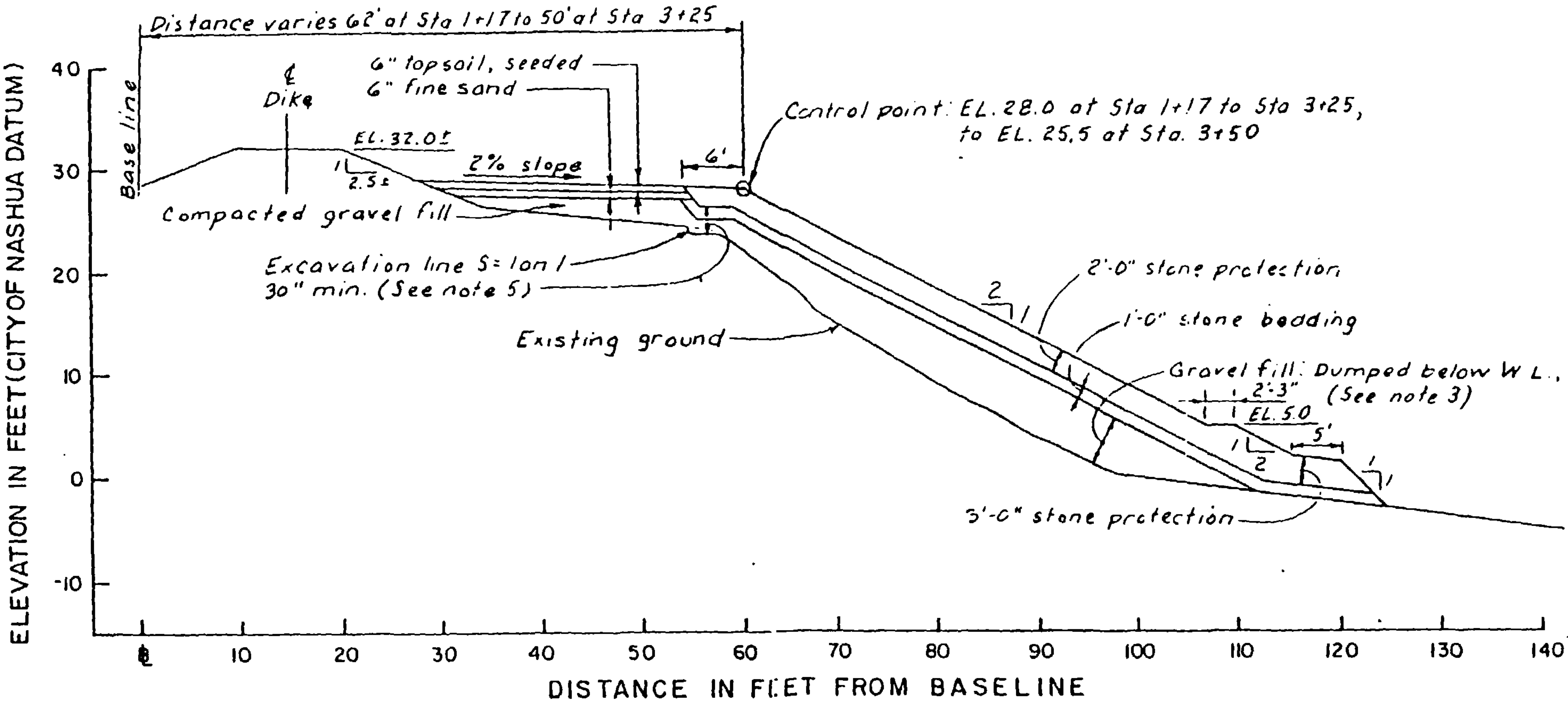
8. Future Use of Property:

Deeds or property records should be noticed with the location, size, and depth of buried asbestos wastes. Property where asbestos has been buried on-site can be used in ways limited only in so far as a cap or burial pit cover should not be disturbed. If it is necessary to disturb the cap, care should be taken to rebury asbestos contaminated soil securely; strict health and safety procedures should be observed during additional construction. U.S. EPA consent decrees have included requirements for a deed notice and advance notification and prior approval of Federal and State agencies for any activity which would disturb the cap over an asbestos waste disposal site.

9. Conclusions:

An asbestos waste disposal site shares many considerations and features of its investigation, remedial design and remedial implementation with other hazardous waste sites. However, there are a number of important differences. Asbestos is not a regulated hazardous waste under RCRA Subtitle C but is a regulated hazardous air pollutant under the Clean Air Act. Asbestos is, however, a hazardous substance under CERCLA.

The primary endangerment from asbestos results from air transport and inhalation exposure. There is little evidence that asbestos moves downward or laterally in subsurface soils. The primary remedial response for asbestos is burial. There are three major means of accomplishing this response: (1) excavation, transport, and off-site disposal; (2) excavation and on-site disposal; and (3) capping in place. A number of considerations are discussed in this paper for selection of remedies which are appropriate whether the government or a private party will perform remedial construction.



STATION 2+00

(TYPICAL STA.1+17 TO STA.3+25)

SCALE: 1" = 10'

3. W.L. refers to water level in The Merrimack River at the site during the period of construction.

4. Stone protection and stone bedding material may be placed underwater without dewatering.

5. A minimum of 30" of clean fill consisting of dumped or compacted gravel and stone bedding must be maintained between on-site contaminated soil and the bottom of the stone protection

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION
CORPS OF ENGINEERS
WALTHAM, MASS.

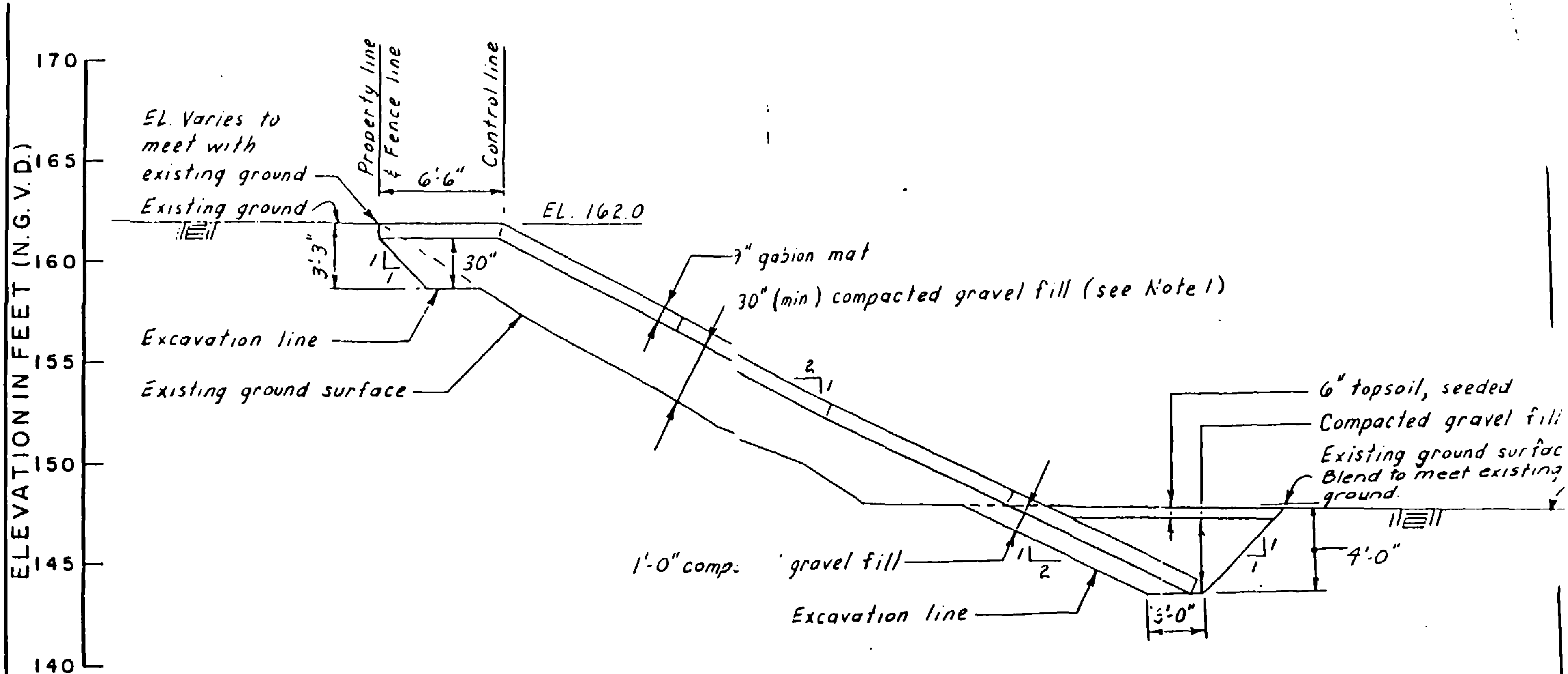
RIVERSIDE SLOPE PROTECTION AND FILL DETAILS

TYPICAL SECTION

GEOTECH. ENG. BR. SCALE: 1" = 10'

SK. NO 1 DATE: 16 Oct. 1987

S-121



SECTION A-A

SCALE: 1" = 5'

NOTES:

1. A 30" minimum cover of gravel fill shall be maintained between existing on-site contaminated material and the bottom of the 9" gabion mat.

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WALTHAM, MASS.

