FINAL DESIGN COMPUTATIONS

FOR

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FCD NO. 90-43
SCI NO. 10924

JULY, 1991

REGISTERED PROFESSIONAL ENGINEER (Civil)
CERTIFICATE NO. 23120
BENNETT D. REISCHAUER
Date Signed 7-19-91
ARIZONA, U.S.A.

Final Design Computations

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Computed by Bennett R. Date 2-8-91
Checked by ____ Date ____
Reviewed by BDR Date 5-10-91
Approved by ____ Date ____

Job No. ____ Page No. 2
Subject Cross Bends Detention Basin - 100 year flood elevation
Sheet No. 1 of 4

Set floor elevation of pump station to be above 100-yr flood elevation.

Pump Station will be located adjacent to Eastern Canal. Existing ground elevation at P.S. site is 1269.0 ±.

Determine 100 year flood elevation at this location from the FIS study done by Franzoy-Corey. Refer to attached pages of FIS on Table 1 for flood elevation upstream of railroad (which is our project site).

100 yr flood elevation = 1271.4

Allow 1.6 feet for freeboard above the 100-year flood elevation

Set P.S. floor slab

100 yr flood	=	1271.4
freeboard	= +	1.6
floor elev	=	1273.0

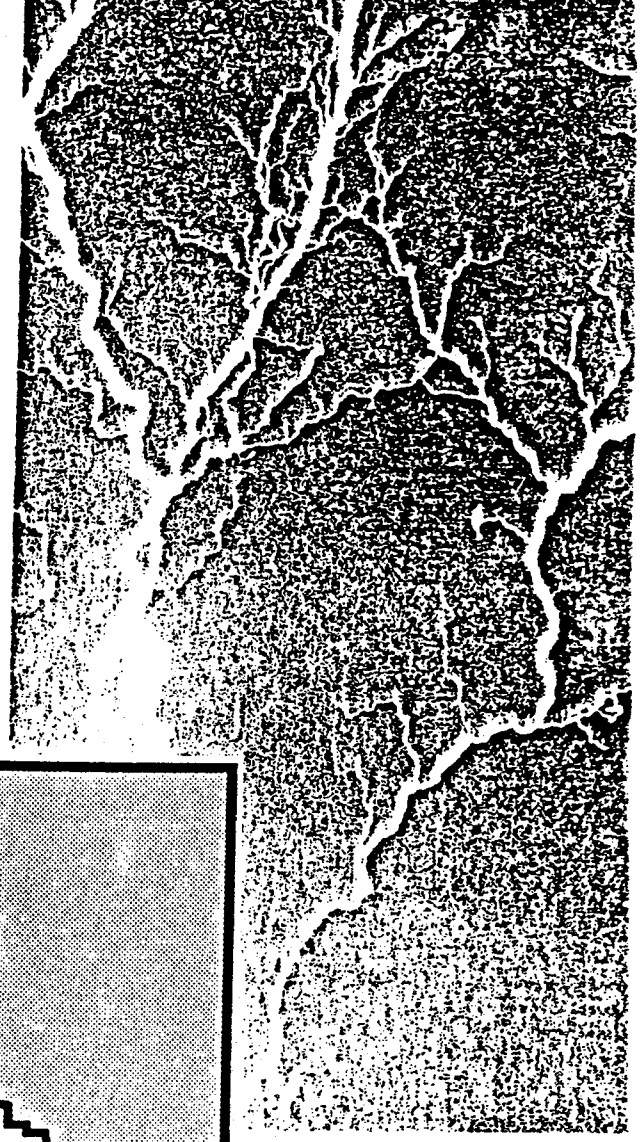
Pump Station Floor Elevation = 1273.0

Grade area around P.S. so that top of slab does not stick out of ground more than about 8".

The 100-yr flood Elevation with the basin in place will be 1271.1 per Lonnie Frost on 5-9-91. Therefore pump station floor elevation is safe. 5-10-91 by BDR

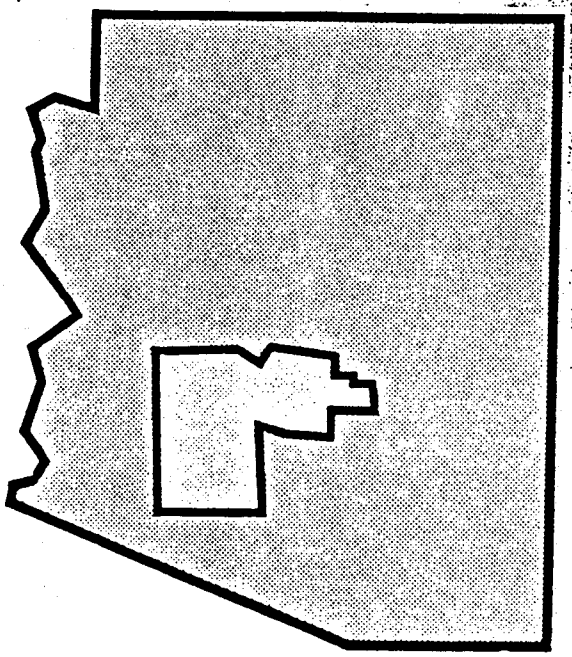
2 of 4

FLOOD INSURANCE STUDY



GILBERT-CHANDLER AREA
Maricopa County, Arizona

Prepared for the
Flood Control District
of
Maricopa County



DRAFT • October 1989
REVISED • July 1990

Registered Professional Engineer (Civil)
CERTIFICATE NO. 15372
RUSSELL R. MIRACLE
Date Signed 7/26/90
ARIZONA, U.S.A.

Franzoy•Corey
ENGINEERING COMPANY

(CMCN) completed in 1986 by Greiner Engineering. The basis for the CMCN are the benchmarks "RYAN" and "BM-F". "RYAN" is an Arizona Department of Transportation monument established in 1973. "BM-f" was established in 1977 by the Maricopa County Highway Department.

The horizontal control is based on the Arizona State Plane Coordinate system. The coordinates of the brass cap (BC) at the quarter corner of Section 10/15, Township 1 South, Range 5 East were set by Greiner Engineering for the CMCN in 1985. This BC is point 1 (RO-21) of the survey for the Gilbert-Chandler floodplain delineation.

All of the control points, vertical and horizontal, used in this study were established by Franzoy Corey during August 1987 through November 1987 and are shown on sheet G-1 of appendix D titled "Survey Control and Sheet Index." Topographic mapping used for this study were obtained from photography taken in September 1987.

Locations of the cross sections used in the hydraulic analyses are shown on the topographic plan and profile drawings (appendix D).

Water-surface elevations at ponded locations were computed utilizing the level-pool reservoir routing routines in the COE HEC-1 computer program. Tables 1 through 4 describe the ponded areas along the Eastern Canal, Consolidated Canal, SPRR (Arizona Avenue), and the SPRR (Rittenhouse Road), respectively. The description of the location on these tables describes the concentration point where the ponding occurs. The tables also give an approximate station along the barrier and the cross sections closest to the pond. The pond elevation is also given in the table.

Water-surface elevations of flows along the hydraulic barriers between ponding sites were computed using the COE HEC-2 water-surface profiles computer program. Peak flows

TABLE 1 (Page 2 of 2)
Summary of Water Surface Elevations
Eastern Canal
Gilbert-Chandler Flood Insurance Study
October 1989
REV January 1990

100-yr flood

Location	Approximate Station Location	Cross Section	Water Surface Elevation
Approximately one-half mile north of Queen Creek Road	217+10	5074	1261.2[1]
Approximately one-half mile north of Queen Creek Road	217+70	5073	1261.2[1]
Ryan Road north approximately one-half mile	218+40 - 245+50	5072 - 5070 (ponded)	1259.8
Germann Road north approximately one-half mile	253+10 - 279+20	5068 - 5065.2 (ponded)	1262.5
One-quarter mile south of Pecos Road to one-half mile north of Pecos Road	293+80 - 369+00	5064 - 5055 (ponded)	1264.9
Chandler Blvd north to Val Vista Road	389+60 - 403+50	5054 - 5051 (ponded)	1265.8
Val Vista Road north to Ray Road	404+00 - 433+50	5049 - 5042 (ponded)	1266.6
Ray Road north of the SPRR	433+80 - 467+40	5041 - 5038 (ponded)	1267.7
SPRR north to Warner Road	477+70 - 509+20	5037 - 5033 (ponded)	1271.4
Warner Road north approximately one-half mile	509+40 - 538+20	5032 - 5027 (ponded)	1273.2
Approximately one-quarter mile south of Elliot Road	557+50	5026	1277.3
South side of Elliot Road	571+30	5025	1277.1[1]
Elliot Road north to Guadalupe Road	571+90 - 625+10	5024 - 5017 (ponded)	1277.1
Guadalupe Road north to Houston Road	625+60 - 652+90	5016 - 5012 (ponded)	1279.4
Houston Road north to Baseline Road	653+30 - 679+20	5011 - 5007.1 (ponded)	1280.8
Baseline Road north to Superstition Freeway	679+60 - 700+00	5007 - 5001 (ponded)	1282.4

[1] Flowing water overtops the canal bank. The elevation of the canal maintenance road plus 6 inches is used.

1 foot

5

Computed by GJFmK Date 5-6-91
Checked by BDR Date 5-8-91
Reviewed by Date
Approved by BDR Date 5-8-91

Job No. 10924 Page No. 6
Subject Gilbert Flood Protection
Flood Pumps
Final Comps
Sheet No. 1 of 3

Client wants to use constant speed pumps. Need to select a pump that can operate over the entire head range and pump an acceptable quantity of flow.

Determine range of pump discharge head.

Static Head

℄ Pump Discharge = El 74.5'
Low Lake Water level = El 44.7'
Normal Lake = El 46.7'
High Lake Level = EL 70'
During storms exceeding the design the high level can over top the storage basin @ El. 70'

Min. Static Head = 74.5' − 70' = 4.5'
Max. Static Head = 74.5' − 46.7' = 27.8'

Friction Head

Discharge Column = 35' @ 10"
Discharge Elbow = 25' @ 10" Equivalent
Total equivalent Length = 60'

From Cameron (for steel pipe)

	C = 100 (Old Pipe)		C = 130 (New Pipe)	
Flow	h_L	H_L	h_L	H_L
1000 gpm	1.00	0.60 feet	.49'/100'	.29 feet
1500	2.11	1.27	1.07	.64
2000	3.60	2.16	1.86	1.12
2500	5.44	3.26	2.85	1.71
3000	7.61	4.57	4.08	2.45

Job No. 10924 Page No. 7

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Subject Flood Pumps
Final Comps
Sheet No. 2 of 3

System Head = Static + Friction

@ High basin level System head

Flow gpm	C=100	C=130
1000	5.1' ✓	4.8' ✓
2000	6.7' ✓	5.6' ✓
3000	9.1' ✓	7.0' ✓

@ Low basin level System Head

Flow	C=100	C=130
1000	28.4' ✓	28.1' ✓
2000	30.0' ✓	28.9' ✓
3000	32.4' ✓	30.2' ✓

Pump Selection.

See attached Fairbanks-Morse Pump Performance Curve. Select Impeller B-1432-T. This requires a 30 hp motor. ✓

Pump capacity range = 2500 gpm @ 30' TDH ✓
3000 gpm @ 10' TDH ✓

8312

1770 RPM

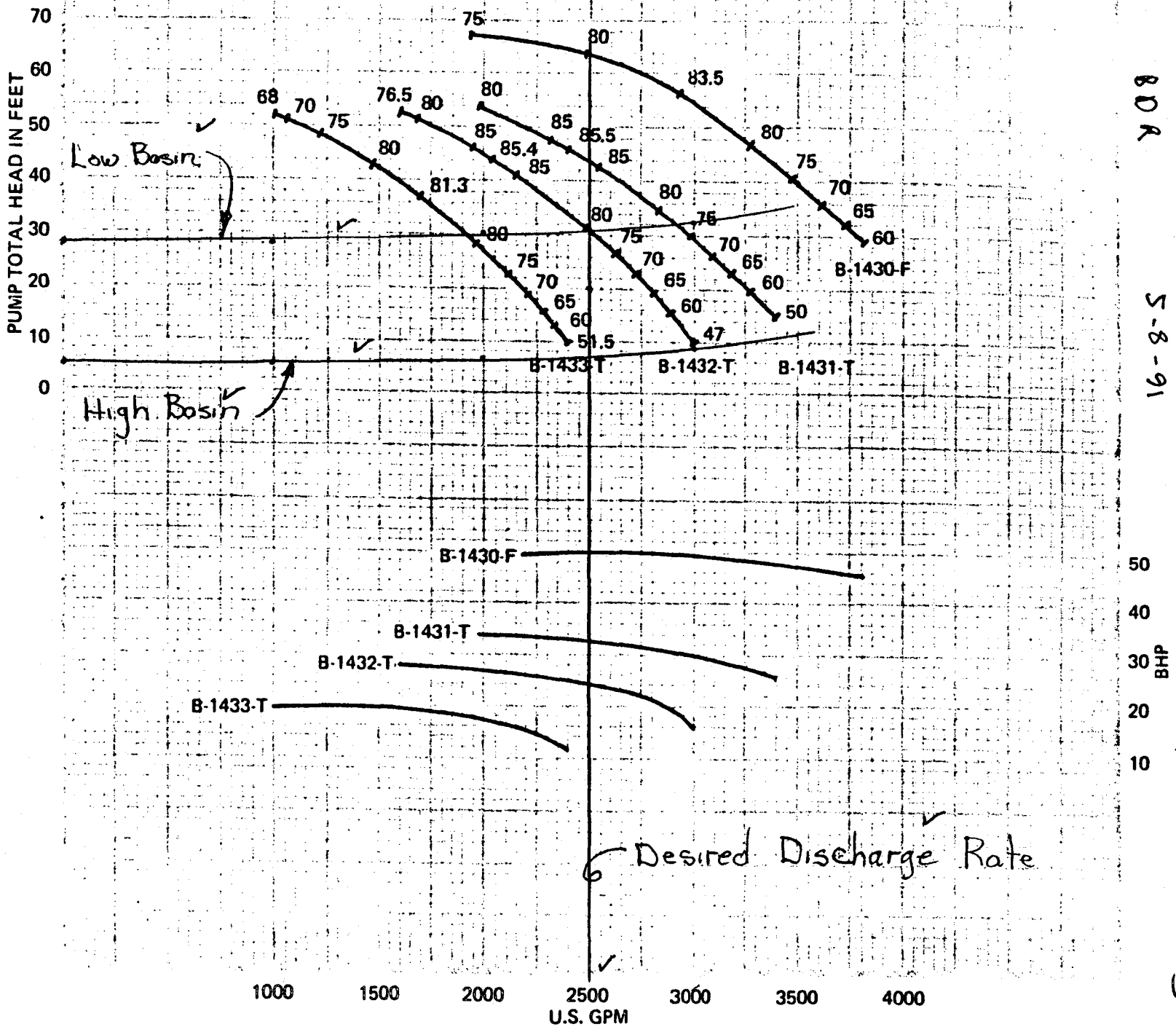
2 STAGE

12" COLUMN

12" CAST IRON ELBOW

1 3/16" LINESHAFT

2" ENCLOSING TUBE



GJFmK
BDR

5-6-91
5-8-91

Flood Pumps

3/3

Computed by GJFmK Date 5-6-91
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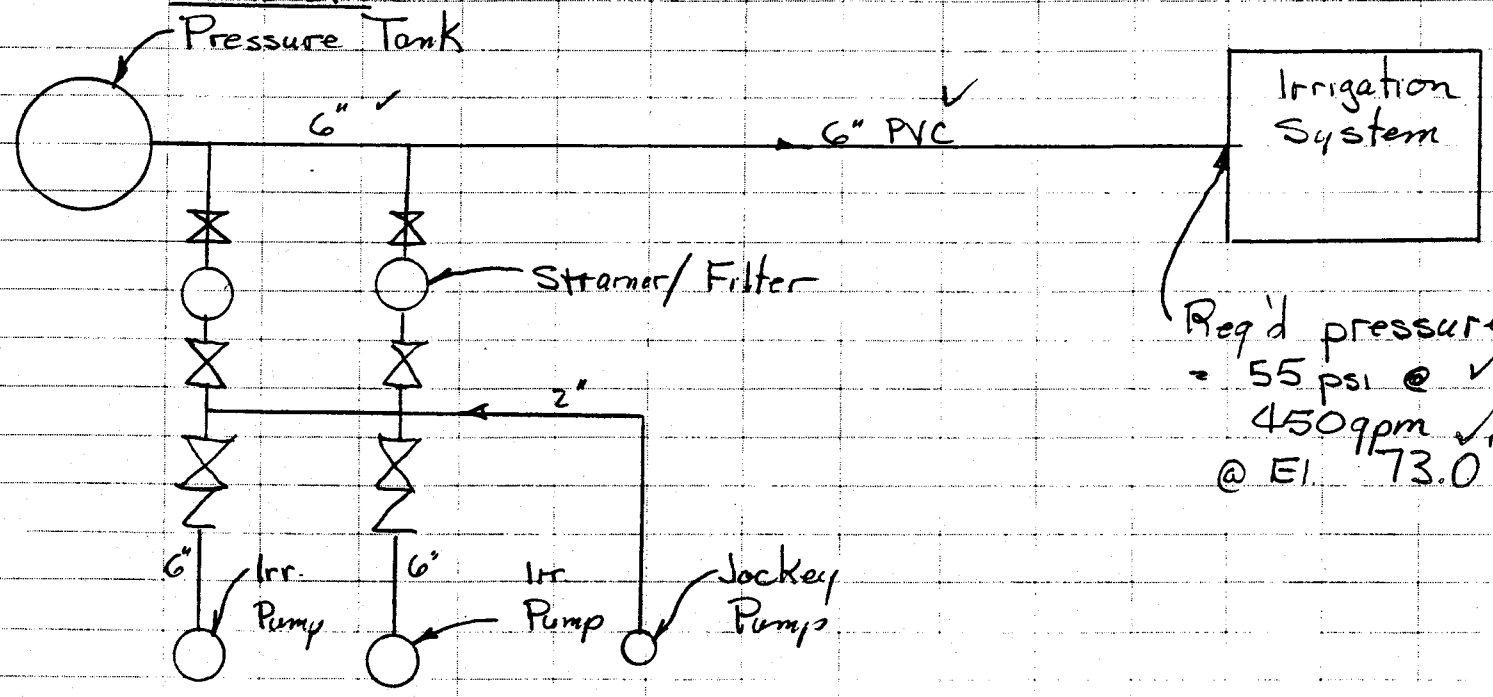
Reviewed by Date
Approved by BDR Date 5-8-91

Job No. 10924 Page No. 9
Subject Irrigation System Pump Sizing
Sheet No. 1 of 6

Average water use = 450 gpm ✓

Schematic



Calculate System Head (On Irrigation Pump) ✓

Static Head

Minimum HGL = El. 73.0 + (2.31)(55) = EL 200 ✓
Wet Well Water Level = − EL 44 ✓
Minimum Total Static Head = 156 Feet ✓

Job No. 10924 Page No. 10

Computed by G J Fink Date 5-6-91

Checked by BDR Date 5-8-91

Reviewed by Date

Approved by BDR Date 5-8-91

Subject Irrigation System Pump Sizing

Sheet No. 2 of 6

Friction Losses @ 450 gpm

a little concervative

1200' ± 6" PVC: (1.81/100)(1200) H_L 21.72

3- ⋈ @ 3' = 9 ~~18'~~

1- N @ 39' = 39'

2- L @ 15' = 30'

2 ⊢⊣ @ 31' = 62'

1 ⊥ @ 10' = 10'

Straight pipe = 35'

185 ~~194'~~ - use 200' of 6" @ C = 100

(200')(2.62'/100') = 5.24'

Pressure loss across strainer/filter:
Allow 8 psi total across unit: = 18.48

45.44'

Total Head

Static	156.
Friction	45.4
Total	201 Feet @ 450 gpm.

Pump Selection

See attached Fairbanks-Morse Pump Performance Curve for model 8M 7000.

Select 2 stages

Check pump hp @ Q ≈ 600 gpm.

$$BHP = \frac{GPM \times TH}{3960 \times eff} = \frac{(600)(75')}{(3960)(.68)} = 16.7 \text{ hp/stage}$$

For two stages BHP = 2 x 16.7 = 33.4

Use 40 hp motor

Job No. 10924 Page No. 11

Computed by G J Fink Date 5-6-91

Checked by BDR Date 5-8-91

Reviewed by Date

Approved by BDR Date 5-8-91

Subject Irrigation System Pump Sizing

Sheet No. 3 of 6

Check pump capacity @ automatic shut off

Minimum pressure in pressure tank @ pump start.