DES BY	STEEP INLAND SLOPE PROTECTION AND FILL DETAILS
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SK. NO. 2	DATE: 16 OCT 1987

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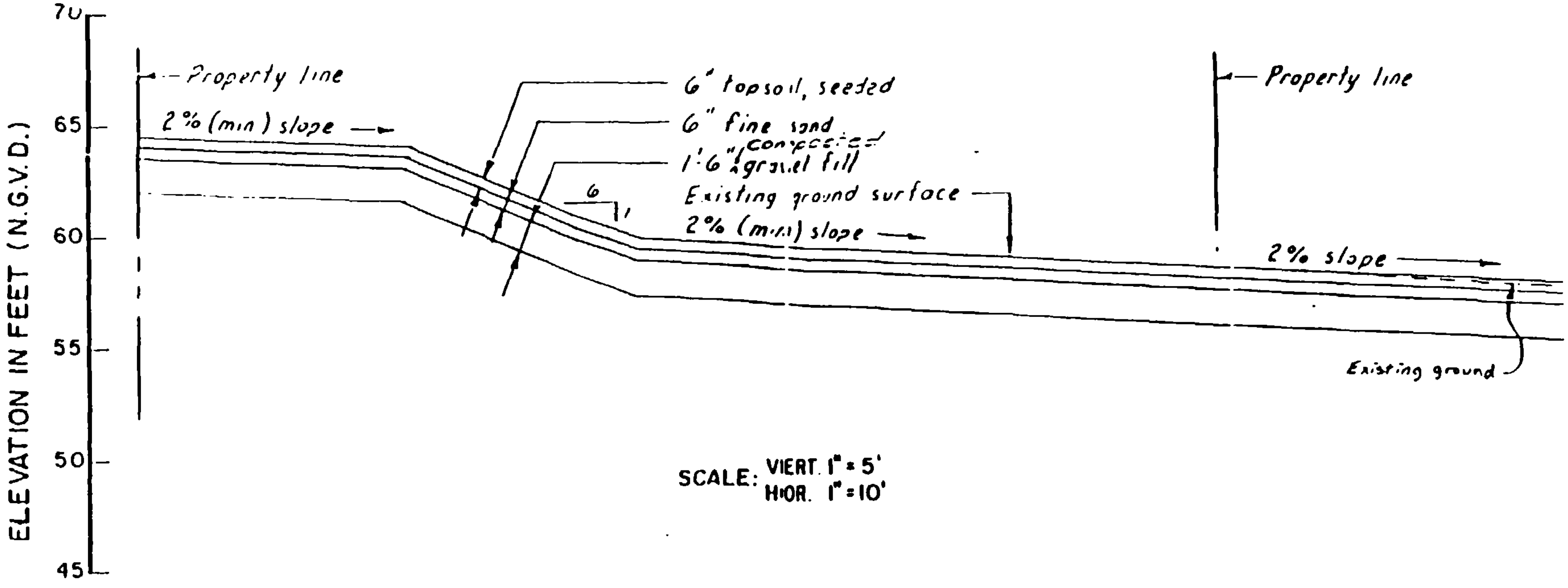

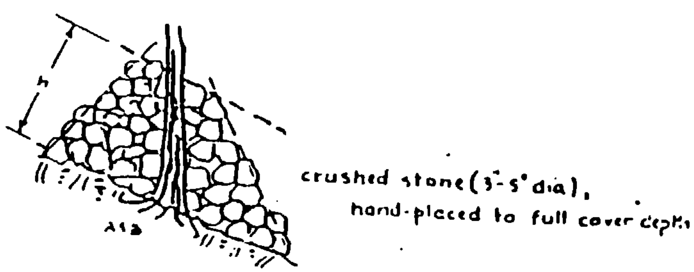

Tree well detail

DEPARTMENT OF THE ARMY NEW ENGLAND DIVISION CORPS OF ENGINEERS WALTHAM, MASS	
	GENTLY SLOPING GROUND TYPICAL SECTION
GEOTECH ENG BR SK NO 3	SCALE As shown DATE 16 OCT 1981

5-124

REFERENCE 5-3

A Guide to Hydrologic Analysis Using SCS Methods, R.H. McCuen

SECTION 4

THE SCS RAINFALL-RUNOFF RELATION

The volume of runoff (Q) depends on the volume of precipitation (P) and the volume of storage that is available for retention. The actual retention (F) is the difference between the volumes of precipitation and runoff. Furthermore, a certain volume of the precipitation at the beginning of the storm, which is called the initial abstraction (I_a), will not appear as runoff. The SCS assumed the following rainfall-runoff relation, which is shown schematically in Fig. 3:

$$\frac{F}{S} = \frac{Q}{P-I_a} \tag{2}$$

in which S = the potential maximum retention. The actual retention, when the initial abstraction is considered, is:

$$F = (P-I_a) - Q \tag{3}$$

Substituting Eq. 3 into Eq. 2 yields the following:

$$\frac{(P-I_a) - Q}{S} = \frac{Q}{P-I_a} \tag{4}$$

Rearranging Eq. 4 to solve for Q yields:

$$Q = \frac{(P-I_a)^2}{(P-I_a)+S} \tag{5}$$

The factors in Eq. 5 are best understood when placed in the form of a mass curve. Fig. 4 shows a schematic of the mass curve of Q vs. P. The volume of precipitation is separated into the initial abstraction, the retention, and the runoff.

The initial abstraction is a function of land use, treatment, and condition; interception; infiltration; depression storage; and antecedent soil moisture. An empirical analysis was performed for the development of the SCS rainfall-runoff relation, and the following formula was found to be best for estimating I_a:

$$I_a = 0.2\ S \tag{6}$$

Research performed since the development of Eq. 6 has suggested that Eq. 6 may not be correct under all circumstances; however, it remains in use until

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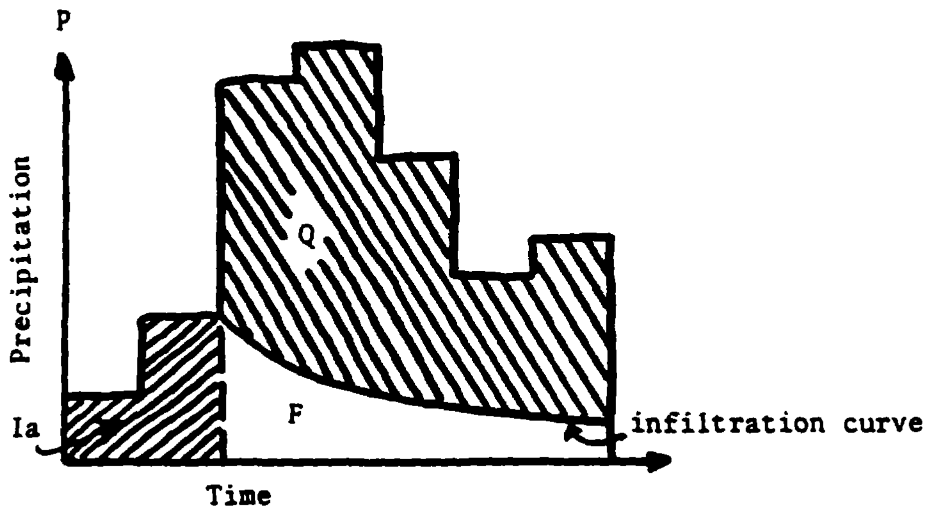

FIGURE 3. Relationship Between Precipitation, Runoff, and Retention

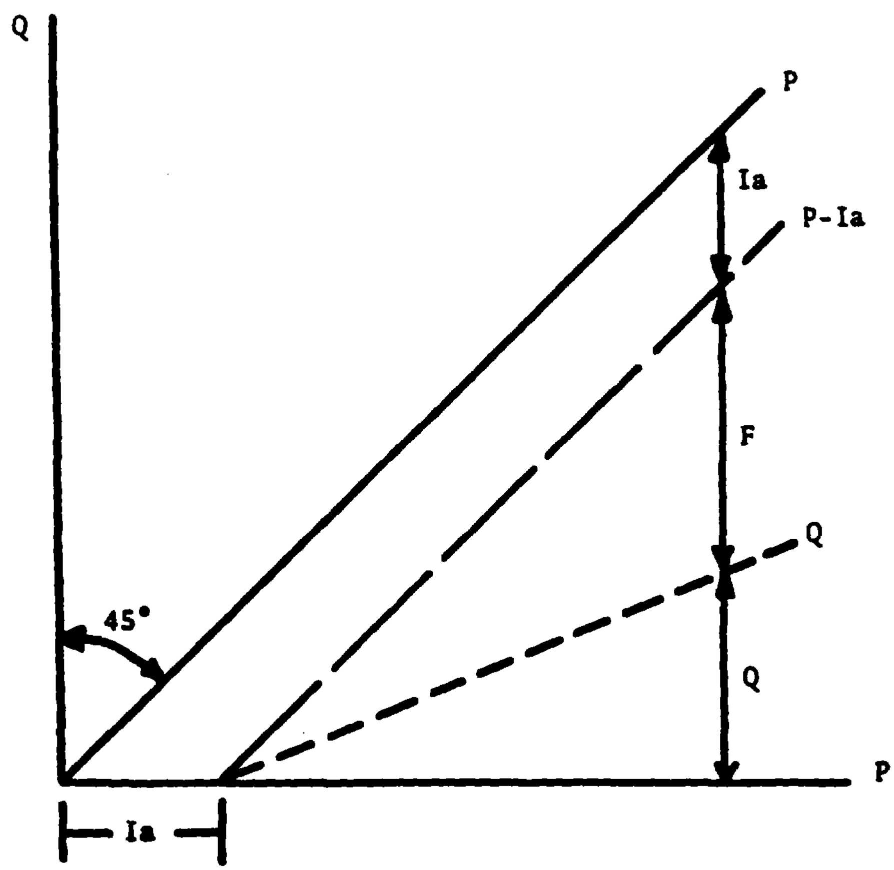

FIGURE 4. A Mass Curve Representation of the SCS Rainfall-Runoff Relationship

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a more comprehensive study is accepted. It is important to note that Eq. 6 implies that the factors affecting I_a would also affect S. Substituting Eq. 6 into Eq. 5 yields:

$$Q = \frac{(P-0.2\ S)^2}{P+0.8\ S} \tag{7}$$

While Eq. 5 has two unknowns, I_a and S, Eq. 7 has been reduced to an equation with one unknown, S. Empirical studies indicate that S can be estimated by:

$$S = \frac{1000}{CN} - 10 \tag{8}$$

in which CN = runoff curve number. Thus, the rainfall relationship of Eq. 7, which has one unknown, has been replaced with another relationship with one unknown, CN. Since S is a function of the factors that affect I_a, one should expect that the CN would also be a function of land use, antecedent soil moisture, and other factors that affect runoff and retention.

SECTION 5

CURVE NUMBER ESTIMATION

The volume and rate of runoff depends on both meteorologic and watershed characteristics, and the estimation of runoff requires an index to represent these two factors. The precipitation volume is probably the single most important meteorological characteristic in estimating the volume of runoff. The soil type, land use, and the hydrologic condition of the cover are the watershed factors that will have the most significant impact in estimating the volume of runoff. The antecedent soil moisture will also be an important determinant of runoff volume.

The SCS developed an index, which was called the runoff curve number (CN), to represent the combined hydrologic effect of soil, land use, agricultural land treatment class, hydrologic condition, and antecedent soil moisture. These factors can be assessed from soil surveys, site investigations, and land use maps; when using the SCS hydrologic methods for design the specification of the antecedent soil moisture condition is often a policy decision that suggests average watershed conditions rather than a recognition of a hydrologic condition at a particular time and place.

Soil Group Classification

SCS developed a soil classification system that consists of four groups, which are identified by the letters A, B, C, and D. Soil characteristics that are associated with each group are as follows:

Group A: deep sand, deep loess, aggregated silts

Group B: shallow loess, sandy loam

Group C: clay loams, shallow sandy loam, soils low in organic content, and soils usually high in clay

Group D: soils that swell significantly when wet, heavy plastic clays, and certain saline soils

The SCS soil group can be identified at a site using one of three ways:

1. soil characteristics
2. county soil surveys
3. minimum infiltration rate

The soil characteristics associated with each group are listed above. County soil surveys, where they are made available by Soil Conservation Districts, give a detailed description of the soils at locations within a county; these surveys are usually the best means of identifying the soil group. Soil analyses can be used to estimate the minimum infiltration rates, which can be used to classify the soil using the following values:

Group	Minimum Infiltration Rate (in/hr)
A	0.30 - 0.45
B	0.15 - 0.30
C	0.05 - 0.15
D	0 - 0.05

Cover Complex Classification

The SCS cover complex classification consists of three factors: land use, treatment or practice, and hydrologic condition. There are approximately fifteen different land uses that are identified in the tables for estimating curve number. Agricultural land uses are often subdivided by treatment or practices, such as contoured or straight row; this separation reflects the different hydrologic runoff potential that is associated with variation in land treatment. The hydrologic condition reflects the level of land management; it is separated with three classes: poor, fair, and good. Not all of the land uses are separated by treatment or condition.

Curve Number Estimation

Table 2, which is a compilation of the CN tables provided in NEH-4 and TR-55, show the CN values for the different land uses, treatment, and hydrologic condition; separate values are given for each soil group. For example, the CN for a wooded area with good cover and soil group B is 55; for soil group C, the CN would increase to 70. If the cover (on soil group B) is poor, then the CN will be 66.

Antecedent Soil Moisture Condition

Antecedent soil moisture is known to have a significant effect on both the volume and rate of runoff. Recognizing that it is a significant factor, SCS developed three antecedent soil moisture conditions, which were labeled I, II, and III. The soil condition for each is as follows:

Condition I: soils are dry but not to wilting point; satisfactory cultivation has taken place.

Condition II: average conditions

Condition III: heavy rainfall, or light rainfall and low temperatures have occurred within the last 5 days; saturated soil.

The following table gives seasonal rainfall limits for the three antecedent soil moisture conditions:

	Total 5-day Antecedent Rainfall (inches)	
AMC	Dormant Season	Growing Season
I	Less than 0.5	Less than 1.4
II	0.5 to 1.1	1.4 to 2.1
III	over 1.1	over 2.1

TABLE 2. Runoff Curve Numbers for Hydrologic Soil-Cover Complexes
(Antecedent Moisture Condition II, and $I_a = 0.2\ S$)

Land Use Description/Treatment/Hydrologic Condition			Hydrologic Soil Group A	B	C	D
Residential:[1/]						
Average lot size	Average % Impervious[2/]					
1/8 acre or less	65		77	85	90	92
1/4 acre	38		61	75	83	87
1/3 acre	30		57	72	81	86
1/2 acre	25		54	70	80	85
1 acre	20		51	68	79	84
Paved parking lots, roofs, driveways, etc.[3/]			98	98	98	98
Streets and roads:						
paved with curbs and storm sewers[3/]			98	98	98	98
gravel			76	85	89	91
dirt			72	82	87	89
Commercial and business areas (85% impervious)			89	92	94	95
Industrial districts (72% impervious)			81	88	91	93
Open Spaces, lawns, parks, golf courses, cemeteries, etc.						
good condition: grass cover on 75% or more of the area			39	61	74	80
fair condition: grass cover on 50% to 75% of the area			49	69	79	84
Fallow	Straight row	---	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured & terraced	Poor	66	74	80	82
	Contoured & terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured & terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close -seeded	Straight row	Poor	66	77	85	89
legumes[4/]	Straight row	Good	58	72	81	85
or	Contoured	Poor	64	75	83	85
rotation	Contoured	Good	55	69	78	83
meadow	Contoured & terraced	Poor	63	73	80	83
	Contoured & terraced	Good	51	67	76	80
Pasture		Poor	68	79	86	89
or range		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods or		Poor	45	66	77	83
Forest land		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		---	59	74	82	86

[1/] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[2/] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[3/] In some warmer climates of the country a curve number of 95 may be used.

[4/] Close -drilled or broadcast.

In design, the antecedent soil moisture condition is often a policy decision rather than a statement of actual soil conditions at the site during development.

The CN values obtained from Table 2 are for soil moisture condition II. If either soil condition I or III is to be used, the CN can be adjusted using the following table:

CN for Condition II	Corresponding CN for Condition I	Corresponding CN for Condition III
100	100	100
95	87	99
90	78	98
85	70	97
80	63	94
75	57	91
70	51	87
65	45	83
60	40	79
55	35	75
50	31	70
45	27	65
40	23	60
35	19	55
30	15	50
25	12	45
20	9	39
15	7	33
10	4	26
5	2	17
0	0	0

SECTION 7

ESTIMATING THE TIME-OF-CONCENTRATION

Time is an important element in hydrologic forecasting. This is reflected in the fact that most hydrologic methods include a time variable as input. The SCS methods are no different, and the time-of-concentration was selected as the best indicator of the effects of time.

The time-of-concentration (t_c) is a measure of the time for a particle of water to travel from the hydrologically most distant point in the watershed to the point where the design is to be made. Additionally, the following operational definition is sometimes used with respect to unit hydrographs: the time-of-concentration is the time from the end of rainfall excess to the point of inflection on the recession. While this operational definition will be used in developing the SCS unit hydrograph, the former definition should be understood for the computation of time-of-concentration estimates.

Hydrologists have developed numerous methods for estimating the time-of-concentration. Two methods are recommended within NEH-4 and TR-55, the lag method and the upland, or velocity, method. Almost all methods of estimating the time-of-concentration use the slope, the hydraulic length, and some measure of land use; the lag and velocity methods are no different in that they use these three factors. The hydraulic length is the distance from the hydrologically most distant point in the watershed to the point where the design is to be made.

The Lag Method

The lag method relates the time lag (L) which is defined as the time in hours from the center of mass of rainfall excess to the peak discharge, to the slope (Y) in percent, the hydraulic length (ℓ) in feet, and the maximum retention (S):

$$L = \frac{\ell^{0.8}(S+1)^{0.7}}{1900\,Y^{0.5}} \qquad (9)$$

in which S is given by Eq. 8. The time lag can also be determined from the nomograph of Fig. 7. Empirical evidence used in developing the SCS methods resulted in the following relationship between the time-of-concentration and the lag:

$$t_c = \frac{5}{3} L \qquad (10)$$

in which t_c is measured in hours.

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SECTION 8

THE GRAPHICAL METHOD

Part of Chapter 5 in TR-55 describes a method for estimating the peak discharge. The method, which is referred to as the graphical method, derives its name from a graph that relates the time-of-concentration (hours) and the unit peak discharge ($cfs/mi^2/in$). The input data requirements are minimal and include the return period in years (T), the 24-hour, T-year precipitation in inches (P), the runoff curve number (CN), the drainage area in square miles (A), the slope in percent (Y), and the hydraulic length in feet (HL or ℓ). The land use is also required if the velocity method is used to estimate the time-of-concentration.

The procedure requires the volume of runoff to be estimated from either Eq. 7 or Fig. 5 using T, P, and CN as input. The time-of-concentration can be estimated using either the lag method or the velocity method. The unit peak discharge is estimated from Fig. 9. The peak discharge equals the product of the unit peak discharge, the drainage area, and the volume of runoff.

The computation sheet of Table 3 provides a convenient means of summarizing the input data and the resulting peak discharge.

The graphical method is recommended: 1) where valley routing is <u>not</u> required, and 2) for watersheds where land use, soil, and cover are uniformly distributed throughout the watershed.

<u>Example 8-1</u>

Find the peak discharge for a 300 acre watershed having a slope of 4 percent, a CN of 70, and a hydraulic length of 6400 feet. Assume the 24-hour, 25-year precipitation is 5 inches.

The computations are summarized on the computation sheet of Table 4. The lag method yielded a lag of 0.95 hours and a time-of-concentration of 1.58 hours. The runoff volume for a precipitation of 5 inches and a CN of 70 is 2.05 inches. The unit peak discharge for a time-of-concentration of 1.58 hours is 228 $cfs/mi^2/in$. The resulting peak discharge is 219 cfs.

<u>Accuracy of the Graphical Method</u>

The graphical method was developed from the tabular method of Section 10 for the case where the travel time was zero. Thus, the graphical method is subject to the limitations of the tabular method. Specifically, the graphical method should not be used when runoff volumes are less than about 1.5 inches for curve numbers less than 60. Furthermore, the subareas should be less than 20-square miles in area and there should not be large changes in runoff curve numbers among the subareas of the watershed. These limitations are given in TR-55.