= 55 psi + Friction

= 55 psi + $\frac{21.72}{2.31}$ = 64.4 psi

Set pump start @ 65 psi

Pressure tank is sized for 25 psi operating range: @ Pump Shut off pressure = 90 psi

Head on pump @ shut off

Static = (90 psi)(2.31) + [77'-47'] = 238'

Friction: Assume: 6'

Total head @ Shutoff ≈ 244'

For 2 stage pump = 122'/stage

See attached pump curve.

Pump capacity @ shut off = 330 gpm

Computed by G JFmK Date 5-6-91
Checked by BDR Date 5-8-91
Reviewed by Date
Approved by BDR Date 5-8-91

Job No. 10924 Page No. 12
Subject Irrigation System Pump Sizing
Sheet No. 4 of 6

Jockey Pump

The jockey pump will be set to operate as follows:

Pump Start: 80 psi in tank
Pump Off 90 psi in tank

Req'd capacity ≈ 33 gpm

Static Head

HGL @ 80psi = (80)(2.31) + 77'± = Elev. 262'
Wet Well Level = - 44
Minimum Total Static Head = 218'

Friction Head

Allow for 5 psi ≈ 12'

Total Head

Static = 218
Friction = 12
Total 230' (= 100 psi)

Pump Selection

See attached Red Jacket Performance Curves

Pump On @ 80 psi (230' TDH) Q = 33 gpm
Pump Off @ 90 psi (230' + 23') Q = 30 gpm

Pump Off @ 65 psi (230 - 35' = 195 TDH) Q = 37 gpm.
(Pump shuts off when irrigation pump starts. All pumps stop @ 90 psi in tank)

BDR 5-8-91

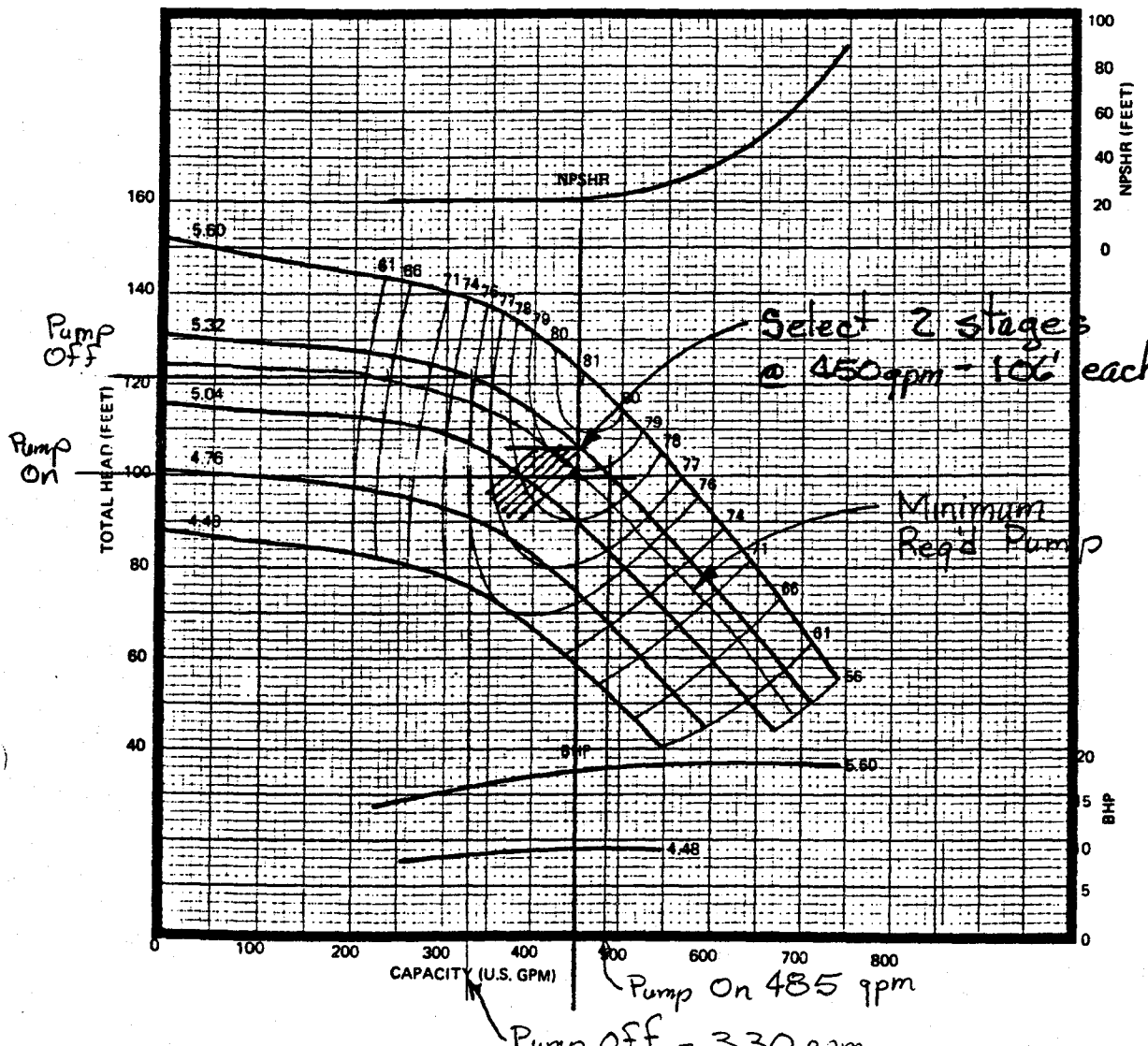
(13)

DEEP WELL VERTICAL TURBINE PUMPS
PERFORMANCE

8M 7000

3550 RPM

IMPELLER T7CA92



EFFICIENCY CORRECTIONS (1)

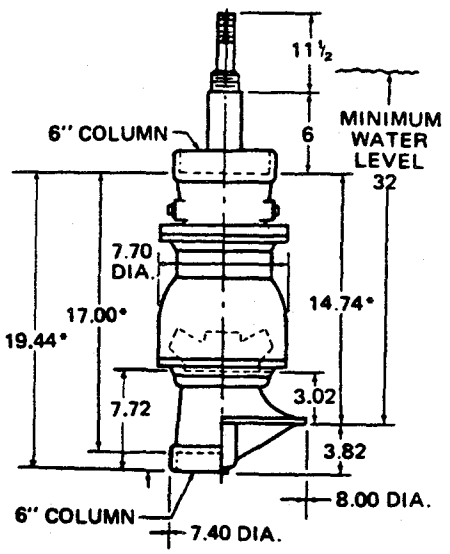
NUMBER OF STAGES	EFFICIENCY CHANGE
1	-7.0 POINTS
2	-3.0 POINTS
3	-1.0 POINTS
4	NO CHANGE
5	NO CHANGE
6 OR MORE	NO CHANGE

BOWL MATERIAL	EFFICIENCY CHANGE
CAST IRON	-2.0 POINTS
ENAMELED C.I	NO CHANGE

IMPELLER MATERIAL	EFFICIENCY CHANGE
CAST IRON	-1.0 POINTS
BRONZE	NO CHANGE
ENAMELED C.I	NO CHANGE

(1) Refer to "Application and Reference Data" for head correction.

DIMENSIONS (Inches)



*Add 6.58 for each additional stage

TECHNICAL DATA

DATA	VALUE
MAXIMUM OPERATING SPEED	3600 RPM
MAXIMUM NUMBER OF STAGES	8**
PUMP SHAFT DIAMETER	1 3/16 IN.
IMPELLER EYE AREA	6.93 SQ. IN.
MAXIMUM SPHERE SIZE	.56 IN.
K_t (THRUST FACTOR)	4.15 LBS./FT.
K_a (ROTOR WT. PER STAGE)	5.25 LBS.
BOWL WT. (FIRST STAGE)	82 LBS.
BOWL WT. (EACH ADD'L. STAGE)	31 LBS.
ALLOWABLE SHAFT STRETCH	.39 IN.**
WK^2 (FIRST STAGE)	.17 LBS.-FT.2
WK^2 (EACH ADD'L. STAGE)	.16 LBS.-FT.2
BOWL RING CLEARANCE	.014/.018 IN.

**These are nominal values. Refer to "Application and Reference Data" for information further limting or extending these values.

[redacted]

BDR 5-8-91

[redacted]

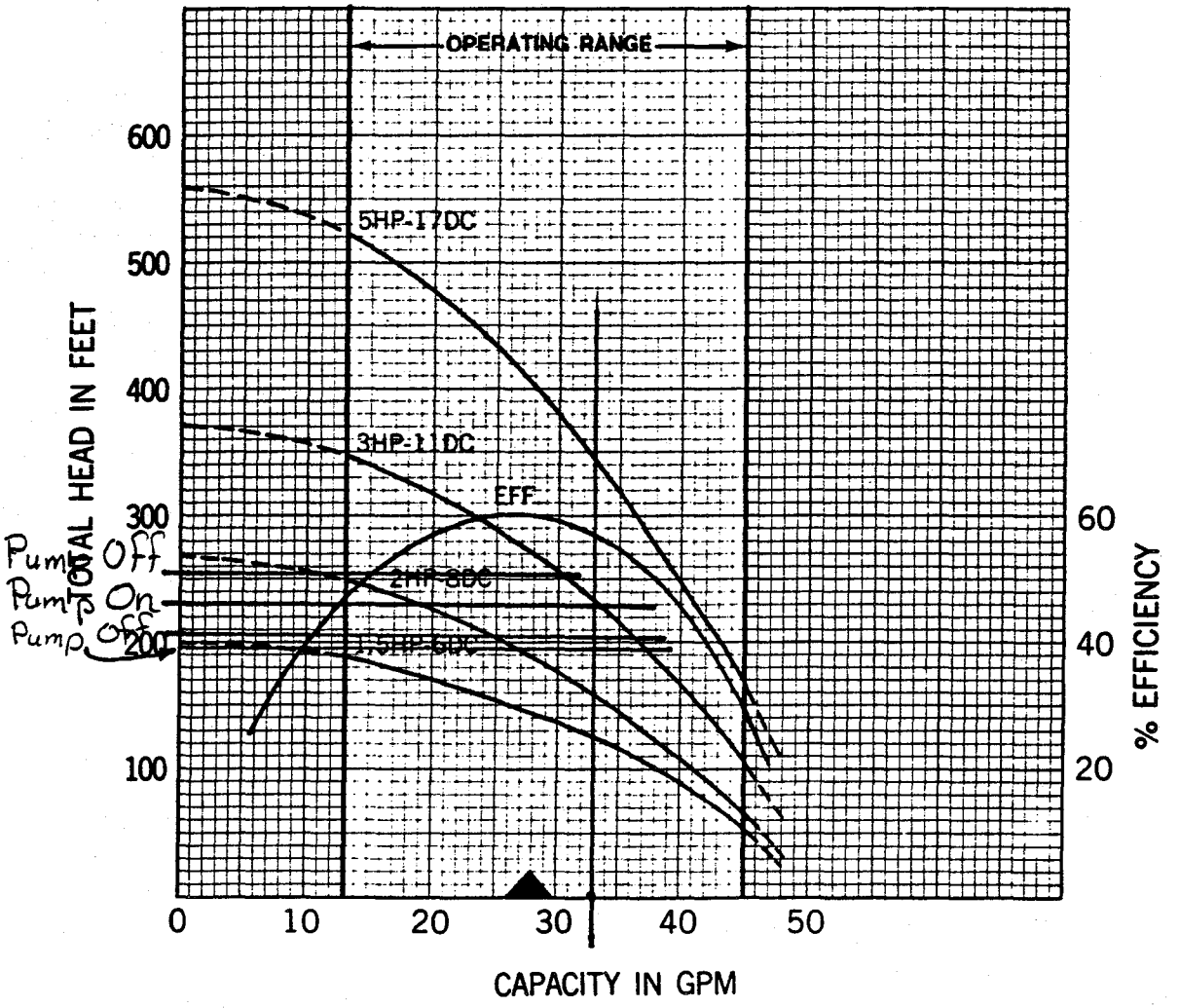
RED JACKET®
28 GPM SERIES "DC" PUMPS

BIG-FLO® (14)

DESIGNED FOR COMMERCIAL CAPACITIES FROM WATER LEVELS TO 500 FEET

The efficient design of the 28 GPM Series allows it to pump from deeper than average wells and deliver rated capacity. Ideally suited for farms, dairies, and commercial establishments. All fit 4" wells.

PERFORMANCE CURVES



RED JACKET PUMP ● A MARLEY PUMP COMPANY
DAVENPORT, IOWA

MODEL 28 GPM "DC" ▲ RATED FLOW

RPM 3450 STAGE "DC" SERIES DATE 2-1-68

GUARANTEED AS MINIMUM PERFORMANCE ONLY IF CERTIFIED

MINIMUM WELL SIZE: 4" I. D.

Computed by: G.J. Fox Date: 3-14-91
Checked by: BDR Date: 3-21-91
Reviewed by: Date:
Approved by: BDR Date: 3-21-91

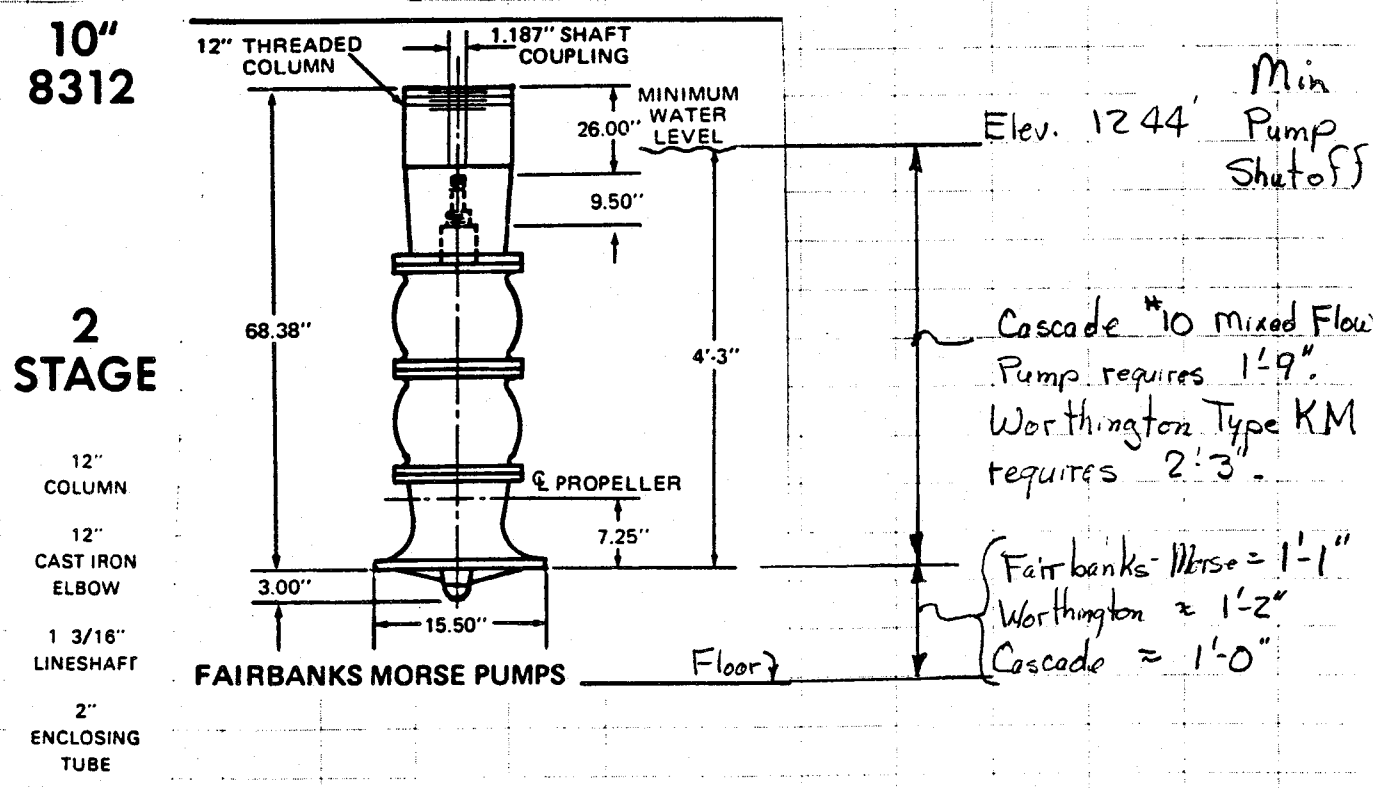
Job No.
Page No. 15
Subject: Pump Station Water levels And Pump Submerge
Sheet No. 1 of 3

Reference: See the attached Pump Station Cross Section
The minimum water level in the holding basin is elevation 1244'-8" ✓

The inlet pipe will be large (48" ✓) for the ✓ maximum flow and will have negligible head loss (0.015'/100
(The primary head loss will occur at the inlet structure.)

Thus set the minimum water level for flood pump shut off at elevation 1244'-0" in the wet well.
Normal flood pump shut off will be elev. 1246'-8" with irrigation pumps operating between 1246'-8" and 1244'-8".

Pump Submergence



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Checked by BDR Date 3-21-91
Reviewed by Date
Approved by BDR Date 3-21-91

Job No. Page No. 16
Subject Pump Station Water Levels
Sheet No. 2 of 3

Wet Well Floor Level

The bottom of the wet well should be set so that minimum submergence can be provided for several pump manufacturers.

Minimum Water level (Pump Off)	1244'-0" ✓
- less submergence	(4'-3") ✓
- less floor clearance	(1'-2") ✓
Max. Floor level based on submergence ⇒	1238'-7" ✓

The floor may have to be lower to allow for the inlet structure/inlet pipe design. The inlet structure/inlet pipe should drain towards the pump station wet well

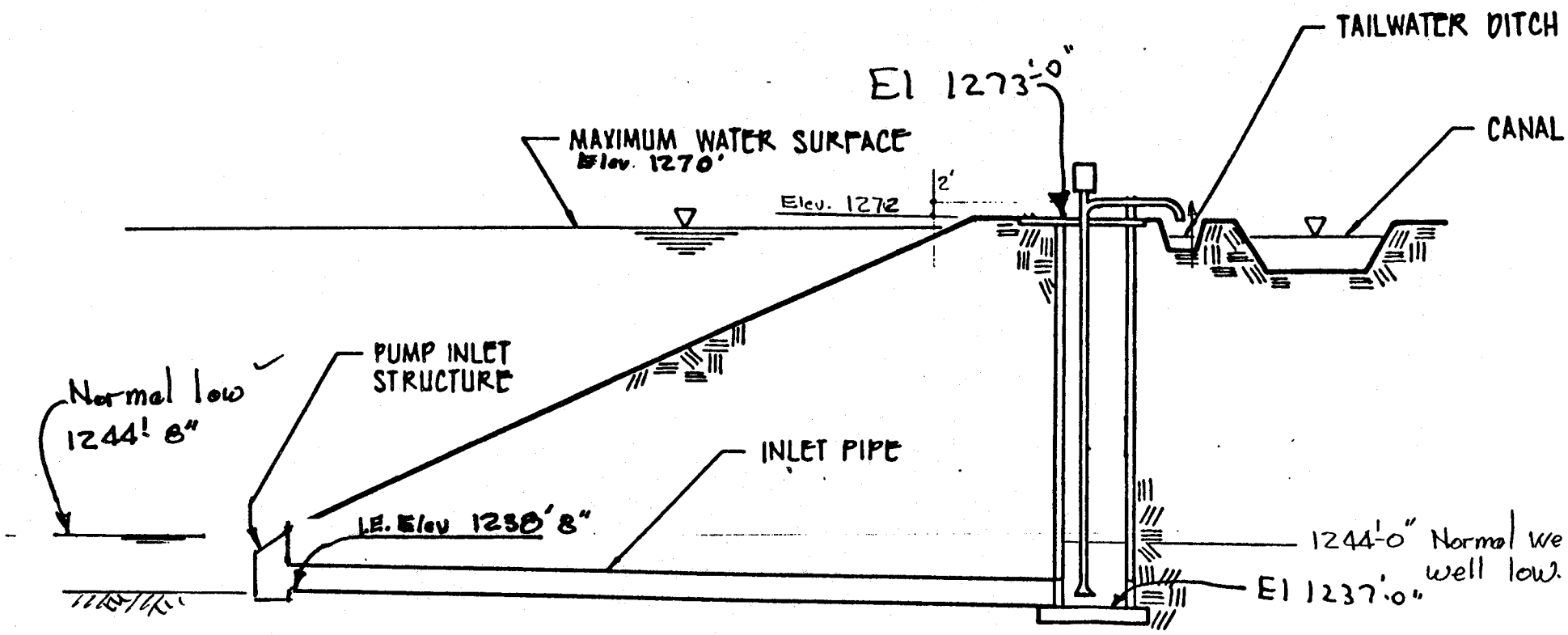
See G J FmK Comp dated 3/14/91 "Pump Station Intake Bar Screen"