TABLE 3. COMPUTATION SHEET: TR-55 GRAPH METHOD

1. Estimate the volume of runoff

   *a. T = ________________(years): return period for design

   *b. P = ________________(inches): 24-hr, T-year precipitation volume (i.e., depth)

   *c. CN = ________________ : runoff curve number

   d. Q = ________________(inches): runoff volume obtained from Eq. 7 or Fig. 5

2. Drainage Area: A = ____________(Square miles)

3. Estimate Time-of-Concentration (use either the lag method or the velocity method)

LAG METHOD	VELOCITY METHOD
*a. CN = ______________	*a. land use ______________
*b. Slope = ____________(%)	*b. slope = ______________(%)
*c. hydraulic length = __________(ft)	*c. hydraulic length (HL) = ____________(ft)
d. L = ____________(hours): from Fig. 7	d. velocity (V) = ______________(fps): from Fig. 8
e. t_c = ____________(hours) = $\frac{5}{3}$ L	e. $t_c = \frac{HL}{3600V}$ ____________(hours)

4. Estimate unit peak discharge (q'_p) = ________________($cfs/mi^2/in$): use Fig. 9

5. Estimate peak discharge $q_p = q'_p AQ$ = ______________(cfs)

*indicates required input

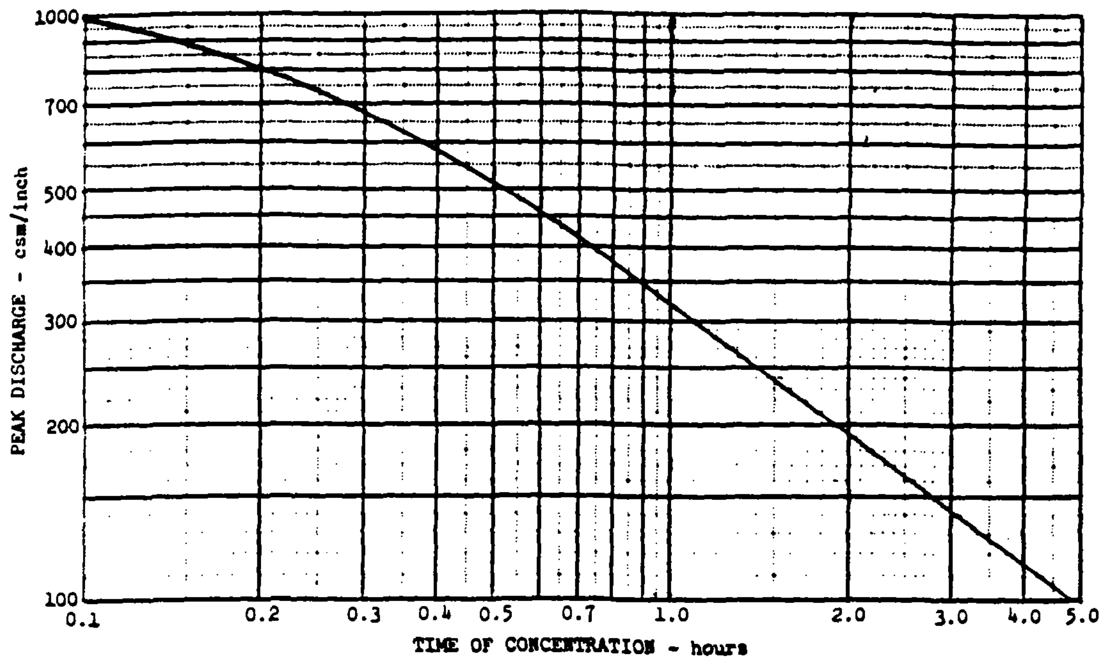


Figure 9. Peak discharge in csm per inch of runoff versus time of concentration (T_c) for 24-hour, type-II storm distribution.

To further define limitations on the graphical method the results of numerous TR-20 runs were compared with estimates of peak discharge made with the graphical method. The runs were made for ranges of the time of concentration (hours), the precipitation volume (inches), and the curve number of 0.5 to 5.0 hours, 1.0 to 10.0 inches, and 50 to 95 curve number units, respectively. The results indicate that the graphical method is a valid approximation of TR-20 as long as the initial abstraction is less than 25 percent of the total 24-hour rainfall; this constraint is easily assessed using the following tabular representation of the constraint, which relates the curve number (CN) and the minimum precipitation:

CN	minimum precipitation
50	8.00 inches
60	5.33
70	3.42
80	2.00
90	0.88
95	0.42

SECTION 6

6.0 SANITARY SEWER AND STORM DRAIN DESIGN AND RELOCATION

The existing on-facility sanitary sewer bridge that crosses the Neponset River and the sanitary sewer line that parallels the river must be abandoned and the corresponding flow rerouted. The existing storm drains that currently discharge into the Neponset River on the west side of South Street must also be relocated. The sewers and storm drain must be abandoned since they are located within the proposed area of containment (see Section 5.0).

Canonie contracted GCG Associates, Inc. (GCG) to investigate the most practical relocation route for the sanitary sewer. The final report from GCG will be provided in an independent document, subject to approval by the Town of Walpole.

There are two storm drains that currently discharge into the Neponset River. One storm drain collects the stormwater from the north side of the river. The second storm drain collects water on the south side of the river. The existing storm drains will be rerouted to discharge into the Neponset River on the east side of the South Street bridge. The discharge points will be located approximately 15 feet east of the South Street bridge. This will eliminate the possibility of the storm drain discharge interfering with the installation of the steel dam.

Currently, stormwater that is collected from the vacant building is discharged into the area designated as Lagoon Number Two. This storm drain will be rerouted to the north around Lagoon Number Two. This is necessary since the area in the vicinity of Lagoon Number Two will be part of the asbestos containment area and will be capped. The storm drain will be rerouted around the containment area and will discharge into the mill tail race.

Canonie

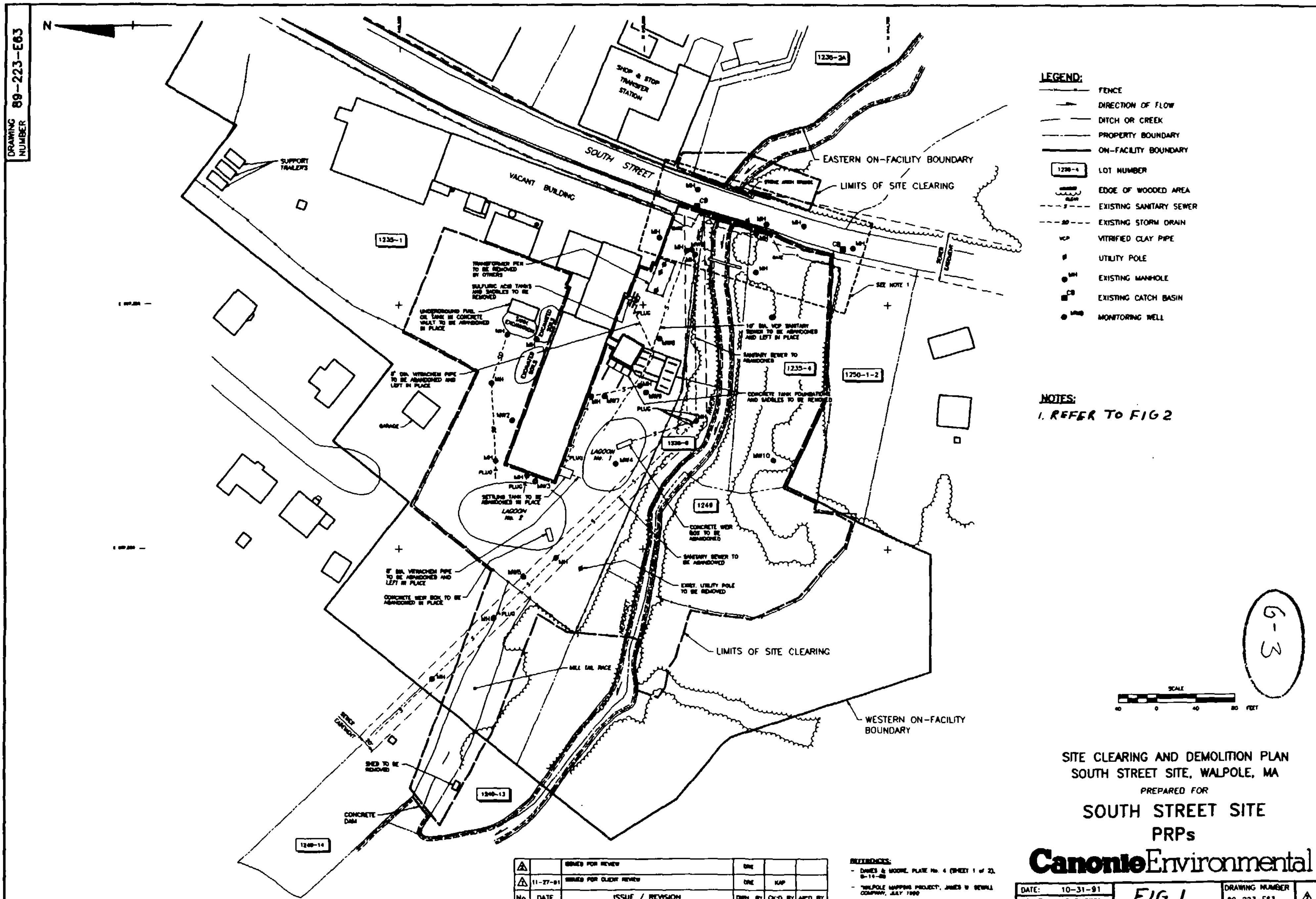
By KAP Date 01-25-92 Subject South St, Walpole, Mass. Sheet No. 1 of 1
Chkd. By CC Date 1-28-92 Rerouting Sanitary Sewer Proj. No. 89-223-05
1/4" X 1/4"