Invert of 48" RCP @ Intake = 1238'-8" ✓

Set Invert of 48" RCP @ Wet Well = 1238'-0" ✓

Set Floor of Wet Well at 1237'-0" ✓

STANLEY CONSULTANTS

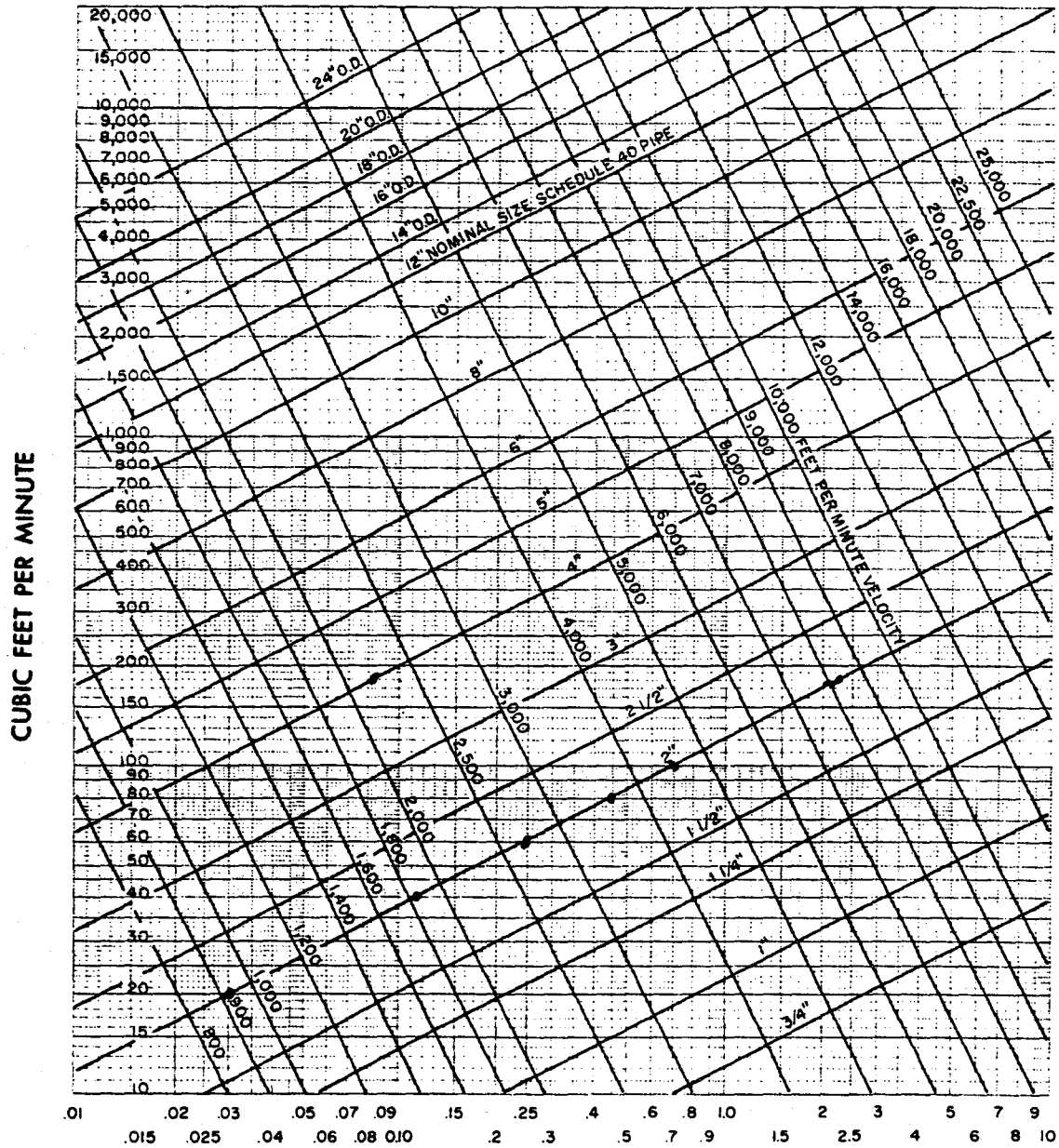


BDR 3/21/91

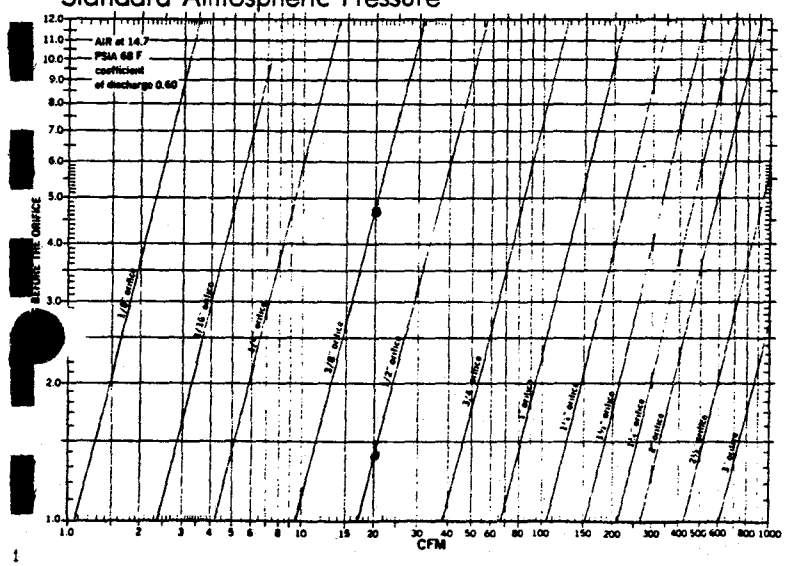
PUMP STATION CROSS-SECTION

3/3

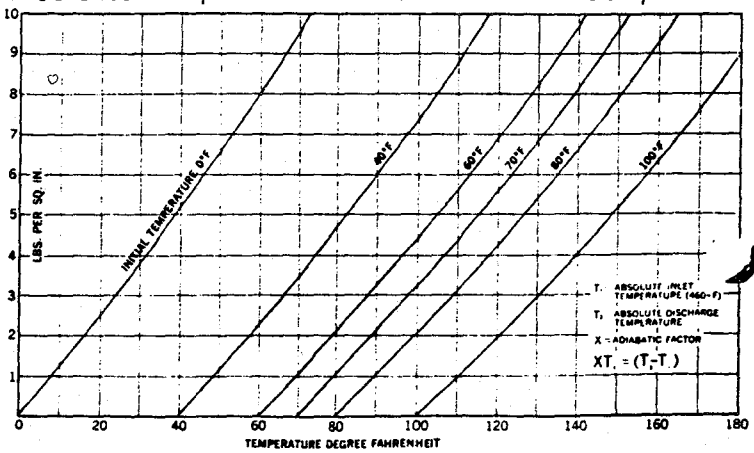
Friction loss per 100 feet—schedule 40 steel pipe air density 0.075 pounds per cubic feet



Discharge of Air Through an Orifice in CFM Free Air at Standard Atmospheric Pressure



Theoretical Temperature Rise due to Adiabatic Compression



Job No. 10924 Page No. 18

Computed by G.J.Fink Date 5-6-91
Checked by B.D.R Date 5-8-91
Reviewed by Date
Approved by B.D.R Date 5-8-91

Subject Irrigation System Pressure Tank Sizing

Sheet No. 1 of 5

Operating Conditions / Parameters

1. Irrigation pump cap. range = 330 gpm — 450 gpm
2. Jockey pump cap range = 30 gpm — 37 gpm
3. Irrigation system demand = 450 gpm. max.

Per discussions with irrigation system subconsultant, the normal flow range for irrigation will be maintained from 420± gpm to 450 gpm.

The above irrigation pump range will handle all flows between 330 gpm and 450 gpm and not require cycling the pump On and Off. The pump will cycle for all flows between 37 gpm and 330 gpm.

The pump will cycle between tank pressures of 65 psig - 90 psig.

Size Pressure Tank to limit the motors to 6 starts per hour.

3004

Computed by G J Fink Date 5-6-91
Checked by BDR Date 5-8-91
Reviewed by Date
Approved by BDR Date 5-8-91

Job No. 10924 Page No. 19
Subject Irrigation System Pressure Tank Sizing
Sheet No. 2 of 5

Determine Net Operating Volume

The average pump capacity = 390 gpm over the pressure range of operation.

The minimum working volume req'd = $\frac{TD}{4}$

T = Cycle Time, min.
D = Pump Capacity, gpm.

$= \frac{(10\,min)(390\,gpm)}{4}$

$= 975$ gallons

Refer to Fig 24 (attached on sheet 5) for Hydropneumatic Tank operation

$P_1 = 90\,psig = 104.7\,psia$
$P_2 = 65\,psig = 79.7\,psia$

$\frac{P_1}{P_2} = \frac{104.7}{79.7} = 1.31$

Select a tank that is 30% full @ pump OFF (90 psig)
Thus @ Pump Start (65 psig), the tank is 30 - 22 = 8% full
The working volume = 22% of total tank volume.

Gross Tank Volume Req'd = $\frac{975\,gal}{.22}$ = 4432 gallons
For single pump operation

Tank Size

Dia	Cap/Ft	Length, ft.
5	$.785(5)^2(7.48) = 147$ gal	30
6	211	21
7	287	15.4
8	376	11.8

This size of tank is too costly. The tank capacity can be reduced to 1/2 if the irrigation pumps are alternated in operation.

3004

Job No. 10924 Page No. 20

Computed by G. J. Fink Date 5-6-91

Checked by BDR Date 5-8-91

Reviewed by Date

Approved by BDR Date 5-8-91

Subject Irrigation System
Pressure Tank Sizing

Sheet No. 3 of 5

Summary

Select a tank w/ 2230 gallon capacity ✓

Use 5' dia x 15' long. ✓

Control Settings

@ 90 psi shutoff - % full = 30% ✓

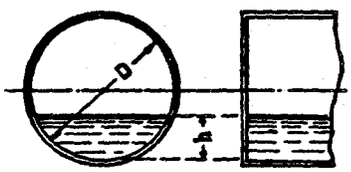
@ 65 psi start - % full = 8% ✓

TANK CAPACITIES, HORIZONTAL CYLINDRICAL

Contents of Tanks with Flat Ends when Filled to Various Depths

Contents in U. S. gallons per 1 foot of length.

Diameter of tank inches	*Full tank*	Depth of liquid, in inches = *h*																			
		3"	6"	9"	12"	15"	18"	21"	24"	27"	30"	33"	36"	39"	42"	45"	48"	51"	54"	57"	60"
12"	5.88	1.15	2.94	.....	.....																
18"	13.22	1.45	3.86	6.61	.....																
24"	23.50	1.70	4.60	8.05	11.75																
30"	36.72	1.91	5.23	9.27	13.72	18.36	.....	.....													
36"	52.88	2.12	5.79	10.34	15.43	20.85	26.44	.....													
42"	71.97	2.28	6.31	11.31	16.97	23.07	29.47	35.99													
48"	94.01	2.45	6.78	12.20	18.38	25.10	32.20	39.54	47.00	.....	.....										
54"	118.98	2.60	7.22	13.04	19.68	26.97	34.72	42.80	51.08	59.49	.....										
60"	146.89	2.75	7.64	13.82	20.91	28.72	37.06	45.82	54.87	64.11	73.44										
66"	177.73	2.89	8.04	14.56	22.07	30.37	39.28	48.65	58.39	68.41	78.59	88.86	.....	.....							
72"	211.52	3.02	8.42	15.26	23.17	31.92	41.36	51.32	61.71	72.45	83.41	94.54	105.76	.....							
78"	248.24	3.15	8.78	15.94	24.21	33.41	43.34	53.86	64.87	76.27	87.97	99.90	111.97	124.13							
84"	287.90	3.26	9.12	16.57	25.24	34.85	45.24	56.29	67.87	79.91	92.30	104.98	117.85	130.87	143.95	.....	.....				
90"	330.49	3.43	9.46	17.20	26.20	36.21	47.05	58.61	70.75	83.39	96.43	109.81	123.45	137.28	151.23	165.25	.....				
96"	376.02	3.50	9.79	17.80	27.13	37.52	48.81	60.84	73.52	86.73	100.39	114.44	128.79	143.40	158.17	173.06	188.01				
102"	424.50	3.61	10.10	18.37	28.01	39.00	50.49	62.99	76.18	89.94	104.20	118.89	133.92	149.25	164.81	180.53	196.37	212.25	.....	.....	.....
108"	476.10	3.71	10.39	18.94	28.90	40.03	52.14	65.09	78.74	93.04	107.87	123.17	138.87	154.89	171.19	187.71	204.37	221.14	238.05	.....	.....
114"	530.25	3.78	10.74	19.49	29.75	41.22	53.73	67.10	81.24	96.05	111.43	127.31	143.63	160.33	177.33	194.60	212.05	229.65	247.37	265.13	.....
120"	587.54	3.91	10.98	20.02	30.57	42.39	55.26	69.06	83.65	98.95	114.87	131.32	148.25	165.58	183.27	201.24	219.46	237.87	256.43	275.08	293.77



To ascertain the contents of a tank over one-half full: Let *h* = depth of unfilled portion. Find from the table the quantity corresponding to a depth *h*. Subtract this quantity from the contents of a full tank.

Determine h for 30% and 8% full ✓
Disregard the capacity in the dished heads. ✓

3004

Job No. 10924 Page No. 21

Computed by G. J. Fink Date 5-6-91

Checked by BDR Date 5-8-91

Reviewed by Date

Approved by BDR Date 5-8-91

Subject Irrigation System Pressure Tank Sizing

Sheet No. 4 of 5

Full tank cap. = 147 gal/ft

@ 30% cap = (.3)(147) = 44 gal/ft ⇒ 20" depth
Pump off

@ 8% Cap = (.08)(147) = 11.76 gal/ft ⇒ 8" depth
Pump start

3004

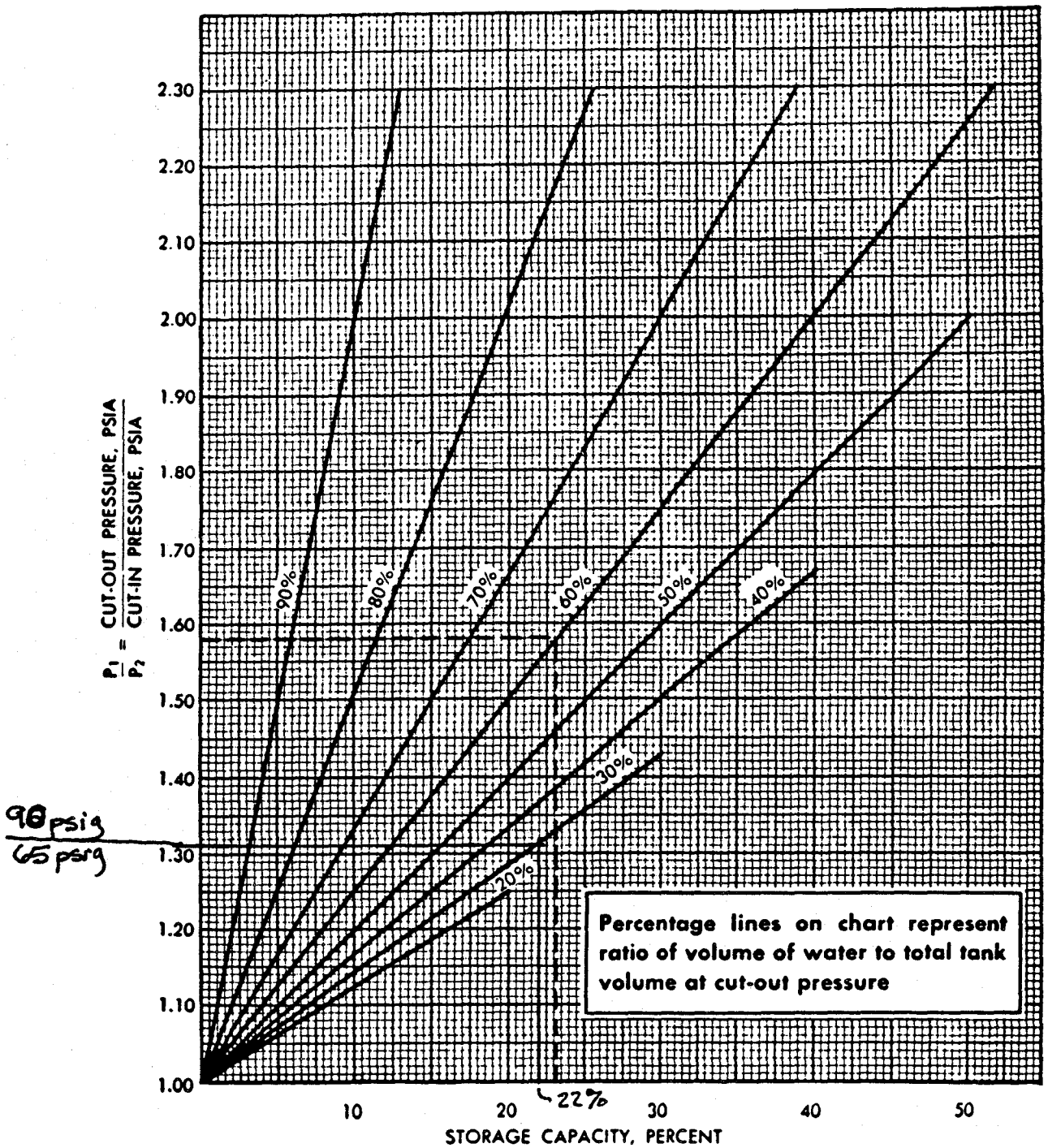


FIG. 24. Hydro-pneumatic tanks—relation between pressure range and storage capacity.

GALVANIC CORROSION †

(a) *Definition of Galvanic Corrosion* — Galvanic corrosion may be defined as the accelerated electro-chemical corrosion produced when one metal is in electrical contact with another more noble metal, both being immersed in the same corroding medium, which is called the electrolyte. Corrosion of this type results usually in an accelerated rate of solution for one member of the couple and protection for the other. The protected member, the one that does not corrode, is called the nobler metal. Note that as galvanic corrosion is generally understood, it consists of the total corrosion, which comprises the normal

Computed by G. J. Fink Date 3-26-91
Checked by BDR Date 4-26-91
Reviewed by Date
Approved by BDR Date 4-26-91

Job No. 10924 Page No. 23
Subject Irrigation System
Water Line Size
Sheet No. 1 of 3

Refer to attached site plan for line location and length.