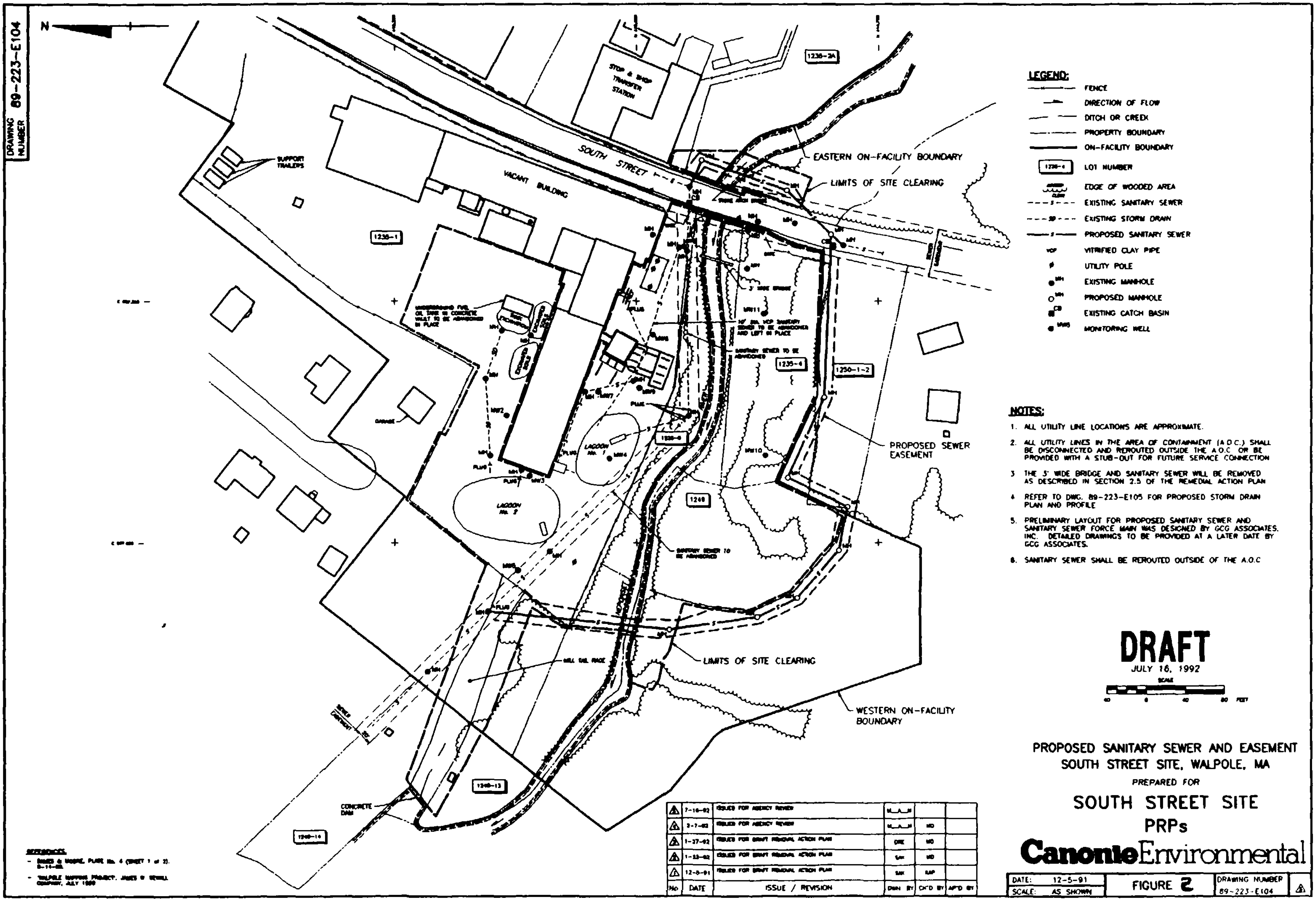
6.1 Rerouting of Existing sanitary sewer

The existing sanitary sewer line crosses the Neponset River on the west side of South Street and runs parellel to the river on the north bank of the river. This portion of the sewer is in the proposed area of containment (AOC) and must be rerouted. The relocation of the sewer will avoid the future possibility of having to excavate the portion of the sewer in the AOC for repairs or maintenance. The relocation is also necessary since the existing sewer bridge that crosses the Neponset River must be removed to accomedate the proposed pipe arch culvert (see section 2).

The existing location of the sanitary sewer is shown in Figure 1. Canonie has contracted GCG Associates, Inc. to investigate the most appropriate means of rerouting the sanitary sewer. The results of their preliminary sanitary sewer relocation design is shown in Figure 2.

GCG has not presented Canonie with the proposed invert elevations or slopes. This information will be included in the sanitary sewer rerouting design that will be presented to the town of Walpole for final approval. Canonie will ensure that any ammendments to the design will not adversly effect the integrity of the AOC


DRAWING NUMBER 89-223-E63
N
SOUTH STREET
VACANT BUILDING
SHOP & STOP TRANSFER STATION
SUPPORT TRAILERS
GARAGE
EASTERN ON-FACILITY BOUNDARY
LIMITS OF SITE CLEARING
WESTERN ON-FACILITY BOUNDARY
LAGOON No. 1
LAGOON No. 2
MILL TAIL RACE
CONCRETE DAM
SEE NOTE 1
1235-1
1235-4
1235-6
1236-2A
1250-1-2
1249
1249-13
1249-14
LEGEND:
FENCE
DIRECTION OF FLOW
DITCH OR CREEK
PROPERTY BOUNDARY
ON-FACILITY BOUNDARY
LOT NUMBER
EDGE OF WOODED AREA
EXISTING SANITARY SEWER
EXISTING STORM DRAIN
VITRIFIED CLAY PIPE
UTILITY POLE
EXISTING MANHOLE
EXISTING CATCH BASIN
MONITORING WELL
NOTES:
1. REFER TO FIG 2
G-3
SITE CLEARING AND DEMOLITION PLAN
SOUTH STREET SITE, WALPOLE, MA
PREPARED FOR
SOUTH STREET SITE
PRPs
Canonie Environmental
DATE: 10-31-91
SCALE: AS SHOWN
FIG 1
DRAWING NUMBER 89-223-E63
No. DATE ISSUE / REVISION DWN BY CK'D BY APP'D BY
11-27-91

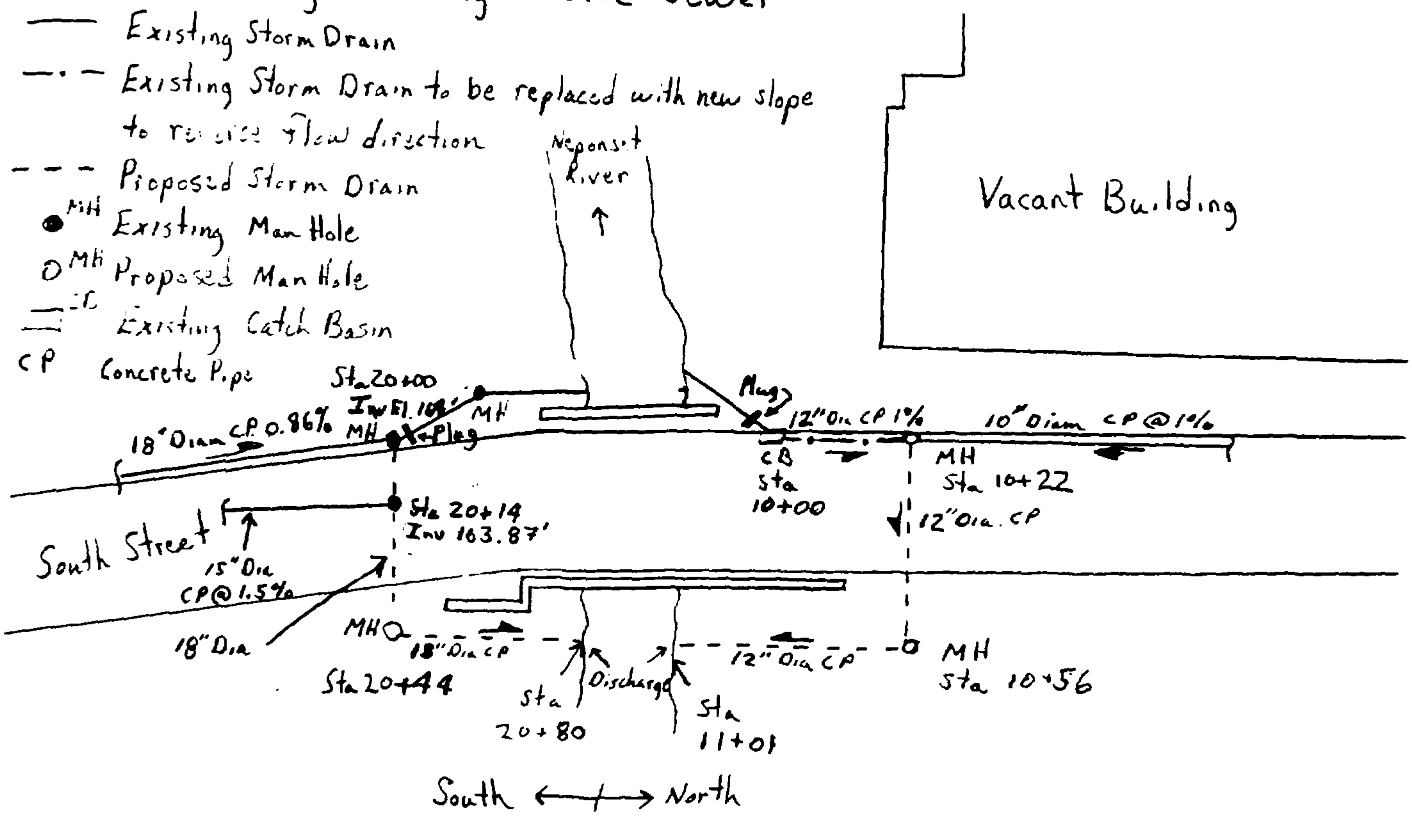


DRAWING NUMBER 89-223-E104
N
SOUTH STREET
VACANT BUILDING
STOP & SHOP TRANSFER STATION
SUPPORT TRAILERS
EASTERN ON-FACILITY BOUNDARY
LIMITS OF SITE CLEARING
PROPOSED SEWER EASEMENT
LIMITS OF SITE CLEARING
WESTERN ON-FACILITY BOUNDARY
LAGOON NO. 1
LAGOON NO. 2
CONCRETE DAM
1235-1
1235-2A
1235-4
1250-1-2
1249
1249-13
1249-14
LEGEND:
FENCE
DIRECTION OF FLOW
DITCH OR CREEK
PROPERTY BOUNDARY
ON-FACILITY BOUNDARY
LOT NUMBER
EDGE OF WOODED AREA
EXISTING SANITARY SEWER
EXISTING STORM DRAIN
PROPOSED SANITARY SEWER
VITRIFIED CLAY PIPE
UTILITY POLE
EXISTING MANHOLE
PROPOSED MANHOLE
EXISTING CATCH BASIN
MONITORING WELL
NOTES:
1. ALL UTILITY LINE LOCATIONS ARE APPROXIMATE.
2. ALL UTILITY LINES IN THE AREA OF CONTAINMENT (A.O.C.) SHALL BE DISCONNECTED AND REROUTED OUTSIDE THE A.O.C. OR BE PROVIDED WITH A STUB-OUT FOR FUTURE SERVICE CONNECTION
3. THE 3' WIDE BRIDGE AND SANITARY SEWER WILL BE REMOVED AS DESCRIBED IN SECTION 2.5 OF THE REMEDIAL ACTION PLAN
4. REFER TO DWG. 89-223-E105 FOR PROPOSED STORM DRAIN PLAN AND PROFILE
5. PRELIMINARY LAYOUT FOR PROPOSED SANITARY SEWER AND SANITARY SEWER FORCE MAIN WAS DESIGNED BY GCG ASSOCIATES, INC. DETAILED DRAWINGS TO BE PROVIDED AT A LATER DATE BY GCG ASSOCIATES.
6. SANITARY SEWER SHALL BE REROUTED OUTSIDE OF THE A.O.C
DRAFT
JULY 16, 1992
PROPOSED SANITARY SEWER AND EASEMENT
SOUTH STREET SITE, WALPOLE, MA
PREPARED FOR
SOUTH STREET SITE
PRPs
Canonie Environmental
DATE: 12-5-91
SCALE: AS SHOWN
FIGURE 2
DRAWING NUMBER 89-223-E104
7-16-92 ISSUED FOR AGENCY REVIEW
1-27-92 ISSUED FOR DRAFT REMOVAL ACTION PLAN
12-6-91 ISSUED FOR DRAFT REMOVAL ACTION PLAN
No DATE ISSUE / REVISION DWN BY CK'D BY AP'D BY
REFERENCES:

Canonie

By KAP Date 10/30/91 Subject South St., Walpole Mass Sheet No. 1 of 3
Chkd. By CC Date 11-12-91 Storm Drain Relocation Proj. No. 89-223-05
1/4" X 1/4"

6.2 Rerouting Existing Storm Sewer



All Proposed Storm drain concrete piping will be of the same diameter of the existing storm drain concrete piping it is replacing. Canonie assigned stations 10+00 to 11+01 for the north storm drain and stations 20+00 to 20+80 for the south storm drain

⇒ South Storm Drain connecting sta 20+00 to Sta 20+80 will be 18" Diameter concrete pipe.

⇒ North Storm Drain Connecting sta 10+00 to sta 11+01 will be 12" Diameter concrete pipe.

All proposed Storm drain piping will be placed at a slope equal or greater than the existing storm drain pipe slope.

⇒ South Storm drain connecting Sta 20+00 to 20+14 will have a 1.2% slope
⇒ South Storm drain connecting sta 20+14 to 20+44 will have a 1.5% Slope
⇒ South Storm drain connecting sta 20+44 to sta 20+80 will have a 20.5 Slope
⇒ North Storm drain connecting sta 10+00 to Sta 10+22 will have a 1% Slope
⇒ North Storm drain connecting sta 10+22 to Sta 10+56 will have a 1.6% Slope
⇒ North Storm drain connecting sta 10+56 to Sta 11+01 will have a 9.0% slope

Canonie

By KAP Date 10/30/91 Subject South St, Weirgate Mass Sheet No. 2 of 3
Chkd. By CC Date 11-12-91 Storm Drain Relocation Proj. No. 39-223-05
1/4" X 1/4"

To achieve the desired slopes the storm drains will have the invert elevations shown in the figure.

The existing section of pipe connecting sta 20+00 to 20+19 will be re-sloped to change the direction of flow.

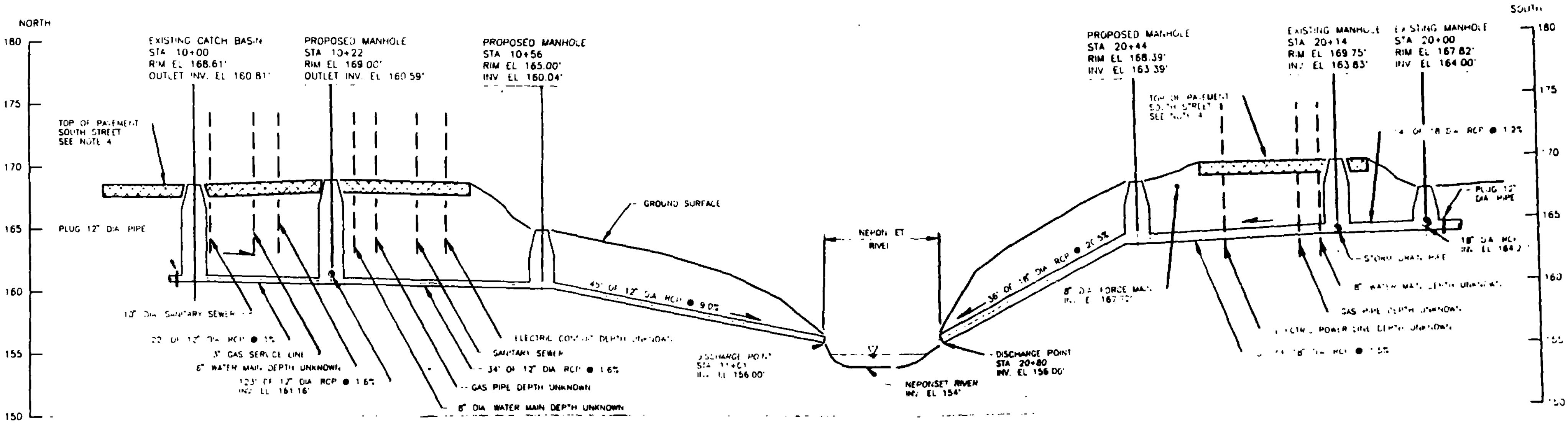
The existing section of pipe connecting sta 10+00 to 10+22 will be re-sloped to change the direction of flow.

6-7

No. 3 of 3

o. 89-223-05

1/4"



STORM DRAIN PROFILE

VIEW LOOKING EASTERLY

HORIZONTAL SCALE 10 0 10 20 FEET

VERTICAL SCALE 5 0 5 10 FEET

Canonie

By KAP Date 10/30/91

Chkd. By CC Date 11-12-91

Canonie

By KAP Date 1/7/92 Subject South Street, Walpole, MA Sheet No. 1 of 5

Chkd. By CC Date 1-9-92 Drainage N-W of Building Proj. No. 89-223-05

1/4" X 1/4"

6.3 Storm Drain North West of Vacant Building ...

Currently two drainage pipes discharge storm water into lagoon number 2 as shown on the figure on the following page. The storm drainage will be rerouted to the north around the present lagoon number 1 area. This will be necessary since the lagoon 1 area will be used for containment of asbestos material and will be included in the final site cap area.

The rerouted path will ensure that any future work on the drainage pipes will not interfere with the final site cap.

The diameter and slope of the rerouted section of drainage pipe will be determined by the maximum flow that is possible in the 10" diameter clay pipe and the flow through the 8" diameter clay pipe (due to drainage from the buildings roof).

The max flow through the 10" diameter pipe will be calculated using Mannings Equation

$$Q_{max_1} = \frac{0.463\, d^{8/3} S^{1/2}}{n_{clay}}$$

$d = 10''/12''$ Length of pipe = 80' Ref 6-1 p 6-14

$$S = \frac{155.6' - 152.5'}{80} = 0.039$$

$$n_{clay} = 0.014$$

$$Q_{max_1} = \frac{0.463\,(10''/12'')^{8/3}(0.039)^{1/2}}{0.014} = 4.02\text{ cfs} \approx 4\text{ cfs}$$

The max flow through the 8" diameter pipe will be calculated from the roof area (A) and a rain fall intensity that corresponds to the 100 yr one hour rainfall (I_{100yr})

$$Q_{max_2} = A \cdot I_{100yr} \qquad A = (53')(205') = 10,865\text{ ft}^2 \qquad I_{100yr} = 3.5''/\text{hr}$$

Ref 6-2 p18-01

$$= (10,865\text{ ft}^2)\left(3.5\,\frac{\text{in}}{\text{hr}} \cdot \frac{1\text{ ft}}{12\text{ in}} \cdot \frac{1\text{ hr}}{3600\text{ sec}}\right)$$