Per telephone call w/ Tim Crall 3/11/91, Total irrigation flow = 445 gpm. ✓

Total Length of line between filter/stainer outlet and junction with city water near parking lot ≈ 1000 ft. ✓

For calculation of head loss - 1). use 1200 equivalent feet of pipe ✓
2). 500 gpm flow. (later changed to 450 gpm)

Compute losses for 6" and 8" line sizes and D.I (C = 130±) and PVC (C = 140)

Head Losses

These may be slightly conservative

PVC	$h_L/100'$	H_L
6"	1.81	22' ✓
8"	0.45	5.4' ✓

D.I.	$h_L/100$	
6"	2.02 ✓	24' ✓
8"	0.465 ✓	5.6' ✓

The material choice of pvc vs ductile iron is not affected significantly by head loss. Thus the size selection should be based on the difference in pipe cost. ✓

3004

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Job No. Page No. 24
Subject Irrigation System
Water line size
Sheet No. 2 of 3

Determine annual power costs difference for 6" vs 8"

Basis of comparison.
1. 12 hrs/day - 6 days/wk for annual average
2. Power @ $0.05/kwhr.

$$BHp = \frac{gpm \times TDH}{3960 \times eff}$$

Head diferential between 6 & 8"

$$= \frac{(450\,gpm)(18')}{(3960)(.80)} = 2.56 = 2.40 @ 85\% eff$$

$$\text{Annual Power} = 2.56\,hp \times \frac{.746\,Kw}{hp} \times \frac{72\,hr}{wk} \times \frac{52\,wk}{yr}$$

$$= 7150\ Kwhr/yr$$

Annual cost Difference $= 7150 \times .05 = \$357/yr$

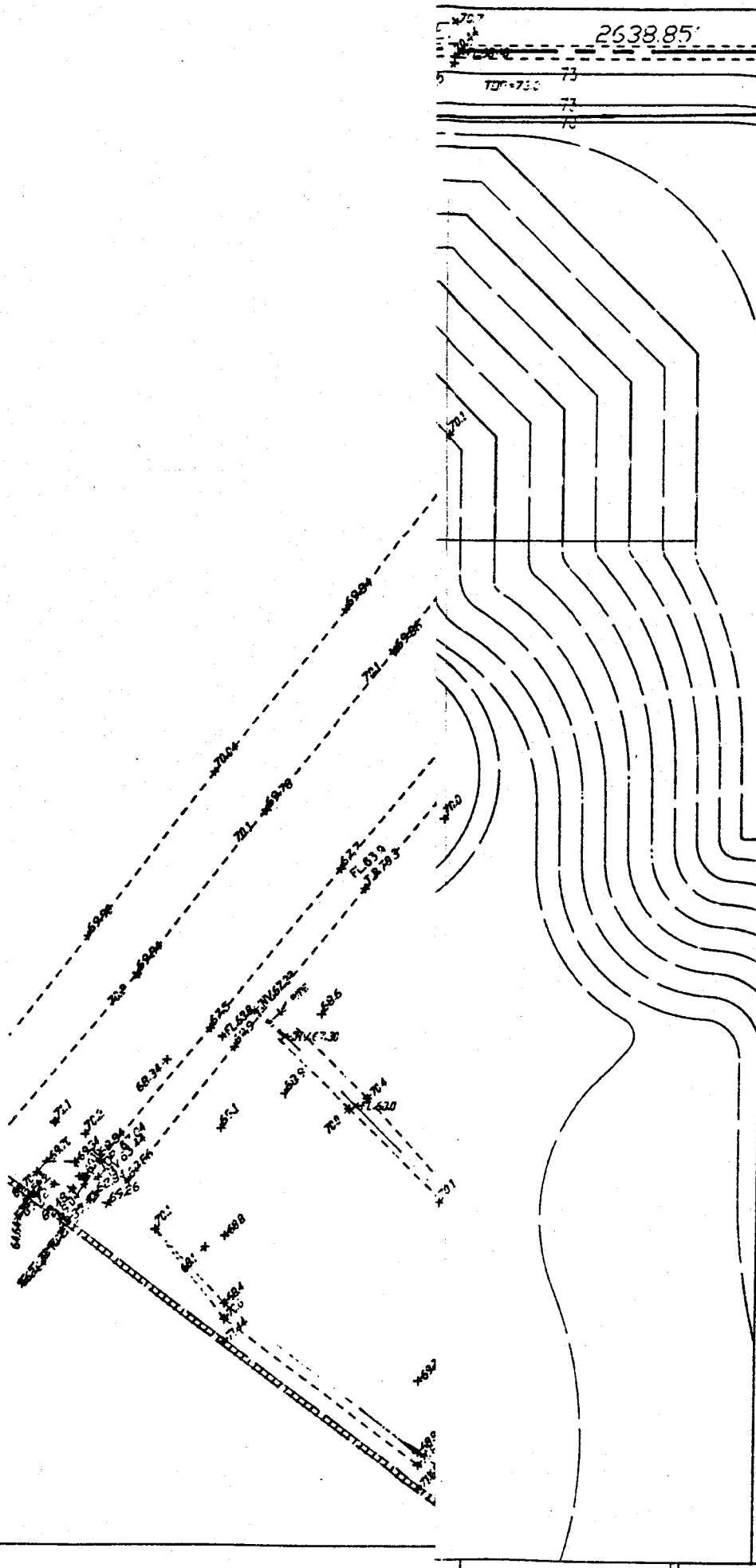
Estimate capital cost difference: $1000\,ft \times \$8/ft \pm$
$= \$8{,}000$

$$\text{Capital Cost Recovery} = \frac{\$8{,}000}{\$357/yr} = 22.4\ yrs.$$

This is too long a time to recovery the higher costs for an 8" water line.

Select 6"ϕ

3004



BERLEY CONSULTANTS INC.

FLOOD CONTROL DISTRICT OF MARICOPA COUNTY

DESIGN

Computed by: G J Fink Date: 3-22-91
Checked by: B.D.R. Date: 4-26-91
Reviewed by: Date:
Approved by: B.D.R. Date: 4-26-91

Job No. 10924 Page No. 26
Subject: Lake Aeration System
Sheet No. 1 of 11

Objective: Provide some mixing and aeration in the lake to keep it "fresh".

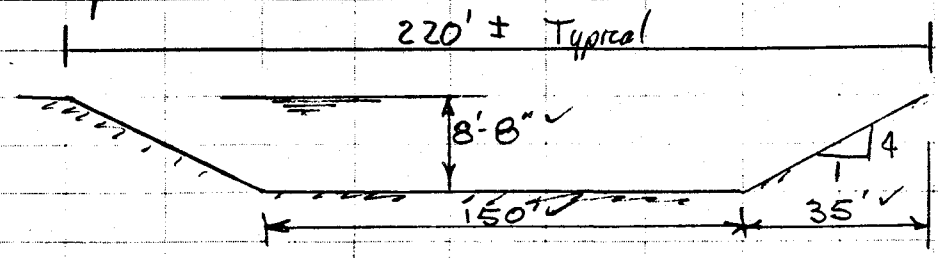
Lake Volume

Max. surface area ≈ 3.7 acre

Maximum depth (normal dry period) = 1246'-8" high water
− 1238 bottom
8'-8"

Gross Volume: (3.7 acres)(8.667') = 32.07 acre feet
= 10.4± million gallons

Actual volume will be less because of side slopes on the bank.



Gross Area = 220' x 8.667' = 1906 ft²
Net Water area = (185')(8.667') = 1603 ft²

$$\frac{\text{Net}}{\text{Gross}} = \frac{185}{220} = 0.84$$

Approximate Net Volume = 0.84 (10.4 mmg)
≈ 8.75 mmg
≈ 1.17×10^6 ft³

3004

Job No. 10924 Page No. 27

Computed by G J Fink Date 3-22-91

Checked by BOR Date 4-26-91

Reviewed by Date

Approved by BOR Date 4-26-91

Subject Lake Aeration System

Sheet No. 2 of 11

Mixing is the primary objective in maintaining "freshness". The dissolved oxygen added to the water is a secondary benefit.

Static tube aerators will be used for mixing. A 12" ϕ tube will pump 400 gpm at an air flow requirement of 20 scfm.
The secondary pumping rate is usually 4-6 times the primary rate. (1600 gpm – 2400 gpm). The pumping rate is directly proportional to air flow. thus

10 cfm ⇒ 200 gpm.

Size the number of aerators by trial and error. Use 2000 gpm secondary pumping rate @ 20 scfm

Sample calculation

$$\text{No. aerators} = \frac{(\text{No. of Turnovers})(\text{Vol. of Lake})}{(2000\ \text{gpm})(1440\ \text{min/day})} = \frac{8{,}750{,}000 \times \text{Turn}}{2{,}880{,}000\ \text{gpd}}$$

Turnovers/day	(@ 20 scfm) No. of Aerators	Total Air Flow, scfm
1	3	60
2	6	120
3	9	180 ←
4	12	240

Select 9 units and space @ 100' intervals along the ℄ of the lake. See attached site plan for general location.

Friction Losses

Want to minimize line losses to allow for near equal distribution of air. The loss across the orifice at each aerator should be the most significant head loss.

Size Lateral pipe to limit total friction loss to 0.075 psi

Size orifice to have head loss of 0.75 psi @ 20 scfm

Length of header with Q = 180 scfm = 300 ft

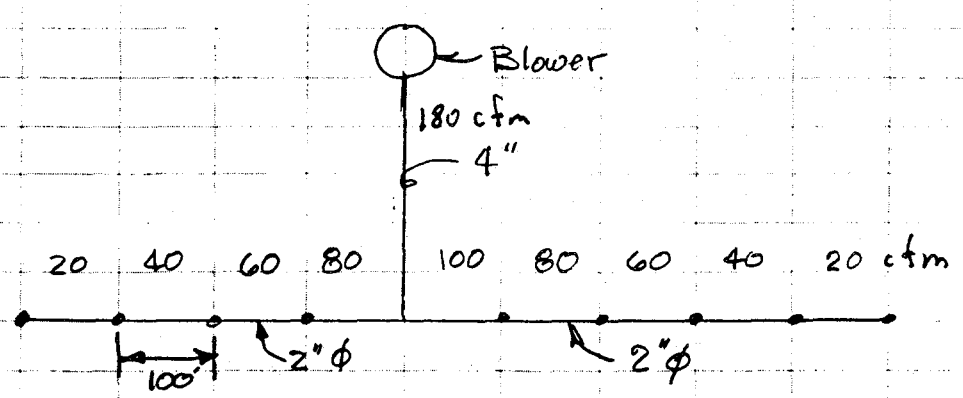
Length of header with Q = 100 scfm = 400 ft
(Assumes all air in a header travels to the end. This is a conservative assumption)

$$H_L/100 = \frac{0.075\ psi}{4} = 0.019\ psi/100'$$

This is too conservative and results in requiring a 4" ϕ pipe.

Use: 2" pipe (polyethylene). Calculate friction losses for each 100' section between aerators

Schematic



Computed by G J Frnk Date 3-22-91
Checked by B.D.R. Date 4-26-91
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Approved by BDR Date 4-26-91

Job No. 10924 Page No. 29
Subject Lake Aeration System
Sheet No. 4 of 11

Flow, cfm	H_L *
20	.03 psi
40	.11 psi
60	.25
80	.45
100	.70
180	.24 ← (300 ft x .08 psi/100 in a 4" ϕ)

Total 1.78 psi for pipe from blower discharge to last aerator.

* These values are for steel pipe. Using PE pipe will be conservative and will offset losses @ bends, fitting etc. not specifically included

Need to allow for losses of blower accessories.

Air Filter	.20 psi
Silencer	.25
	.45 psi

Total piping losses = 1.78 + .45 = 2.23 psi

Use 2.50 psi

Orifice Loss (20 cfm)

@ 3/8" ϕ h_L = 4.8 psi ← Use
@ 1/2" ϕ h_L = 1.4 psi

Total Friction Losses

		Nearest orifice (.94+.45)	Farest orifice
Pipe =	2.50 psi	1.39	2.23
Orifice =	4.80	4.80	4.80
	7.30 psi	6.19	7.03
		88 %	

Use 7.50 psi

3004

Computed by G S Fink Date 3-22-91
Checked by BDR Date 4-26-91
Reviewed by Date
Approved by BDR Date 4-26-91

Job No. 10924 Page No. 30
Subject Lake Aeration System
Sheet No. 5 of 11

Static Head.

Normal High Lake level = EL 46.67'
Average Lake Bottom Elev = 38.0'

Max Water Depth = 8.67'

The aerator orifice will be positioned about 1' above the bottom. However, irregular grading of the bottom will cause some elevation differences between aerators.

7.67' water = 3.32 psi

Total Head Required

Friction:	7.50 psi
Static	3.32 psi
Total	10.82 psi

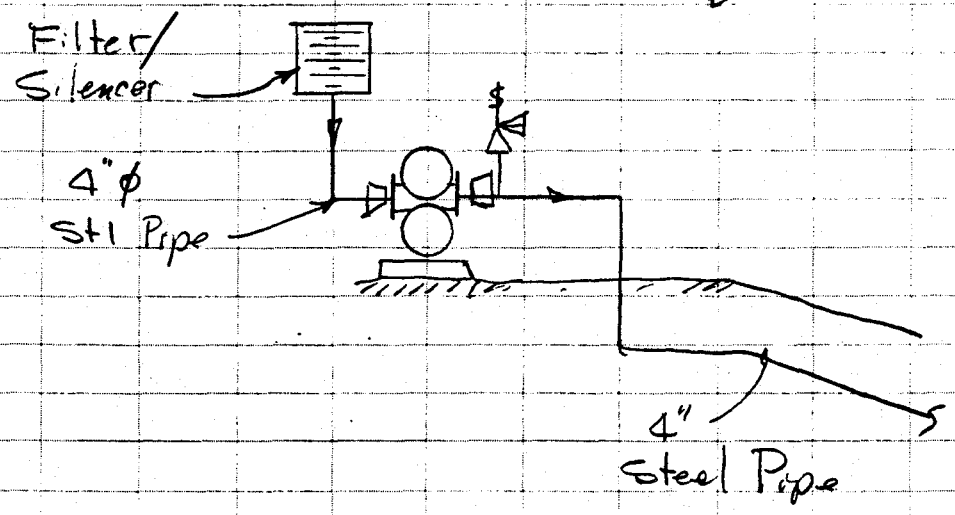
Use 11.00 psi

Blower

See catelog sheets for Tuthill blowers
The minimum required hp ≈ 15. for 180 cfm @ 11.0 psig. Specify a 15 hp.

The blower should be protected by a pressure relief valve.

Typical Installation

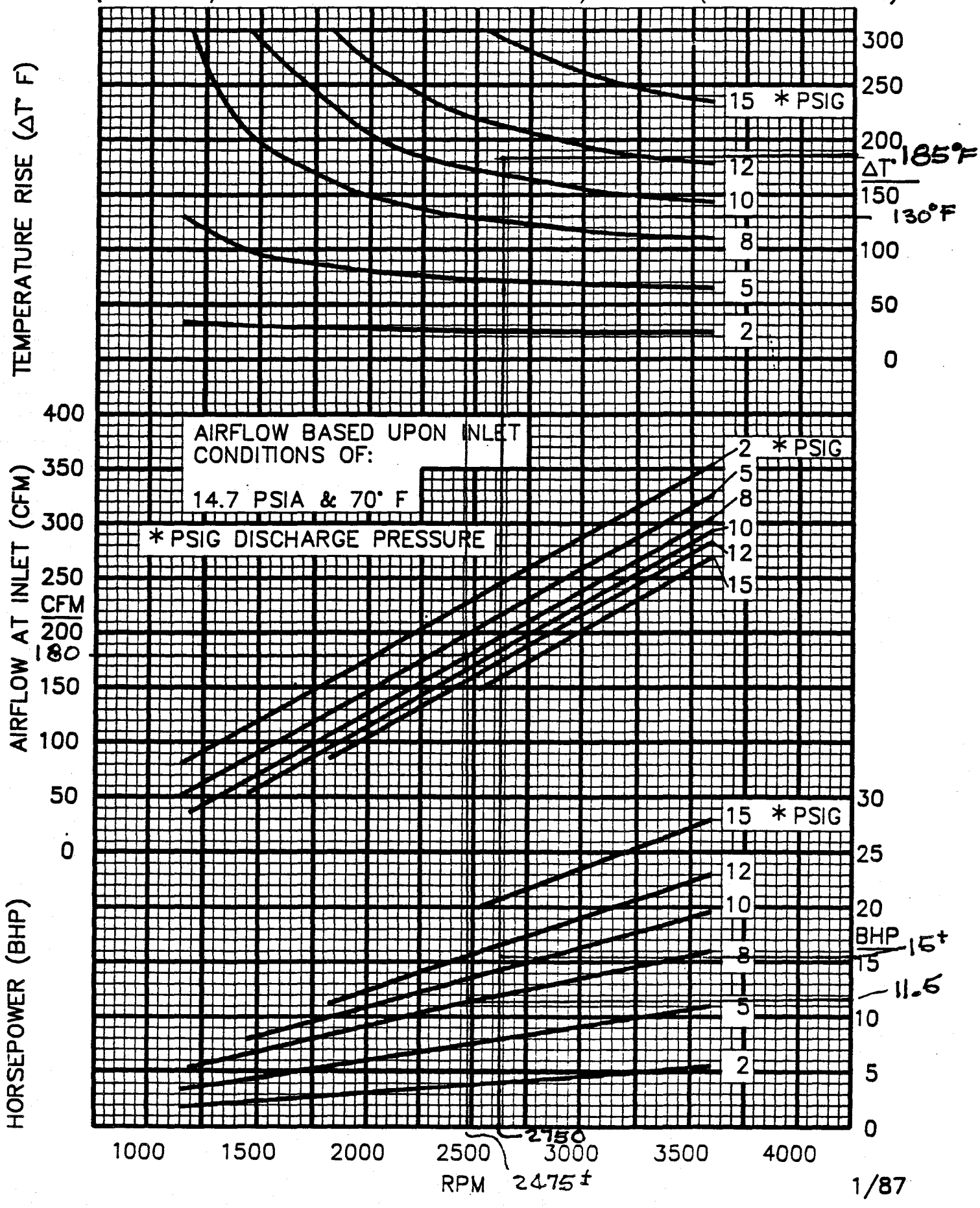


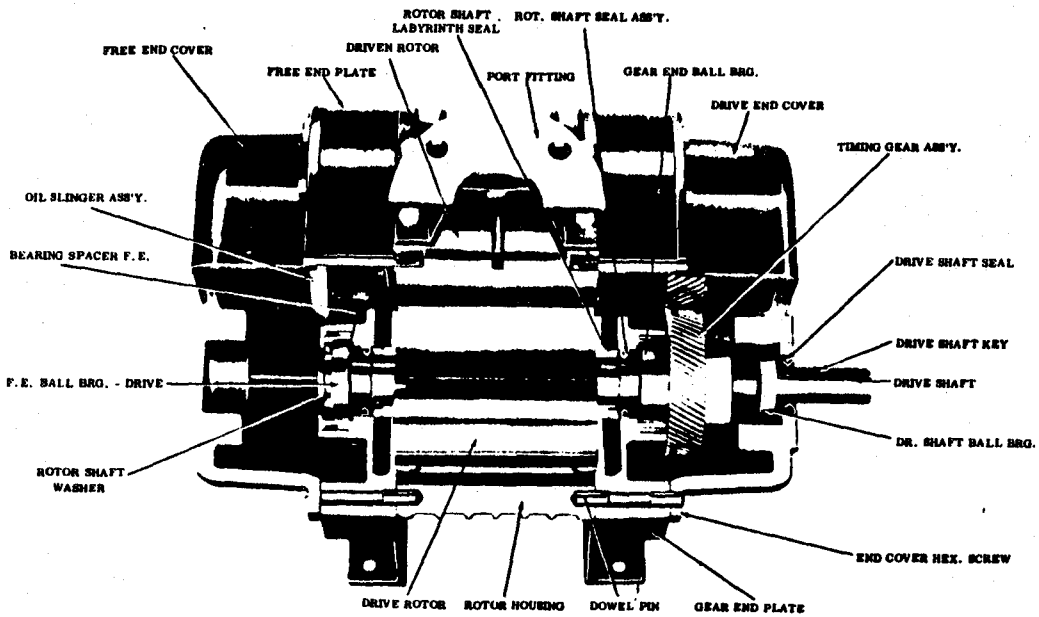
A temperature rise of 185°F ± will be experienced the piping at the blower and underground should be steel. Polyethylene pipe can be used where the pipe is under water.

TUTHILL CORPORATION | M-D Pneumatics Division

3210 PRESSURE SLIP CURVE

(NOTE: 16/47 SERIES LIMITED TO 10 PSIG) (.112 CFR DISPL.)





Material Specifications:

Housing — Cast Iron

End Plates — Cast Iron

End Covers — Aluminum, Free End — Cast Iron, Gear End.

Rotors Ductile Iron — Shafts Cast integrally

Drive Shaft — Steel

Gears — Heat Treated Alloy Steel Helical, Matched

Seals — Lip and Labyrinth Type

Lubrication — Oil Splash System — Both Ends

Optional

Auxiliary External Lube System

When External Lube System is used, the series numbers change as follows:

18 Series Horizontal Air Flow
85 Series Vertical Air Flow

Operating Specifications:

Check with sales rep or factory and instruction manual for actual operating conditions and operating recommendations.

17/46 Series Heavy Duty High Pressure Industrial Blowers

Bi-Directional Rotation

17 Series Horizontal Air Flow
46 Series Vertical Air Flow

This series has wide application in pneumatic conveying, gas handling and the general process industry where high pressure, high volume air is required. Vacuum applications may be used either dry or wet (water injection) for in-plant vacuum systems, machine vacuum pickup, and vacuum conveying, where high volume, low cost vacuum supplies are required.

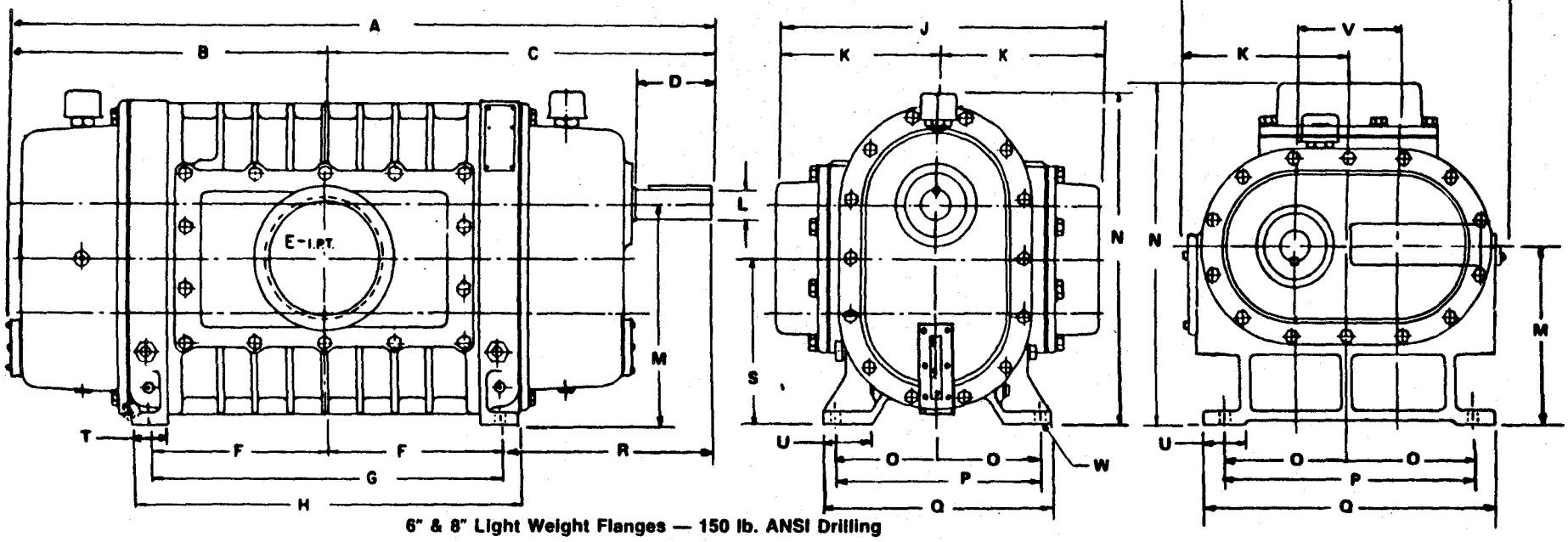
This series has a vented seal area. The labyrinth seal acts as a baffle seal to the air chamber. The lip seal is outboard of the vented area and acts as the oil seal.

MODEL SIZE	Nom Max Press PSI	Nom Max Vac In. Hg	Max Compr. Ratio	Nom Min RPM @ Max disch. press.	Nom Max RPM @ Max disch. press.	Displ. CFR
3202	15	15	2.02:1	2250	3600	.028
3204	15	15	2.02:1	1750	3600	.045
3206	15	15	2.02:1	1750	3600	.0675
3210	15	15	2.02:1	1750	3000	.112
4006	15	15	2.02:1	1750	3600	.102
4009	15	15	2.02:1	1750	3600	.152
4012	15	15	2.02:1	1750	3600	.203
5507	15	15	2.02:1	1750	3600	.238
5509	15	15	2.02:1	1750	3600	.286
5511	15	15	2.02:1	1750	3600	.350
5514	15	15	2.02:1	1750	3600	.445
5516	12	15	1.82:1	1150	3600	.509
5518	12	15	1.82:1	1150	3000	.572
5520	8	12	1.54:1	1150	3000	.636

TUTHILL CORPORATION | M-D Pneumatics Division
P.O. Box 2877, Springfield, MO 65801-0877
(417) 865-8715 TWX 910-775-4704
FAX 417-865-2950

SERIES 17

SERIES 47



6" & 8" Light Weight Flanges — 150 lb. ANSI Drilling

MODEL	SERIES	A	B	C	D	E	F	G	H	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	NET WT.
3202	17 46	17-11/16 17-1/16	6-9/16 5-13/16	11-1/8 11-1/8	2-13/32 2-13/32	1-1/2 1-1/2	2 2-1/16	4 4-1/8	5-1/8 5-1/4	8 10-11/16	4 5-17/32	.938 .937	6-3/4 5-1/2	11-1/2 9-1/2	3 3-3/4	6 7-1/2	7 9	9-1/8 9-1/16	5-1/8 ——	1-5/16 1-3/8	1-1/2 1-5/8	3-1/4 3-1/4	13/32 13/32	115
3204	17 46	19-3/16 18-9/16	7-5/16 6-11/16	11-7/8 11-7/8	2-13/32 2-13/32	2 2	2-3/4 2-13/16	5-1/2 5-5/8	6-5/8 6-3/4	10 10-11/16	5 5-17/32	.938 .937	6-3/4 5-1/2	11-1/2 10-1/2	3 3-3/4	6 7-1/2	7 9	9-1/8 9-1/16	5-1/8 ——	1-5/16 1-3/8	1-1/2 1-5/8	3-1/4 3-1/4	13/32 13/32	130
3206	17 46	21-3/16 20-9/16	8-5/16 7-11/16	12-7/8 12-7/8	2-13/32 2-13/32	2-1/2 2-1/2	3-3/4 3-13/16	7-1/2 7-5/8	8-5/8 8-3/4	10 10-11/16	5 5-17/32	.938 .937	6-3/4 5-1/2	11-1/2 10-1/2	3 3-3/4	6 7-1/2	7 9	9-1/8 9-1/16	5-1/8 ——	1-5/16 1-3/8	1-1/2 1-5/8	3-1/4 3-1/4	13/32 13/32	140
3210	17 46	25-3/16 24-9/16	10-5/16 9-11/16	14-7/8 14-7/8	2-13/32 2-13/32	3 3	5-3/4 5-13/16	11-1/2 11-5/8	12-5/8 12-3/4	11 10-11/16	5-1/2 5-17/32	.938 .937	6-3/4 5-1/2	11-1/2 11	3 3-3/4	6 7-1/2	7 9	9-1/8 9-1/16	5-1/8 ——	1-5/16 1-3/8	1-1/2 1-5/8	3-1/4 3-1/4	13/32 13/32	200
4006	17 46	23-3/16 22-19/32	10-3/8 9-25/32	12-13/16 12-13/16	3 3	3 3	4-1/2 4-1/4	9 8-1/2	10-1/2 10-1/8	12-1/2 12-7/16	4-1/4 6-13/32	1.125 1.124	8-1/4 8	12-1/4 14-1/4	4 5-1/4	8 10-1/2	9 12	8-5/16 8-9/16	6-1/4 ——	2-1/4 2-1/16	2-1/8 2-1/2	4 4	13/32 17/32	135
4009	17 46	26-3/16 25-19/32	11-7/8 11-9/32	14-5/16 14-5/16	3 3	3 3	6 5-3/4	12 11-1/2	13-1/2 13-1/8	12-1/2 12-7/16	6-1/4 6-13/32	1.125 1.124	8-1/4 8	12-1/4 14-1/4	4 5-1/4	8 10-1/2	9 12	8-5/16 8-9/16	6-1/4 ——	2-1/4 2-1/16	2-1/8 2-1/2	4 4	13/32 17/32	150
4012	17 46	29-3/16 28-19/32	13-3/8 12-25/32	15-13/16 15-13/16	3 3	4 4	7-1/2 7-1/4	15 14-1/2	16-1/2 16-1/8	12-1/2 12-7/16	6-1/4 6-13/32	1.125 1.124	8-1/4 8	12-1/4 14-1/4	4 5-1/4	8 10-1/2	9 12	8-5/16 8-9/16	6-1/4 ——	2-1/4 2-1/16	2-1/8 2-1/2	4 4	13/32 17/32	185
5507	17 46	28-15/16 28-5/16	12-5/16 11-11/16	16-5/8 16-5/8	5 5	4 4	5-3/8 5-1/4	10-3/4 10-1/2	13 12-1/2	17 17	8-1/2 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 19	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 11-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	365
5509	17 46	30-7/16 29-13/16	13-1/16 12-7/16	17-3/8 17-3/8	5 5	5 5	6-1/8 6	12-1/4 12	14-1/2 14	17 17	8-1/2 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 19	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 11-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	400
5511	17 46	32-7/16 31-13/16	14-1/16 13-7/16	18-3/8 18-3/8	5 5	5 5	7-1/8 7	14-1/4 14	16-1/2 16	17 17	8-1/2 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 19	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 17-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	450
5514	17 46	35-7/16 34-13/16	15-9/16 14-15/16	19-7/8 19-7/8	5 5	6 flg. 6 flg.	8-5/8 8-1/2	17-1/4 17	19-1/2 19	20 17	10 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 20-1/2	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 11-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	530
5516	17 46	37-7/16 36-13/16	16-9/16 15-15/16	20-7/8 20-7/8	5 5	8 flg. 8 flg.	9-5/8 9-1/2	19-1/4 19	21-1/2 21	20 17	10 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 20-1/2	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 11-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	575
5518	17 46	39-7/16 38-13/16	17-9/16 16-15/16	21-7/8 21-7/8	5 5	8 flg. 8 flg.	10-5/8 10-1/2	21-1/4 21	23-1/2 23	20 17	10 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 20-1/2	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 11-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	600
5520	17 46	41-7/16 40-13/16	18-9/16 17-15/16	22-7/8 22-7/8	5 5	8 flg. 8 flg.	11-5/8 11-1/2	23-1/4 23	25-1/2 25	20 17	10 8-21/32	1.750 1.749	11-1/8 10-1/2	16-3/8 20-1/2	5-1/2 7-1/4	11 14-1/2	12-1/4 16	11-1/4 11-3/8	8-3/8 ——	2-3/4 2-1/2	2-1/2 2-1/2	5-1/2 5-1/2	21/32 21/32	650

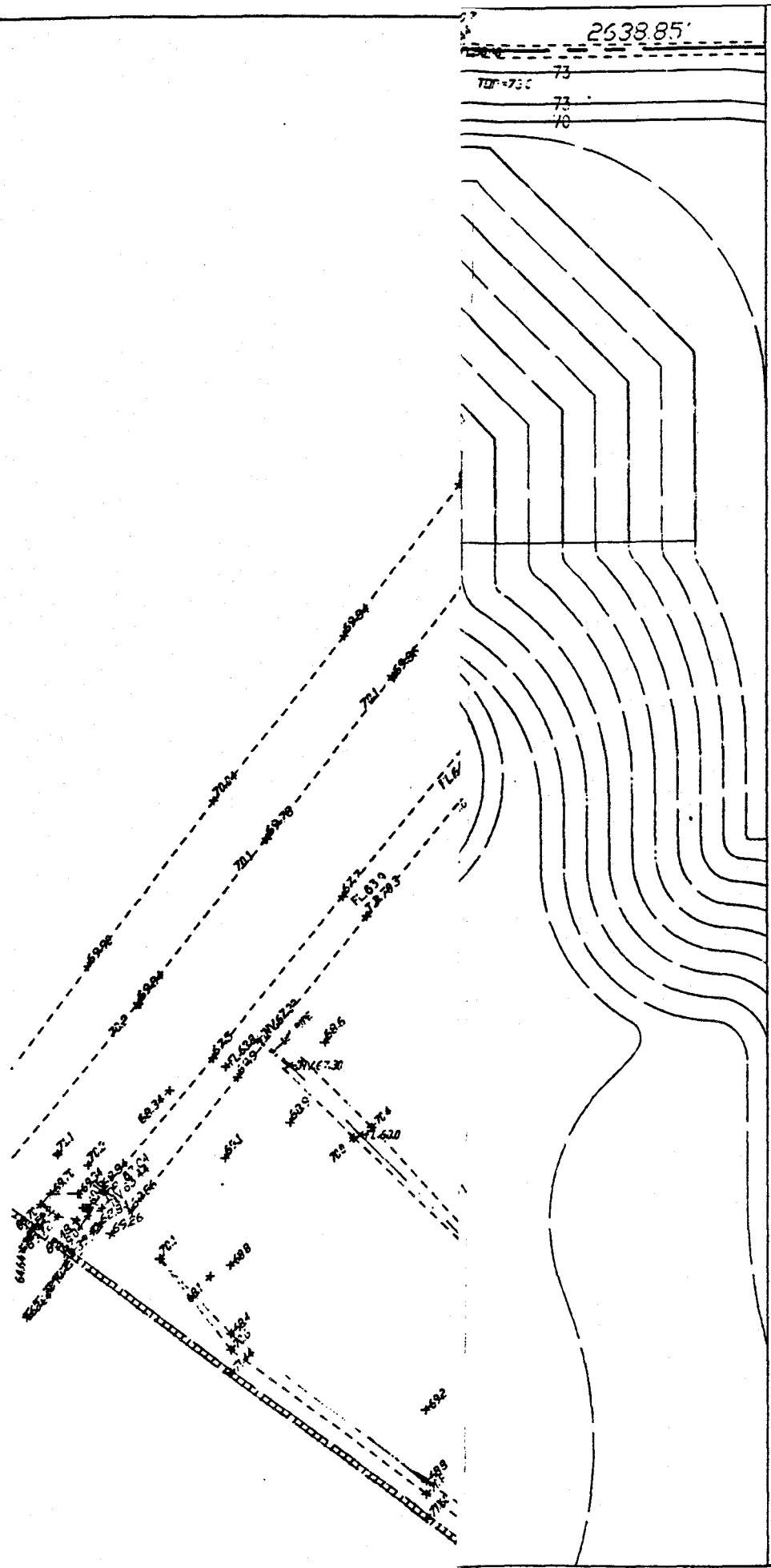
NOTE: Drive Shaft location is optional:
On 16 Series Top Drive is standard while Bottom Drive is Special
On 47 Series Left Drive is standard while Right Drive is Special

1/87

M-D PNEUMATICS, INC.

35

Scale
1" = 80'



BOWLEY CONSULTANTS INC		FLOOD CONTROL DISTRICT OF MARICOPA COUNTY
DESIGN	BY	DATE
DESIGN CHK		CROSSROADS PARK

Job No. 10924 Page No. 37

Computed by G J Fink Date 3/14/91
Checked by BDR Date 3/21/91
Reviewed by Date
Approved by BDR Date 3/21/91

Subject Pump Station Intake Bar Screen
Sheet No. 1 of 2

Design Parameters:

1. Max. flow = 5,000 gpm (11.14 ft³/sec)
2. Max. opening = 2" between bars
3. Assume 90% plugging factor
4. Limit velocity thru bars @ 2 ft/sec maximum.

Determine Minimum Net Area

$$A = \frac{Q}{V} = \frac{11.14\ ft^3/sec}{2\ ft/sec} = 5.57\ ft^2$$

Determine Minmum Open Area

$$\text{Open Area} = 10 \times 5.57\ ft^2 = 55.7\ ft^2$$

Determine Minimum Gross Area

Assume Bar thickness = 1/2"

$$\%\ \text{Open Area} = \frac{2''}{2\frac{1}{2}''} = 0.8$$

$$\text{Minimum Gross Area} = \frac{55.7}{0.8} = 69.62\ ft^2$$

Determine Minimum Length of Intake Screen.

Refer to "Typical Section" on sht. No. 2

$$\text{Length} = \frac{\text{Area}}{\text{wetted width}}$$

$$= \frac{69.62\ ft^2}{1.414 \times 3'} = 16.4'$$

Use 20'-0"

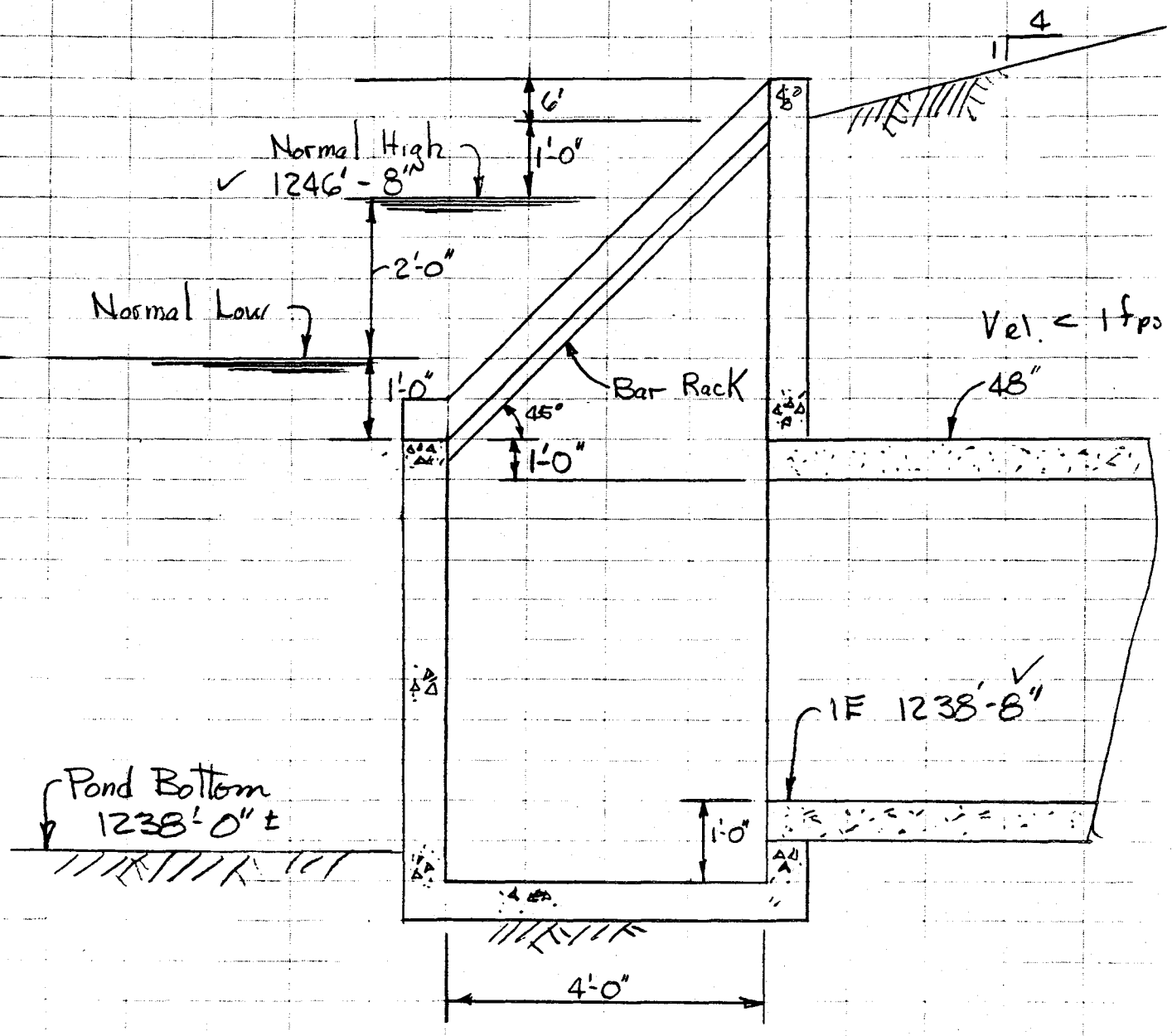
This must be confirmed after the details of the bars and spacing are designed.

Job No. 10924 Page No. 38

Computed by G J Fink Date 3-14-91
Checked by BDR Date 3/21/91
Reviewed by Date
Approved by BDR Date 3/21/91

Subject Pump Station Intake Bar Screen.