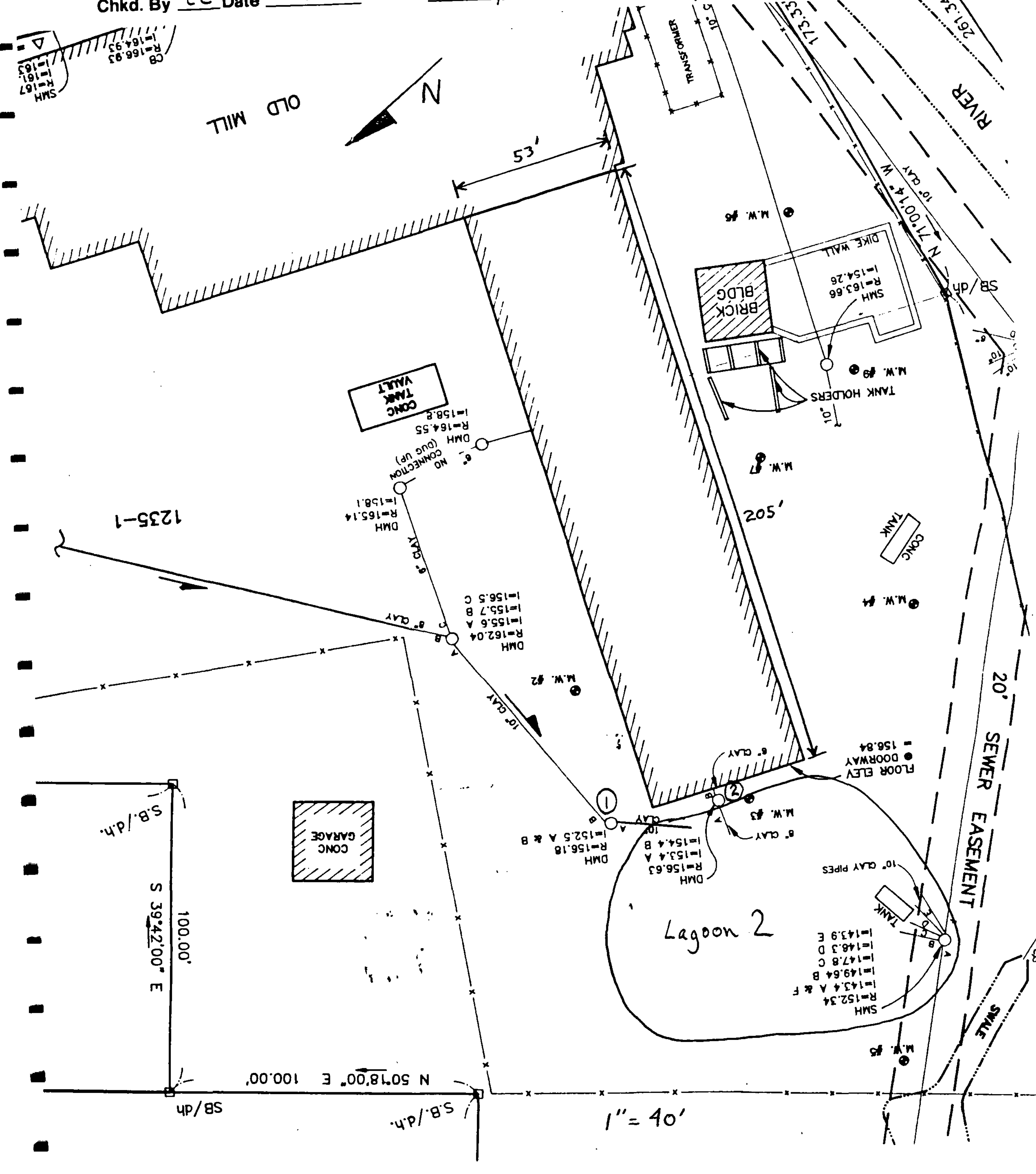
$$Q_{max_2} = 0.88\text{ cfs} \approx 1\text{ cfs}$$

6-9

Canonie

By KAP Date 1/7/92 Subject South Street, Walpole, MA Sheet No. 2 of 5

Chkd. By CC Date 1-9-92 Drainage N-W of Building Proj. No. 89-223-05



Canonie

By KAP Date 1/7/92 Subject South Street, Walpole, MA Sheet No. 3 of 5
Chkd. By CC Date 1-9-92 Drainage N-W of Building Proj. No. 89-223-05
1/4" X 1/4"

The combined total flow:

$$Q_T = Q_{max_1} + Q_{max_2} = 4\,cfs + 1\,cfs = 5\,cfs$$

Use $Q_T = 6$ cfs for design purposes

The rerouted length from ②→①→③ will be approximatly 270 ft. The drainage pipe will be rerouted to drain into the mill tail race which has a ground elevation of 146 ft. The invert elevation of the existing manhole marked ① is 152.5 ft. This limits the maximum rerouted slope to

$$S_{max} = \frac{152.5' - 146'}{270'} = 2.4\,\%$$

Trial #1

Determine the maximum flow rate in the rerouted drainage pipe if $D = 12''$ and $S = 2.2\%$

$$Q = \frac{0.463\, d^{8/3} S^{1/2}}{n_{concrete}} \qquad n_{concrete} = 0.015 \qquad \text{Ref 6-1 p 6-15}$$

$$Q = \frac{0.463\,(1^{ft})^{8/3}(0.022)^{1/2}}{0.015} = 4.6 \text{ cfs} < Q_{Total} \qquad N.G$$

Trial #2

Since the slope used in trial 1 was close to the maximum slope the diameter must be increased to achieve the desired flow rate. Try $D = 15''$ and $S = 1.2\%$

$$Q = \frac{0.463\,(15/12)^{8/3}(0.012)^{1/2}}{0.015} = 6.1 > Q_{Total} \qquad OK$$

6-11

Canonie

By KAP Date 1/7/92 Subject South St Walpole MA Sheet No. 4 of 5

Chkd. By CC Date 1-9-92 Drainage N-W of Building Proj. No. 99-223-05

1/4" X 1/4"

∴ The maximum flow rate can be achieved with a 15" diameter concrete pipe with a slope equal or greater than 1.2 %

The figure on the following page indicates the pipe size, slopes, lengths, manhole invert elevations, and exact location of the rerouted drainage pipe.

The controlling design parameters used are

$D = 15''$

$S = 1.8\% > 1.2\%$ ok

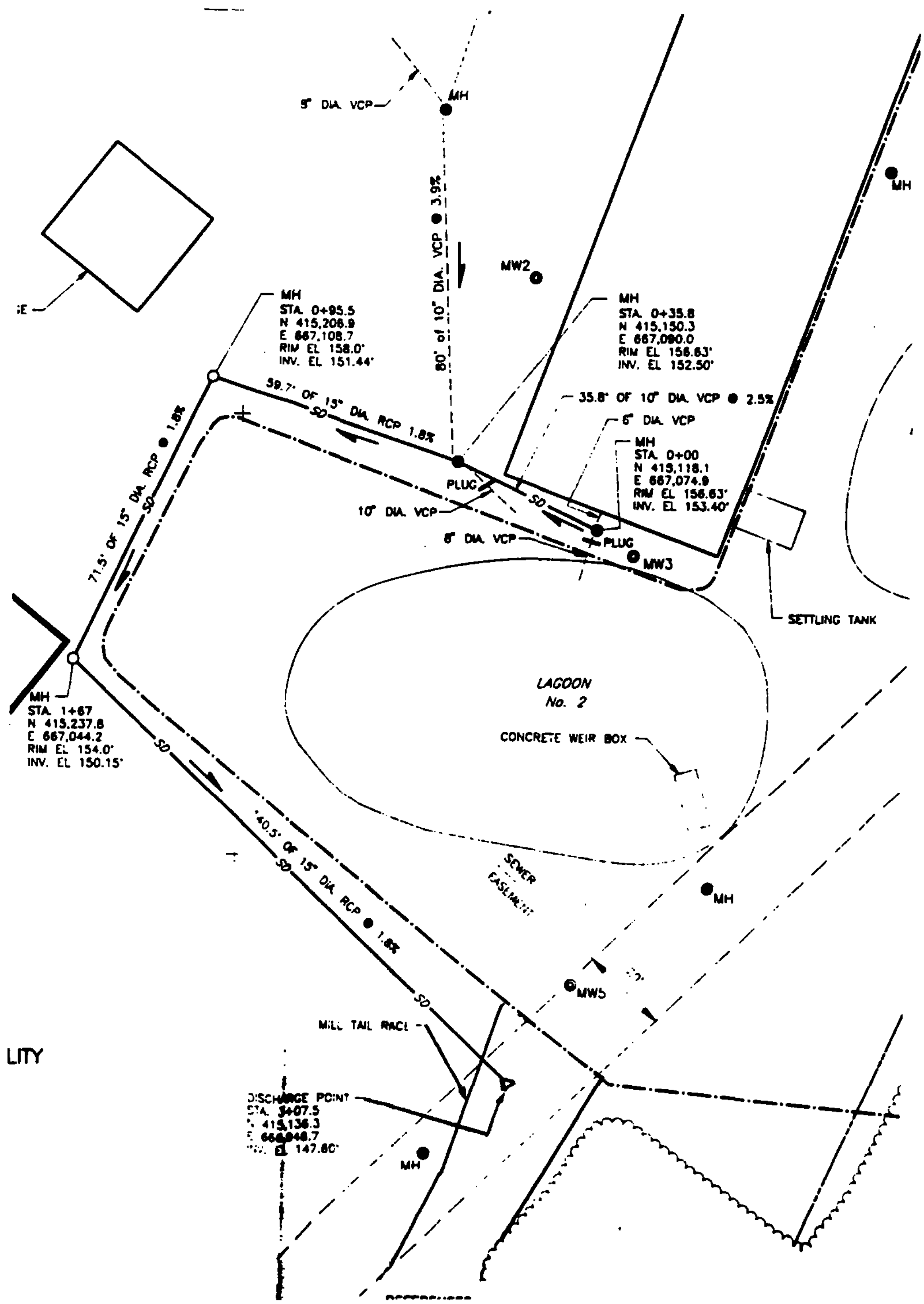
Canonie

(6-12)

By KAP Date 1/7/92 Subject South St Walpole Mass Sheet No. 5 of 5

Chkd. By CC Date 1-9-92 Drainage N-W of Building Proj. No. 89-223-05

1/4" X 1/4"



SECTION 6 REFERENCES

6-13

REFERENCE 6-1

King. H.W. and E.F. Brater, 1963, Handbook of Hydraulics, McGraw-Hill Book Co., 5th Edition.

6-14

from which it can be seen that the Darcy-Weisbach formula [Eq. (6-19)] is a rearrangement of the Chezy formula, the roughness coefficients being related as follows:

$$f = \frac{8g}{C^2} \tag{6-24}$$

From the previous discussion concerning Fig. 6-4 it may be concluded that the *Chezy formula would give excellent results* for flow in rough conduits at large Reynolds numbers, where the exponent of V is approximately 2. When later investigators found that this formula did not adhere to test results for smooth pipe with low velocities, other empirical formulas were devised to satisfy each particular group of tests. Only in recent years has it become generally recognized that all such tests can be unified by means of Reynolds number.