Sheet No. 2 of 2

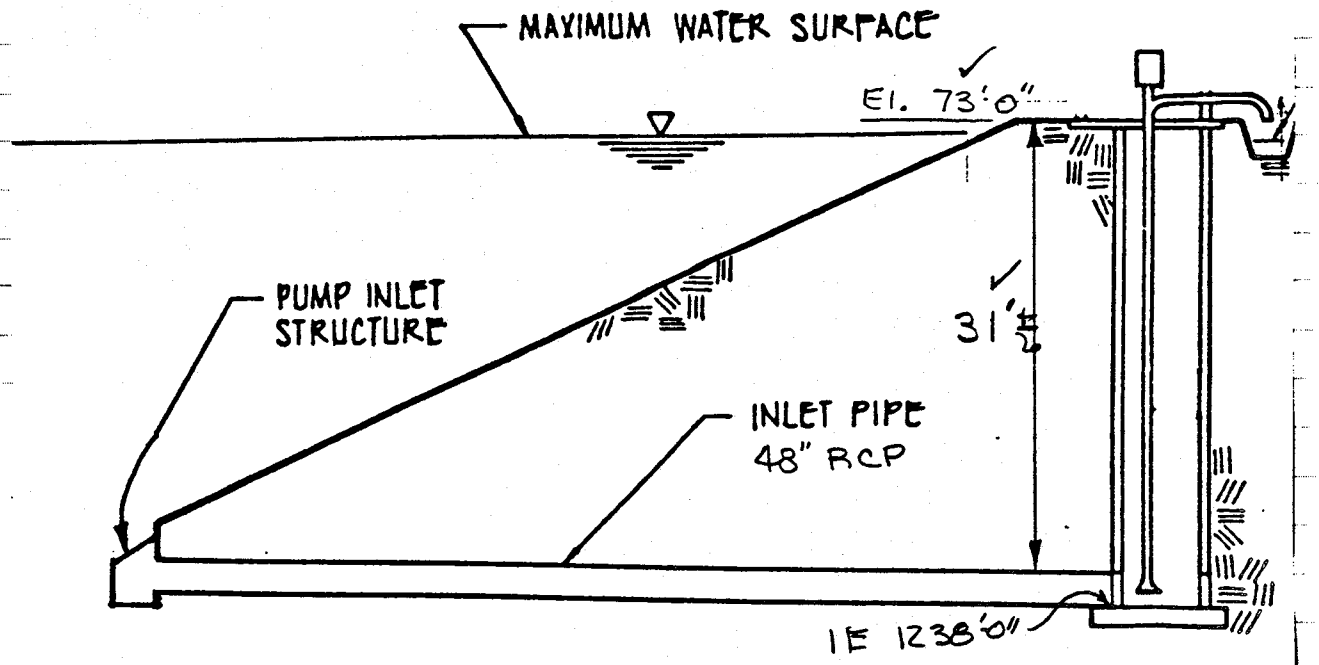


Scale 1/2" = 1'-0"

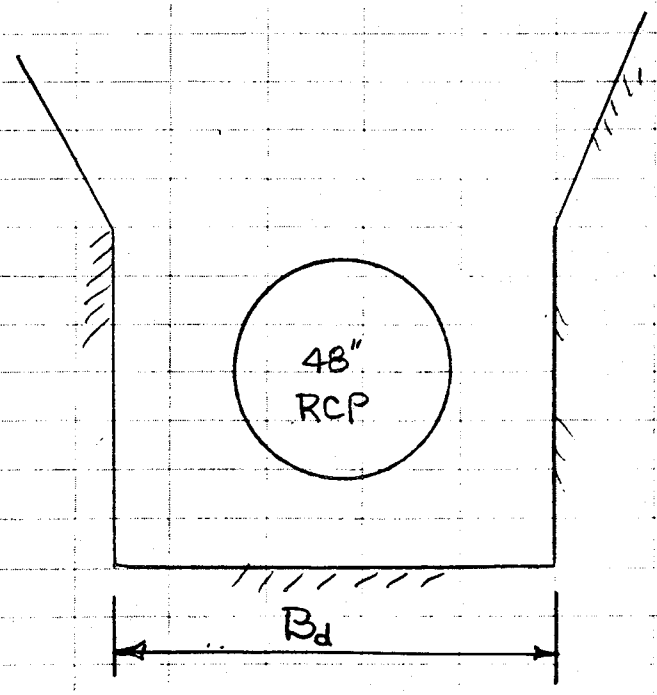
Computed by G J Fink Date 3-14-91
Checked by BDR Date 4-26-91
Reviewed by Date
Approved by BDR Date 4-26-91

Job No. 10924 Page No. 39
Subject Pump Station Intake Pipe
Load on Pipe
Pipe Bedding
Sheet No. 1 of 6

Typical Section



CROSS-SECTION



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Checked by BOR Date 4-26-91
Reviewed by Date
Approved by BOR Date 4-26-91

Job No. 10924 Page No. 40
Subject Pump Station Intake Pipe
Load on Pipe
Pipe Bedding
Sheet No. 2 of 6

Load on Pipe

Refer to SCI Civil Design Memo 135.206-R3 for "Loads on sewer and water pipe"

See attached Table 1-B

Refer to SCI Civil Design Memo 135.207-R2 "Supporting Strength of Sewer and Water Pipe"
See Tables 1-A thru 1-E (includes 1.5 safety factor)

For 48" RCP

Bedding	Safe Supporting Strength, lb/LF II	III	IV	V
A.	13,600	18,133	27,200	34,000
A+	11,200	14,933	22,400	28,000
A*	19,200	25,600	38,400	48,000
B	7,600	10,133	15,200	19,000
C	6,000	8,000	12,000	15,000

For this project use a Class IV RCP for all 48" inlet pipe.

- Use Class A* Bedding next to pump station (See Attached CS-9)
- Use Class B Bedding for remainder. (See Attached CS-10.)

Class A+ Bedding shall be provided for cover above pipe of 23 feet and greater. Limit maximum Bd to 7.0'.

Computed by G J Fmk Date 3-14-91
Checked by BDR Date 4-26-91
Reviewed by Date
Approved by BDR Date 4-26-91

Job No. 10924 Page No. 41
Subject Pump Station Intake Pipe
Load On Pipe
Pipe Bedding
Sheet No. 3 of 6

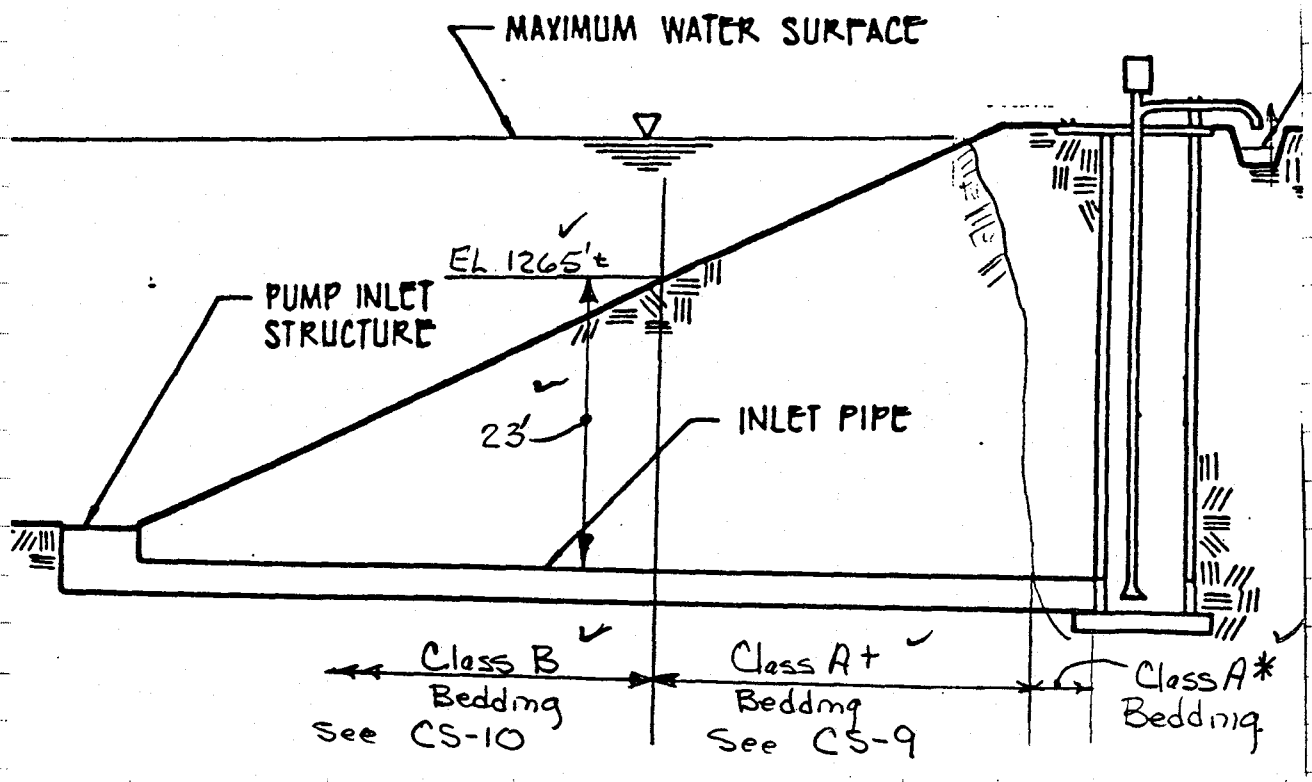


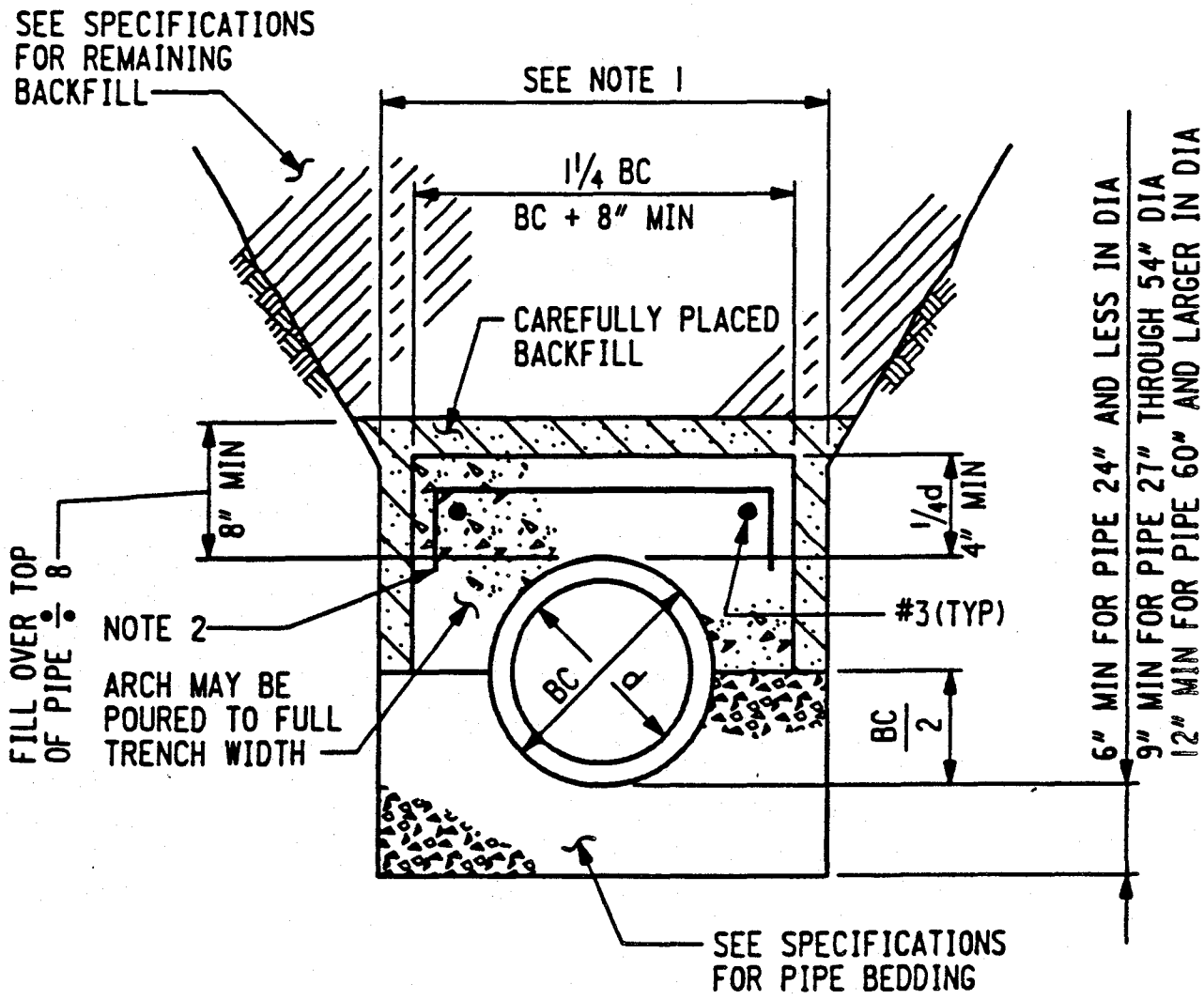
TABLE: I-B (cont.)
LOADS ON RIGID PIPES FOR
SATURATED CLAY

$$W_c\ (lb/ft) = C_d\ w\ (B_d)^2 ; \quad C_d = \frac{1 - \exp(-2ku'H/B_d)}{2ku'}$$

where w = 130 lb/cu ft
ku' = 0.110

Height of fill over pipe, H (ft)	Trench Width at Top of Pipe, Bd (ft)									
	5.00	5.50	6.00	6.50	7.00	7.50	8.00	8.50	9.00	10.00
1	636	701	766	831	896	961	1026	1091	1156	1286
2	1244	1374	1504	1634	1764	1894	2024	2154	2284	2544
3	1827	2021	2216	2411	2605	2800	2995	3190	3384	3774
4	2384	2643	2902	3161	3420	3680	3939	4199	4458	4978
5	2917	3240	3563	3887	4211	4534	4858	5182	5507	6155
6	3428	3814	4201	4588	4976	5364	5752	6141	6529	7307
7	3916	4365	4816	5267	5718	6170	6622	7075	7528	8434
8	4383	4895	5408	5922	6437	6952	7468	7985	8502	9536
9	4831	5404	5979	6556	7134	7712	8292	8872	9452	10615
10	5259	5893	6530	7169	7809	8450	9093	9736	10380	11669
11	5668	6363	7061	7761	8463	9167	9872	10578	11285	12701
12	6060	6814	7572	8333	9097	9863	10630	11398	12168	13711
13	6435	7248	8066	8887	9711	10538	11367	12198	13030	14698
14	6794	7665	8541	9422	10307	11195	12085	12977	13871	15664
15	7137	8065	8999	9939	10884	11832	12783	13736	14692	16609
16	7466	8450	9441	10439	11443	12451	13462	14476	15493	17533
17	7781	8819	9867	10923	11985	13051	14123	15197	16275	18438
18	8082	9174	10278	11390	12510	13635	14765	15900	17038	19322
19	8370	9515	10674	11842	13018	14202	15391	16584	17782	20188
20	8645	9843	11055	12279	13512	14752	15999	17251	18509	21034
21	8909	10155	11423	12701	13989	15286	16591	17901	19217	21862
22	9161	10461	11778	13109	14452	15805	17167	18535	19909	22672
23	9403	10751	12120	13504	14901	16309	17727	19152	20584	23465
24	9634	11031	12449	13885	15336	16799	18272	19754	21243	24240
25	9855	11299	12767	14254	15757	17274	18802	20340	21886	24998
26	10067	11557	13073	14610	16166	17735	19318	20911	22513	25740
27	10270	11805	13368	14955	16561	18184	19820	21467	23125	26466
28	10463	12043	13653	15288	16945	18619	20309	22010	23723	27176
29	10649	12271	13927	15610	17316	19041	20783	22538	24306	27870
30	10826	12491	14192	15922	17676	19452	21245	23053	24874	28550

Most probable range of loadings to be expected. Need to Limit Bd ≤ 7.00'



TYPICAL TRENCH CROSS-SECTION

NOTES:

1. FOR ALLOWABLE TRENCH WIDTH SEE SPECIFICATIONS.
2. AREA TRANSVERSE STEEL:
   CLASS A = 0.004 X AREA CONCRETE ARCH AT CROWN; 12" MAXIMUM SPACING.
   CLASS A* = 0.010 X AREA CONCRETE ARCH AT CROWN; 12" MAXIMUM SPACING.
   CLASS A+ = NO TRANSVERSE STEEL REQUIRED.
3. MAINTAIN ARCH THICKNESS OVER BELL AND BARREL.
4. MINIMUM CONCRETE COMPRESSIVE STRENGTH: 2,500 PSI.
5. CAREFULLY PLACED BACKFILL SHALL BE FINELY DIVIDED MATERIAL FREE FROM DEBRIS OR STONES MEETING OR EXCEEDING SPECIFICATIONS FOR REMAINING BACKFILL.

REVISIONS	DWN.	APP.	APP.	DATE
3 NOTE 2	JDO		CLM	12/12/86
2 BED DEPTH	JBP		WES	6/2/77
1 GRAPHICS	JBP		WES	2/18/76

DESIGNED
DRAWN L.S.
CHECKED R.G.P.
APPROVED CLM
APPROVED FWR
DATE OCT 27, 1967

CIVIL STANDARD SANITARY

CLASS "A" PIPE BEDDING

SCALE NONE

NO. CS-9 REV. 3

CADD A1 R1

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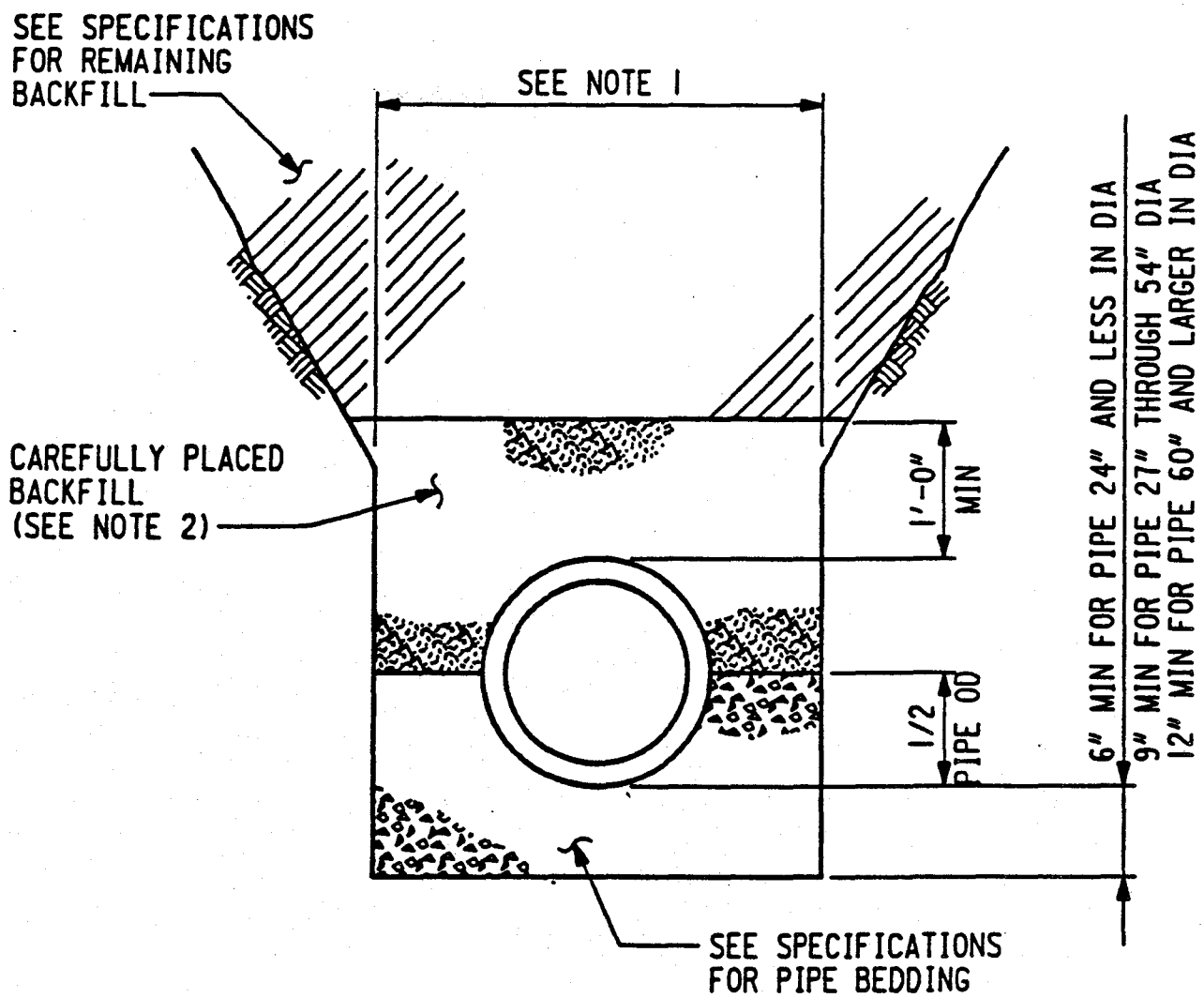

TYPICAL TRENCH CROSS-SECTION

NOTES:

1. FOR ALLOWABLE TRENCH WIDTH SEE SPECIFICATIONS.
2. CAREFULLY PLACED BACKFILL SHALL BE CAREFULLY COMPACTED, FINELY DIVIDED, JOB EXCAVATED MATERIAL, FREE FROM DEBRIS, STONES LARGER THAN ½", ORGANIC MATTER AND FROZEN MATERIAL, COMPACTED TO 90% MAXIMUM DENSITY, ASTM D1557. SELECT BACKFILL MAY BE SUBSTITUTED FOR JOB EXCAVATED MATERIAL AND SHALL BE USED WHERE REMAINING BACKFILL IS SPECIFIED TO BE SELECT BACKFILL MATERIAL.

REVISIONS	DWN.	APP.	APP.	DATE
5 NOTE 2	RJH		ELM	7/6/84
4 NOTE 2	LJO		CLM	1/4/83
3 COMPACTION	GAG		CLM	10/9/81
2 BED DEPTH	JBP		WES	6/2/77
1 REDRAWN	RTD		WES	2/18/76

DESIGNED RLL
DRAWN LJT
CHECKED RLL
APPROVED CLM
APPROVED FWR
DATE OCT 27, 1967

CIVIL STANDARD SANITARY

CLASS "B" PIPE BEDDING

SCALE NONE

NO. CS-10

REV. 5

CADD A1 R1

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Computed by Bennett R Date Mar. 5 1991
Checked by I McAlister Date 10 May 91
Reviewed by I. McAlister Date 10 May 91
Approved by ..... Date .....

Job No. 10924 Page No. 45
Subject Basin Inlets
Location & Capacity
Sheet No. 1 of 4

Determine Required Capacity of Stormwater inlet structures into detention basin.

Location of Structures:
Use Flood Insc Study, USGS map and site visits to select locations. Use 3 main inlet structures at locations shown on sheet 3.

Capacity of Structures
Following based on HEC-1 runs provided by Gilbert and run on model used for flood insc study. See sheet 4 for HEC-1 printout.
Design basin inlets to pass 100 year flood flow into basin without damage to sideslopes.
Basin is sized for 450 Ac-ft storage volume which is 34% of runoff from a 100 year-24 hour storm, (see sheet 4).
Some of the runoff from area 1 will overflow Eastern Canal and bypass the detention basin, especialy during a major storm. All runoff from areas 2 and 3 will pass through the basin.

Inlet Structure 1 Q_{100}
From sheet 4 the peak flow = 1553 cfs ✓
from Flood Insc Study map flow over Eastern Canal = 603 cfs ✓

Inlet Structure #1 Required Capacity = 950 cfs ✓

3004

Computed by Bennett R Date Mar 5 1991
Checked by I. McAlister Date 10 May 91
Reviewed by I. McAlister Date 10 May 91
Approved by ..... Date .....

Job No. 10924 Page No. 46
Subject Basin Inlets
Sheet No. 2 of 4

Capacity of Inlets (cont.)

Inlet Structure No 2

This structure was eliminated due to laser leveling of adjacent field. BDR 3-6-91

Q_{100}

from sheet 4 - peak flow : 146 cfs ✓

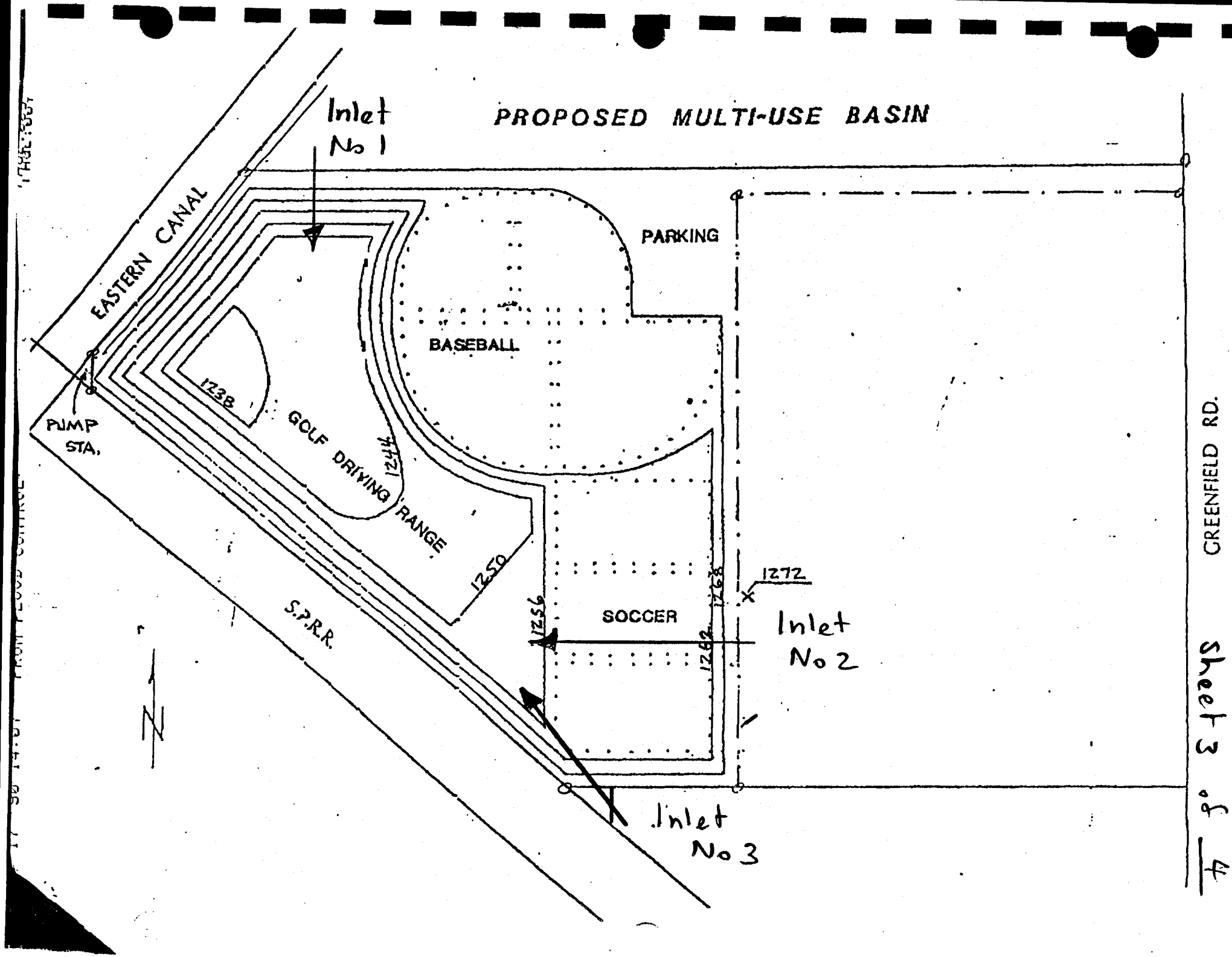
This flow rate includes rainfall directly on the basin as well as rainfall on the ball fields, parking lots etc which will get into basin without passing through Inlet No 2. Actual capacity of Inlet No 2 to be determined based on final grading plan at the site.

Inlet Structure No 3

Q_{100}

from sheet 4 - peak flow = 500 cfs ✓

3004



Bennett K 2-3-91

Sheet 3 of 4

10724

(47)

Bennett R 3-5-91 IV924 (4c

sheet 4 of 4

RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	SUBA	222.	13.42	102.	32.	11.	0.52		
ROUTED TO	0TO1	153.	14.33	100.	32.	11.	0.52	1.68	14.33
HYDROGRAPH AT	SUBB	125.	12.83	42.	13.	4.	0.21		
2 COMBINED AT	PT1a	206.	13.17	139.	45.	15.	0.73		
ROUTED TO	PT1b	206.	13.25	135.	42.	14.	0.73	1301.26	13.25
DIVERSION TO	DIV1	29.	13.25	17.	5.	2.	0.73		
HYDROGRAPH AT	PT1c	176.	13.25	118.	38.	13.	0.73		
ROUTED TO	1TO2	163.	14.33	116.	38.	13.	0.73	1.71	14.33
HYDROGRAPH AT	SUBC	262.	14.75	174.	60.	20.	0.97		
2 COMBINED AT	PT2	422.	14.67	290.	98.	33.	1.70		
ROUTED TO	2TO3	404.	15.17	284.	98.	33.	1.70	2.24	15.17
HYDROGRAPH AT	SUBD	260.	13.67	130.	42.	14.	0.67		
2 COMBINED AT	PT3	567.	14.42	403.	139.	47.	2.37		
ROUTED TO	3TO4	490.	15.67	378.	139.	47.	2.37	2.37	15.67
HYDROGRAPH AT	SUBE	136.	12.75	44.	14.	5.	0.22		
HYDROGRAPH AT	SUBF	138.	13.00	49.	16.	5.	0.25		
3 COMBINED AT	PT4	531.	15.50	437.	168.	56.	2.84		
ROUTED TO	PT4	530.	15.58	423.	152.	51.	2.84	80.04	15.58
ROUTED TO	4TO5	500.	16.50	401.	152.	51.	2.84	2.38	16.50
HYDROGRAPH AT	SA10a	1553.	14.50	1085.	511.	172.	3.16		
HYDROGRAPH AT	SUBG	146.	12.50	36.	11.	4.	0.18		
2 COMBINED AT	SA10b	1570.	14.50	1115.	521.	176.	3.34		
2 COMBINED AT	PT5a	1906.	15.00	1446.	673 = 1335	227.	6.18		
DIVERSION TO	BASIN	1906.	15.75	897.	227. = 450	76.	6.18		
HYDROGRAPH AT	DUMMY	1800.	15.75	1037.	453.	151.	6.18		
ROUTED TO	PT5b	963.	18.50	746.	393.	151.	6.18	1271.13	18.50
DIVERSION TO	DIV5c	736.	18.50	521.	188.	63.	6.18		
HYDROGRAPH AT	PT5c	227.	18.50	225.	205.	88.	6.18		
DIVERSION TO	DIV5d	227.	0.08	225.	205.	88.	6.18		
HYDROGRAPH AT	PT5d	0.	0.08	0.	0.	0.	6.18		
HYDROGRAPH AT	PT5e	227.	18.50	225.	205.	88.	0.00		
HYDROGRAPH AT	PT5f	29.	13.25	17.	5.	2.	0.00		
HYDROGRAPH AT	PTWr1	1546.	14.75	1237.	595.	365.	11.21		
DIVERSION TO	DIVWr2	603.	13.58	308.	77.	26.	11.21		
HYDROGRAPH AT	PTWr2	943.	13.58	929.	518.	339.	11.21		
ROUTED TO	WrTORR	943.	16.17	923.	518.	339.	11.21	1273.00	16.67
HYDROGRAPH AT	PTRR1a	736.	18.50	521.	188.	63.	0.00		
2 COMBINED AT	PTRR1b	1679.	18.50	1259.	706.	402.	11.21		
DIVERSION TO	DIVRR1	1029.	13.67	609.	208.	69.	11.21		
HYDROGRAPH AT	PTRR1c	650.	13.67	650.	498.	333.	11.21		
HYDROGRAPH AT	PTWr3	603.	14.75	308.	77.	26.	0.00		
HYDROGRAPH AT	PTRR2	1029.	18.50	609.	208.	69.	0.00		

Inlet # 3 ✓
Inlet # 1 ✓
Inlet # 2 ✓

$\frac{450}{1335} = 34\%$ ✓

Acre feet.

*** NORMAL END OF HEC-1 ***

$\left(\frac{ft^3}{sec}\ 24hr\right) \frac{Ac}{43560 ft^2} \frac{3600\ sec}{hr} = Ac\ ft$ ✓

Computed by: I. McAlister Date: 8 Mar 91
Checked by: Date:
Reviewed by: Date:
Approved by: BDR Date: 5/10/91

Job No. 10924 Page No. 49
Subject: Cross Roads Detention Basin Inlets
Inlet #1 - North Inlet
Sheet No. 1 of 1

North Inlet - Inlet #1 - HYDRAULICS

Design Flow 950 cfs ✓

from "Handbook of Hydraulics" King & Brater.

Broad crested weir $W \geq 15'$ ✓

for					
$d \Rightarrow$	2'	1.4'	1.0'	0.6'	0.4'
$C \Rightarrow$	2.63	2.64	2.63	2.7	2.7

Use $C = 2.65$ ✓

Weir crest Elevation: 1270.0 ✓
100yr Flood Elevation: 1271.4 from flood Insc study

$\therefore d = 1.4'$

By weir formula.

$$Q = C L H^{3/2}$$ ✓

$$\Rightarrow L = \frac{Q}{C H^{3/2}} = \frac{950}{2.65 \times 1.4^{3/2}}$$ ✓

$$L = 216'$$

$$\boxed{\text{Use } L = 220'}$$

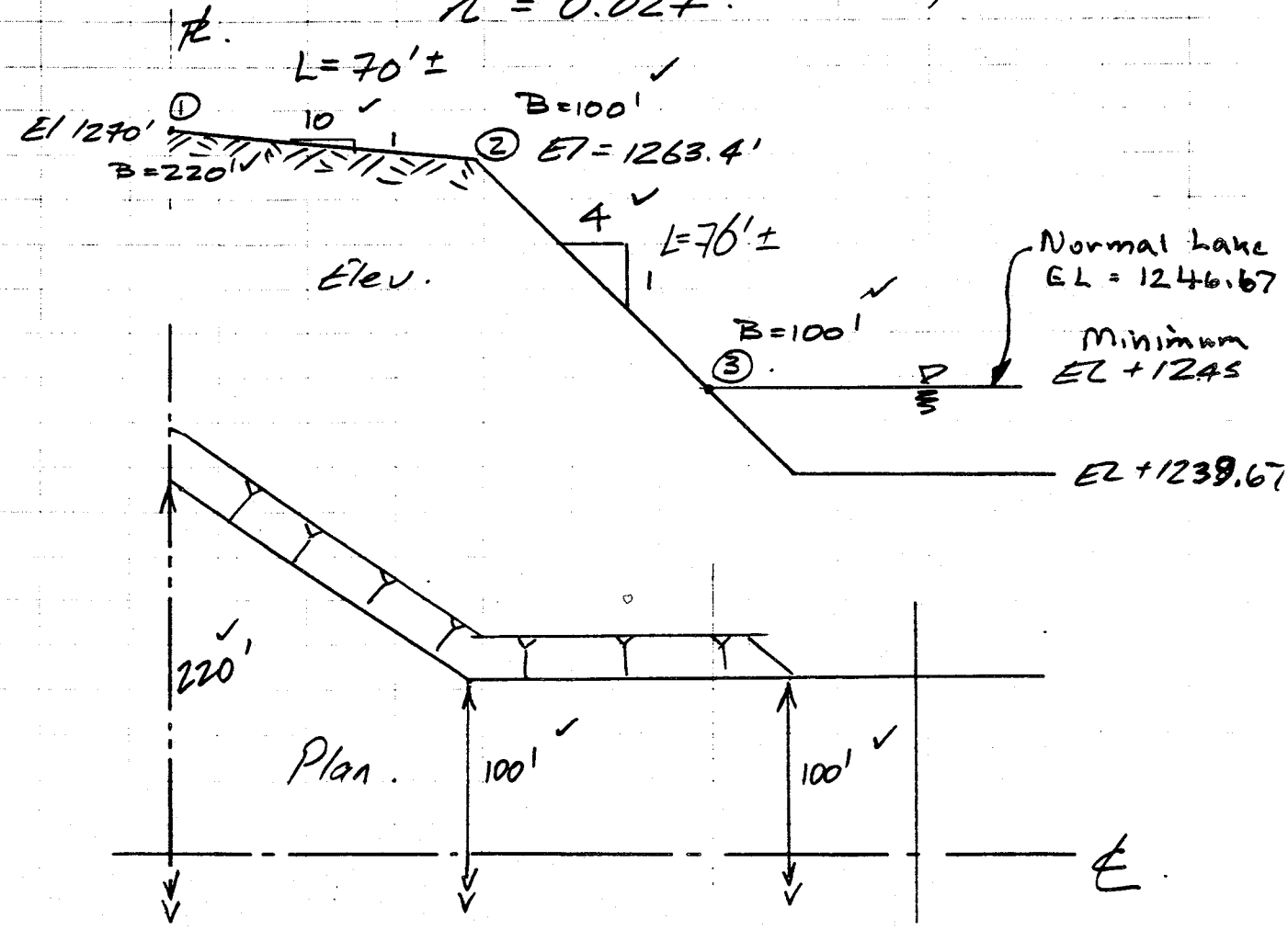
Computed by: J. McAlister Date: 8 Mar 91
Checked by: Date:
Reviewed by: Date:
Approved by: BDR Date: 5/10/91

Job No. 10924 Page No. 50
Subject: Crossroads Detention Basin – Inlet Hydraulic
Inlet #1 – North Inlet
Sheet No. 1 of 7

INLET # 1

Given the opening width (B=220') at the inlet headworks, energy principals are used to determine the depth of flow and flow velocities in the grouted riprap channel.

Criteria: Design Flow 950 cfs.
Headworks Invert Elev. 1270'
Depth Flow at Headworks 1.4'
B = 220' headworks.
B = 100', main spillway.
n = 0.027.



Computed by I. McAlister Date 8 Mar 91
Checked by Date
Reviewed by Date
Approved by BDR Date 5/10/91

Job No. 10924 Page No. 51
Subject Crossroads Detention Basin Inlets. Inlet #1.
Sheet No. 2 of 7

At Point ①

$$E_① = z_① + y_① + \frac{q_①^2}{2g y_①^2}$$

$$E_① = 1270 + 1.4 + \frac{4.32^2}{64.4 \times 1.4^2}$$

$z_① = 1270'$ ✓
$y_① = 1.4'$ ✓
$q_① = Q/B_①$ ✓
$= \frac{950}{220} = 4.3$ cfs/

Energy @ ① = 1271.55' ✓

At Point ②

$$E_② = E_① - h_{f①\to②}$$

$$z_② + y_② + \frac{q_②^2}{2g y_②^2} = 1271.55 - \frac{0.078 \times 70 \times 5.9^2}{8 \times 32.2 \times 1^3}$$

$$1263.4 + y_② + \frac{9.5^2}{64.4 y_②^2} = 1271.55 - 0.75$$

$$y_② + \frac{1.401}{y_②^2} = 7.40$$

$$\boxed{y_② \doteq 0.45'}$$

h_f = head loss $= \frac{fL}{8g} \frac{\bar{q}^2}{\bar{y}^3}$

f = friction $= 0.113 \left(\frac{\bar{q}}{\bar{y}}\right)^{\frac{1}{3}}$

Eqn 4.21. Henderson "Open Channel Flow"
riprap projection
$f = 0.113 \left(\frac{.33}{1}\right)^{\frac{1}{3}}$ Ass.
$f = 0.078$

$\bar{q}$ = mid pt q
$= 950/160$
$= 5.9$ cfs/ft

$z_② = 1263.4$

$q_② = 950/100 = 9.5$

y_2	b
	6.1
.45	2.37

Checking assumption $\bar{y} = 1$ ft

$$\frac{y_① + y_②}{2} = \frac{1.4 + 0.45}{2} = \frac{1.85}{2}$$

$\bar{y} = 0.93 \simeq 1.'$ OK. ✓

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Checking $y_{②} = 0.45 \geq y_{normal}$ at ②.

By Manning.

$$q_{②} = \frac{1.49}{n} y_{n_{②}}^{5/3} S_o^{\frac{1}{2}}.$$

$$\Rightarrow y_{n_{②}} = \left(\frac{q_{②}\, n}{1.49\, S_o^{\frac{1}{2}}}\right)^{0.6}$$

$$= \left(\frac{9.5 \times 0.027}{1.49 \times 0.1^{\frac{1}{2}}}\right)^{0.6}$$

$$y_{n_{②}} = 0.69'$$

$q_{②} = 9.5$ cfs/ft ✓
$n = 0.027$ ✓
$S_o = 0.1$ ✓

$y_{n_{②}} > y_{②}$.