A dimensional investigation of the Chezy formula will show that the left term has the dimension L/T whereas the right side is simply $L^{1/2}$. The expression is therefore not dimensionally homogeneous and can be used in the above form only in the foot-pound-second system.

The Manning[1] *Formula.* Manning concluded that C in Chezy should vary with $r^{1/6}$ as follows:

$$C = \frac{1.486}{n} r^{1/6} \tag{6-25}$$

where n is the roughness coefficient. Substitution of Eq. (6-25) in the Chezy formula [Eq. (6-22)] yields the following equation, which is in the form usually referred to as the Manning formula:

$$V = \frac{1.486}{n} r^{2/3} s^{1/2} \tag{6-26}$$

It is obvious from the variation in values of n derived from various tests that the constant 1.486 could be written 1.49 or 1.5 with no sacrifice in accuracy. It may be noted that the exact value of the constant in the Manning equation was chosen because it is the cube root of the number of feet in one meter (3.281) and is therefore the factor required to convert to the foot-pound-second system from the metric form of the Manning formula.

[1] Robert Manning, Flow of Water in Open Channels and Pipes, *Trans. Inst. Civil Engrs. (Ireland)*, vol. 20, 1890.

6-15

For round pipes flowing full, $d/4$ may be substituted for r and the Manning formula becomes

$$V = \frac{0.590}{n} d^{2/3} s^{1/2} \qquad \text{(6-26a)}$$

A comparison of the Manning equation [Eq. (6-26)] with the Darcy-Weisbach equation [Eq. (6-19)] reveals the following relationship between n and f:

$$f = \frac{185n^2}{d^{1/3}} \qquad \text{(6-27)}$$

It is convenient to express the Manning formula, when used for pipes, in one of the following forms:

$$V = \frac{0.590}{n} d^{2/3} s^{1/2} \qquad \text{(6-26a)}$$

$$Q = \frac{0.463}{n} d^{8/3} s^{1/2} \qquad \text{(6-26b)}$$

$$h = 2.87n^2 \frac{lV^2}{d^{4/3}} \qquad \text{(6-26c)}$$

$$h = 4.66n^2 \frac{lQ^2}{d^{16/3}} \qquad \text{(6-26d)}$$

$$d = \left(\frac{2.159Qn}{s^{1/2}}\right)^{3/8} \qquad \text{(6-26e)}$$

$$d_i = \left(\frac{1,630Qn}{s^{1/2}}\right)^{3/8} \qquad \text{(6-26f)}$$

The solution of the Manning formula expressed in terms of the diameter in inches d_i, s, and the discharge Q [formula (6-26f)] is given in Table 6-2. It is assumed that the length of pipe is known and that the value of n has been selected. The use of this table in solving three types of problems is described:

1. *To determine d_i, s and Q being given:* Table 6-2 gives directly the diameters of pipes in inches.

2. *To determine s, d_i and Q being given:* On the page having the proper n, in the same row with the given discharge, find the diameter nearest to the given diameter. At the top of the column will be the approximate values of s. Closer values can be obtained by interpolation.

3. *To determine Q, s and d_i being given:* On the page having the proper n, in the same column with the given s, find the diameter nearest to the given diameter. At the left end

of this row will be the approximate discharge. Closer values can be obtained by interpolation.

Inasmuch as hydraulic formulas are never accurate enough to justify very close adherence to them, great exactness in interpolation is not desirable.

The values of n contained in the following table are recommended. Practically all the experimental results that have been published lie between the extremes of these values.

Values of n for Pipes, to Be Used with the Manning Formula

Kind of pipe	Variation		Use in designing	
	From	To	From	To
Clean uncoated cast-iron pipe	0.011	0.015	0.013	0.015
Clean coated cast-iron pipe	0.010	0.014	0.012	0.014
Dirty or tuberculated cast-iron pipe	*0.015*	*0.035*		
Riveted steel pipe	0.013	0.017	0.015	0.017
Lock-bar and welded pipe	0.010	0.013	0.012	0.013
Galvanized-iron pipe	0.012	0.017	0.015	0.017
Brass and glass pipe	0.009	0.013		
Wood-stave pipe	0.010	0.014		
Wood-stave pipe, small diameter	.....	.....	0.011	0.012
Wood-stave pipe, large diameter	.....	.....	0.012	0.013
Concrete pipe	0.010	0.017		
Concrete pipe with rough joints	.....	.....	0.016	0.017
Concrete pipe, "dry mix," rough forms	.....	.....	0.015	0.016
Concrete pipe, "wet mix," steel forms	.....	.....	0.012	0.014
Concrete pipe, very smooth	.....	.....	0.011	0.012
Vitrified sewer pipe	0.010	0.017	0.013	0.015
Common clay drainage tile	0.011	0.017	0.012	0.014
Corrugated metal	0.023	0.025		
Rock, unlined	0.035	0.041		
Enameled steel	0.009	0.010		

The following tables will be found helpful in various solutions of the Manning formula: square roots of decimal numbers, Table 7-17; two-thirds powers of numbers, Table 7-18; three-eighths powers of numbers, Table 7-20; areas of circles by hundredths, Table 6-4; diameters of circles in feet and inches with areas in square feet, Table 6-3. Table 6-5 gives circumferences of circles.

REFERENCE 6-2

Seelye, Elwyn E., 1968, Data Book for Civil Engineers Design, 3rd Edition, John Wiley and Sons, Inc.

DRAINAGE — RUNOFF — I

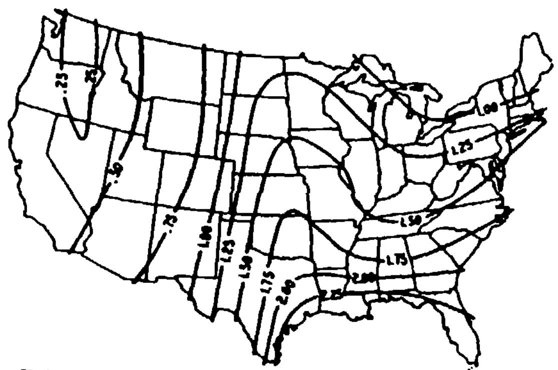


FIG. A. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 2 YEARS.

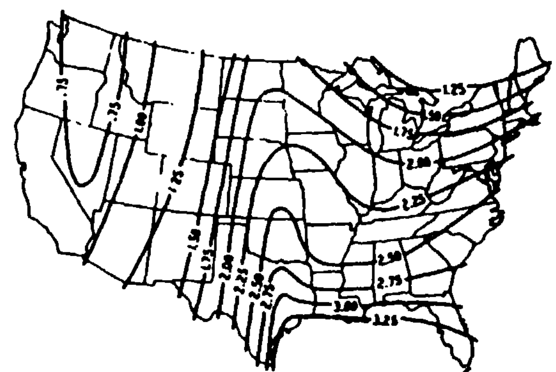


FIG. B. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 10 YEARS.

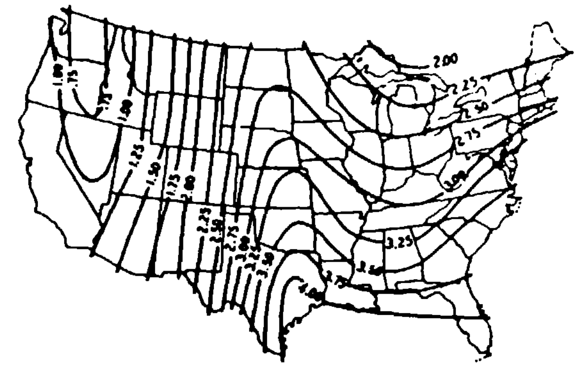


FIG. C. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 50 YEARS.

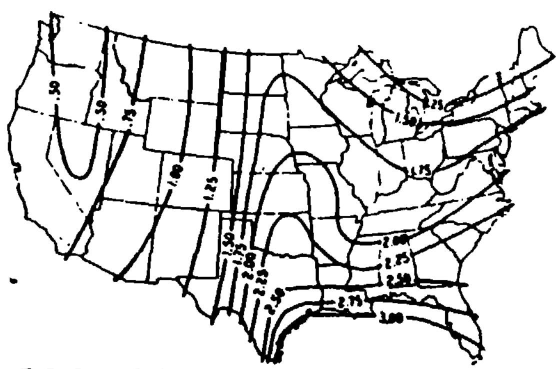


FIG. D. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 5 YEARS.

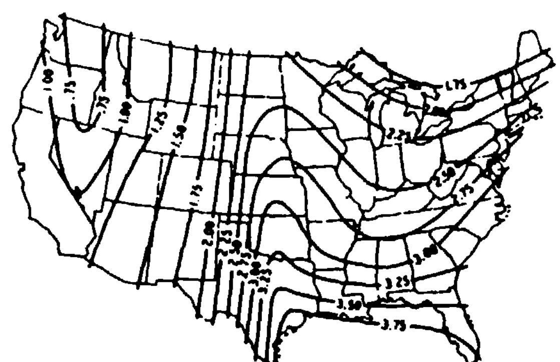


FIG. E. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 25 YEARS.

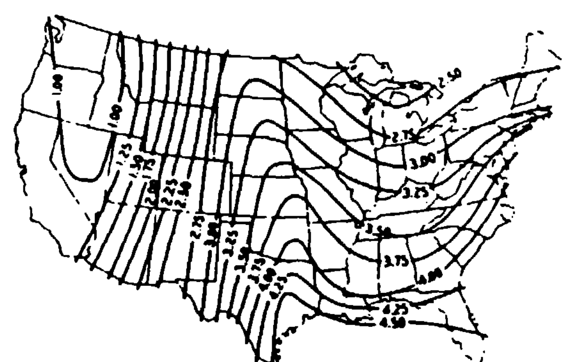


FIG. F. – ONE-HOUR RAINFALL, IN INCHES, TO BE EXPECTED ONCE IN 100 YEARS.

COMPUTATION OF i IN RATIONAL FORMULA.

EXAMPLE: Assume expectancy period = 5 years, see Fig. D, assume locality, find 1 hour intensity = 1.75 in. per hour.