Flow depth cannot ~~exceed~~ be less than normal depth; therefore normal depth applies.

$$\boxed{y_{②} \doteq 0.7'}$$

At point ③

$$E_{③} = E_{②} - h_{f\,②\to③}$$

$$z_{③} + y_{③} + \frac{q_{③}^2}{2g\,y_{③}^2} = z_{②} + y_{②} + \frac{q_{②}^2}{2g\,y_{②}^2} - h_f$$

$$1245 + y_{③} + \frac{1.401}{y_{③}^2} = 1263.4 + 0.7 + \frac{1.401}{.7^2} - 6.83$$

$$y_{③} + \frac{1.401}{y_{③}^2} = 15.13'$$

$$\Rightarrow \underline{y_{③} = 0.30'}$$

$$\Rightarrow \bar{y} = \frac{0.30 + 0.7}{2} = 0.50$$ lower than assumed.

with $\bar{y} = 0.6 \Rightarrow f = 0.093 \Rightarrow h_L = 11.4'$

$$= y_{③} + \frac{1.401}{y_{③}^2} = 7.69$$

$$\Rightarrow \boxed{y_{③} = 0.44}$$

$$\bar{y} = \frac{0.44 + 0.7}{2} = 0.57 \simeq 0.6 \text{ OK}$$

Assume $\bar{y} = 0.7$

$$f = 0.113\left(\frac{d}{\bar{y}}\right)^{\frac{1}{3}}$$

$$= 0.113 \times \left(\frac{.33}{.7}\right)^{\frac{1}{3}}$$

$f = 0.088$

$$h_{f\,②-③} = \frac{f L q_{③}^2}{8g\,\bar{y}^3}$$

$$= \frac{.088 \times 76 \times 9.5}{8 \times 32.2 \times .07^3}$$

$h_f = 6.83'$

$z_{③} = 1245'$

$z_{②} = 1263.4'$

$y_{②} = 0.7'$

$q_{②} = q_{③} = 9.5\,cfs/ft$

Checking normal depth

$$y_{n③} = \left(\frac{q_{③}\,n}{1.49\,S_{o③}^{\frac{1}{2}}}\right)^{0.6}$$

$S_{o③} = 0.25$

$q_{③} = 9.5$

$n = 0.027$

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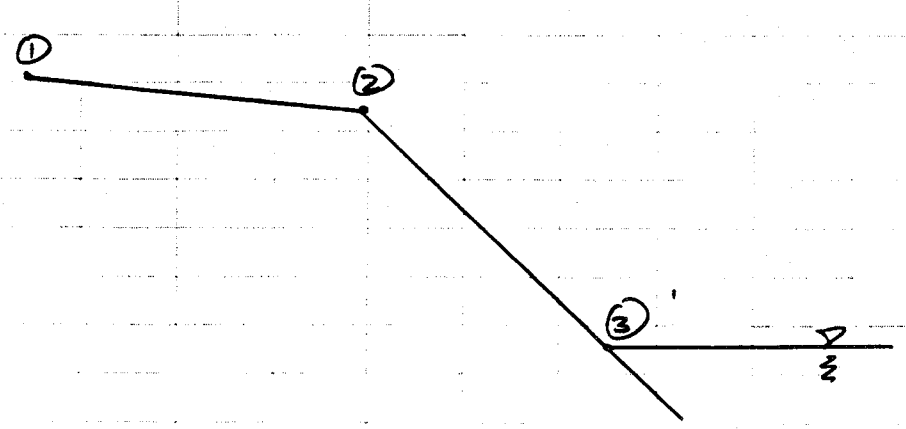
$$y_{n③} = \left(\frac{9.5 \times 0.027}{1.49 \times 0.25^{\frac{1}{2}}}\right)^{0.6} = 0.53'$$

$y_{n③} > y_{③}$ ∴ Normal depth controls again

$$\boxed{y_{③} = 0.53'}$$

Summary for Inlet #1



Point	Depth (ft)	q (cfs/ft)	V = q/y (fps)
①	1.4 ✓	4.3 ✓	3.1 ✓
②	0.7 ✓	9.5 ✓	13.5 ✓
③	0.53 ✓	9.5 ✓	17.9 ✓

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Inlet Hydraulics
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Evaluate need for Stilling Basin

Depth and velocity of spillway flow at ③

$d = 0.53'$ ✓
$V = 18$ fps. ✓

$$\text{Froud \# @ ③} = \frac{V}{\sqrt{gd}} = \frac{18}{\sqrt{32.2 \times 0.53}} = 4.3$$

From Fig 264 "Design Small Dams" USBR, attached.

Required depth to form jump
$\simeq 3.1$ ft.

Now Pool elevation in Basin (permanent)
1245 ft. Min pool 1240 ft ±

Basin Floor @ ~~1238 ft~~

∴ Depth H_2O in Basin (permanent) = 7'

Normal pool depth can vary between 5' and 7' - use 5' min depth

~~7'~~ 5' > 3.1'
available required

∴ No need for Stilling Basin ✓

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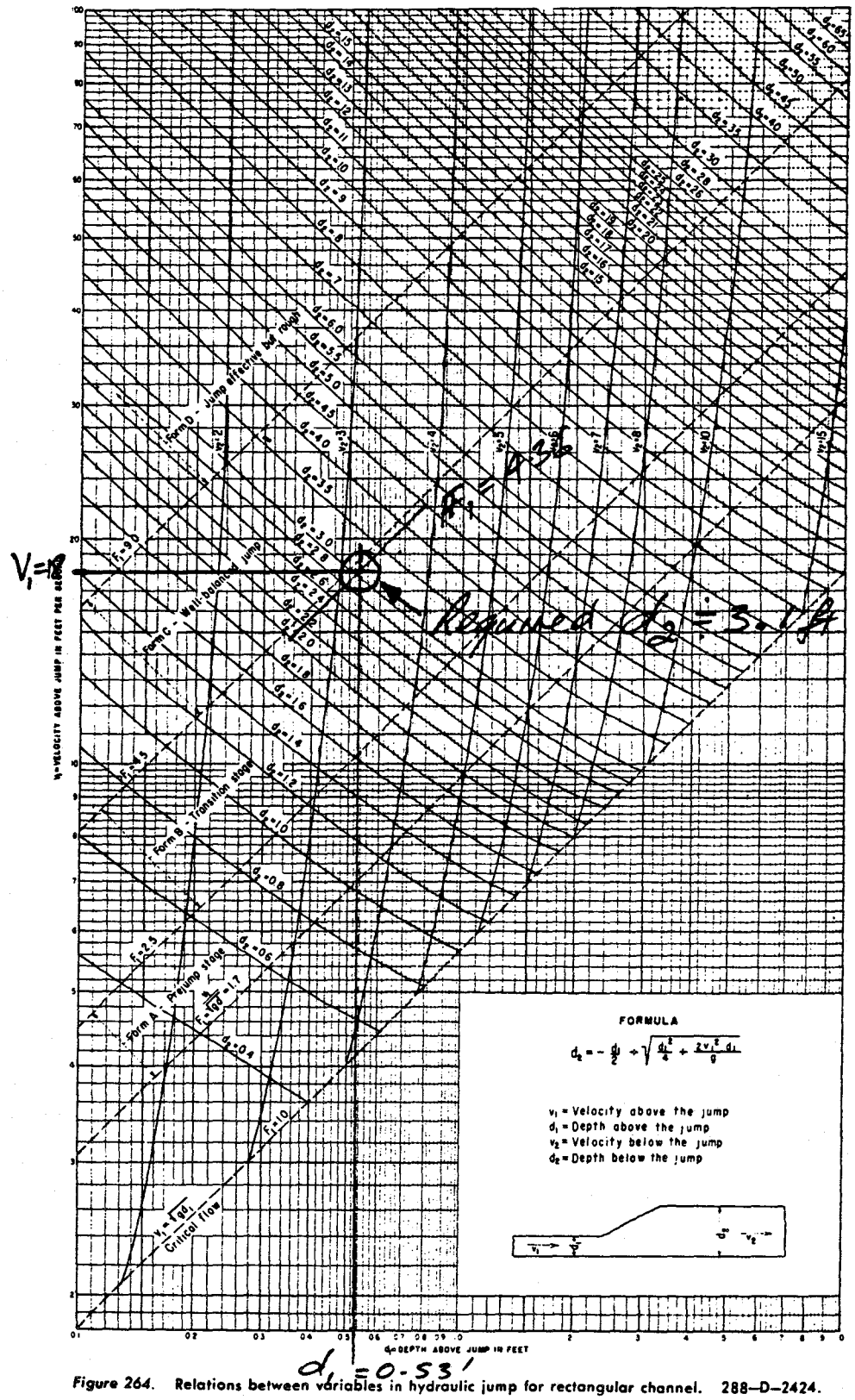

Figure 264. Relations between variables in hydraulic jump for rectangular channel. 288—D—2424.

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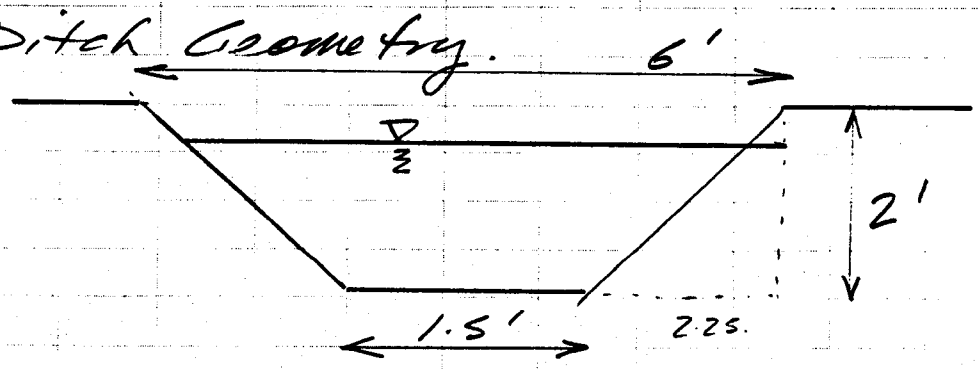
Inlet Hydraulics
Inlet #1 - Return ditch
Sheet No. 1 of 1

RETURN DITCH FLOW AT CAPACITY

Ditch Geometry.



$$\text{Area} = \frac{6' + 1.5'}{2'} \times 2' = 7.5\ ft^2$$

$$\text{Wet. Perim} = 1.5 + 2 \times (2.25^2 + 2^2)^{\frac{1}{2}} = 7.5\ ft$$

$$\text{Hydraulic Rad} = A/P = 1$$

$$S_o = 40/13000 = 0.0031 \text{ from topo maps}$$

$$Q = \frac{1.49}{n} A R^{2/3} S_o^{\frac{1}{2}}$$

$$= \frac{1.49}{.013} \times 7.5 \times 1^{2/3} \times .0031^{\frac{1}{2}}$$

$$\boxed{Q = 48\ cfs.}$$

Equivalent Circular Pipe = 42" Φ @ S = .31%

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Subject Inlet Structure No. 1
Return Ditch Bypass Sewer
and Irrigation Supply Pipe
Sheet No. 1 of 3

Reference: Ian McAlister comp 13 Feb 91, pp. 1 of 1

Capacity of return ditch = 48 cfs.
Invert of main return canal = El. 64.8' feet
Slope of 42"ϕ = 0.31 %
Capacity = 55 cfs

Bypass Sewer

Length of bypass sewer = 320 feet

Invert @ drop box = El. 64.8 + (320)(.0031'/ft)

= 65.80

Invert of Lake = 40.0

Irrigation Supply Pipe

Select a pipe size of 24"ϕ for receiving irrigation water. This can be cleaned easily if req'd

Approx. length of pipe = 220'

$$\text{Slope} = \frac{65.8 - 40.0}{220} \times 100\% = 11.7\%$$

Capacity of 24" @ 11.7% > 75 cfs, thus this pipe can accept the maximum flow the channel can carry.

3004

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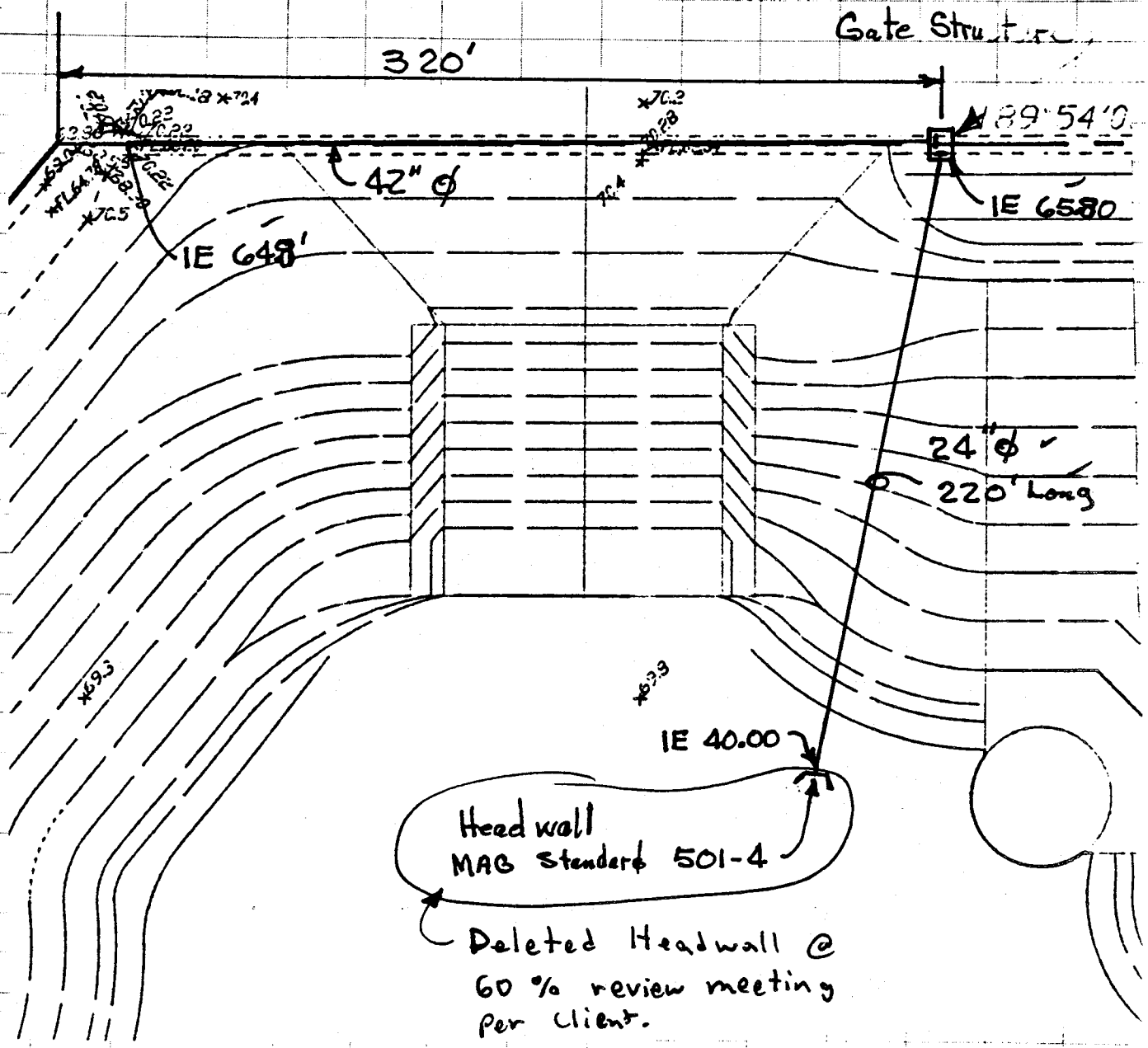
Job No. 10924 Page No. 59
Subject Inlet Structure No. 1
Return Ditch Bypass Sewer
and Irrigation Supply Pipe
Sheet No. 2 of 3

Plan (No Scale)



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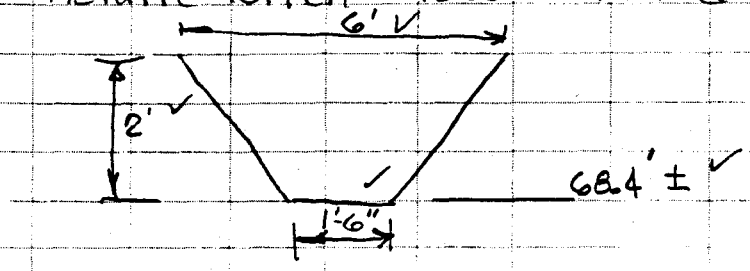
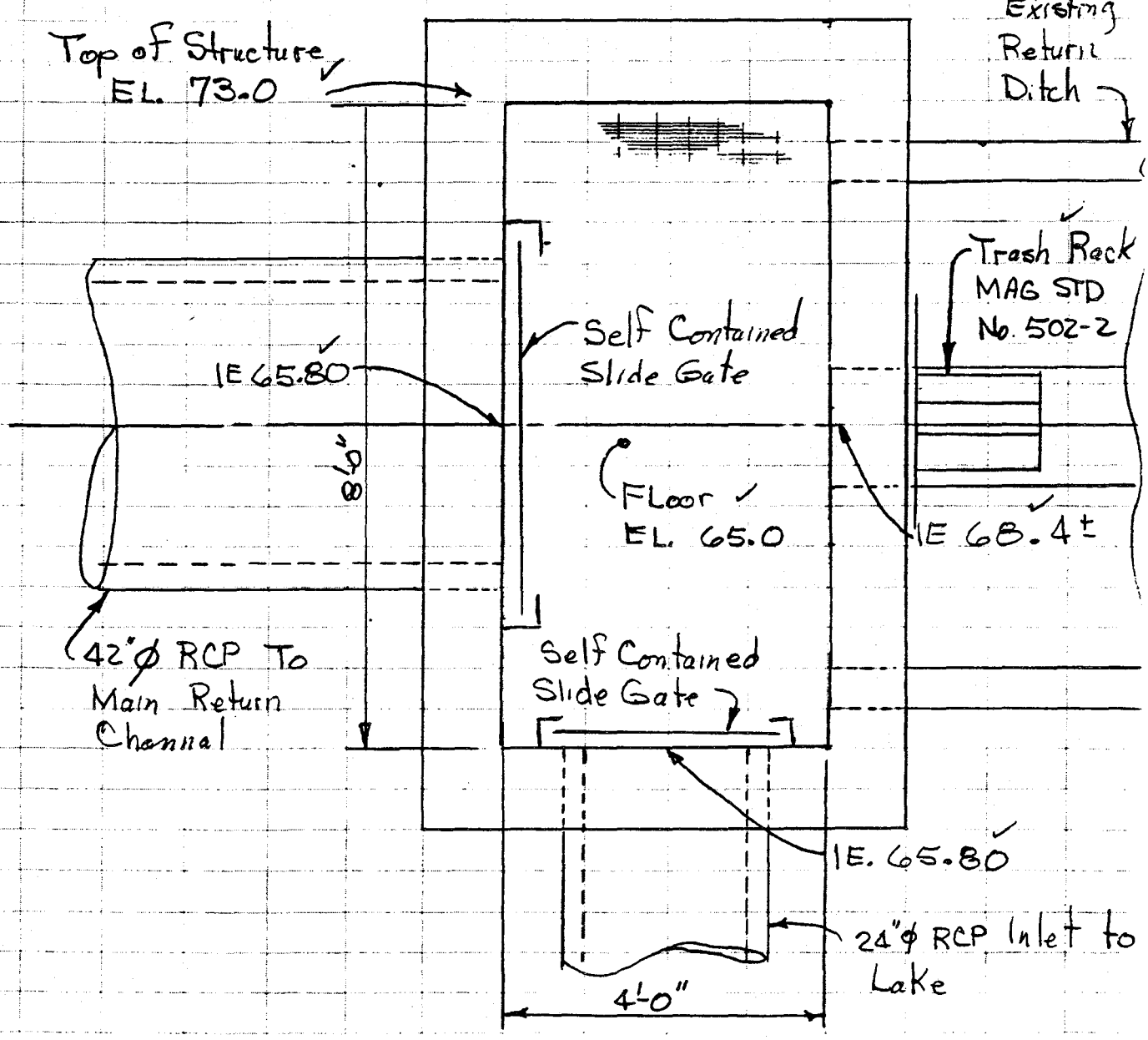
Subject Inlet Structure No. 1
Return Ditch Bypass Sewer
and Irrigation Supply Pipe
Sheet No. 3 of 3

Return Ditch Cross section @ Gate Structure



Plan
Gate Structure



3004

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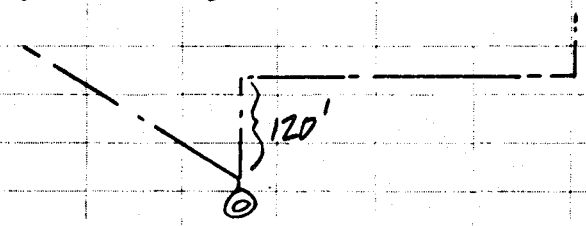
Job No. 10924 Page No. 61
Subject: Crossroads Detention Basin Inlet #3
Sheet No. 1 of 6

INLET #3 - SOUTHEAST INLET - HYDRAULICS

Input Parameters.

$Q_{max} = 500$ cfs.
Natural grade $\simeq 0.0013$.
Natural Channel Width $\simeq 120'$.



Natural depth of Flow by Mannings.

$$q_0 = \frac{1.49}{n} y^{8/3} S_0^{1/2}$$

$$\Rightarrow y_0 = \left(\frac{q n}{1.49 S_0^{1/2}}\right)^{.375}$$

$q = Q_{max}/B$
$= 500/120$
$= 4.17$ cfs/ft

$n = 0.025$ assumed natural.

$$y_0 = \left(\frac{4.17 \times .025}{1.49 \times .0013^{1/2}}\right)^{.375}$$

$$y_0 = 1.28 \text{ ft}$$

This is the assumed depth of natural flow at Location 0.

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Inlet # 3 Hydraulics
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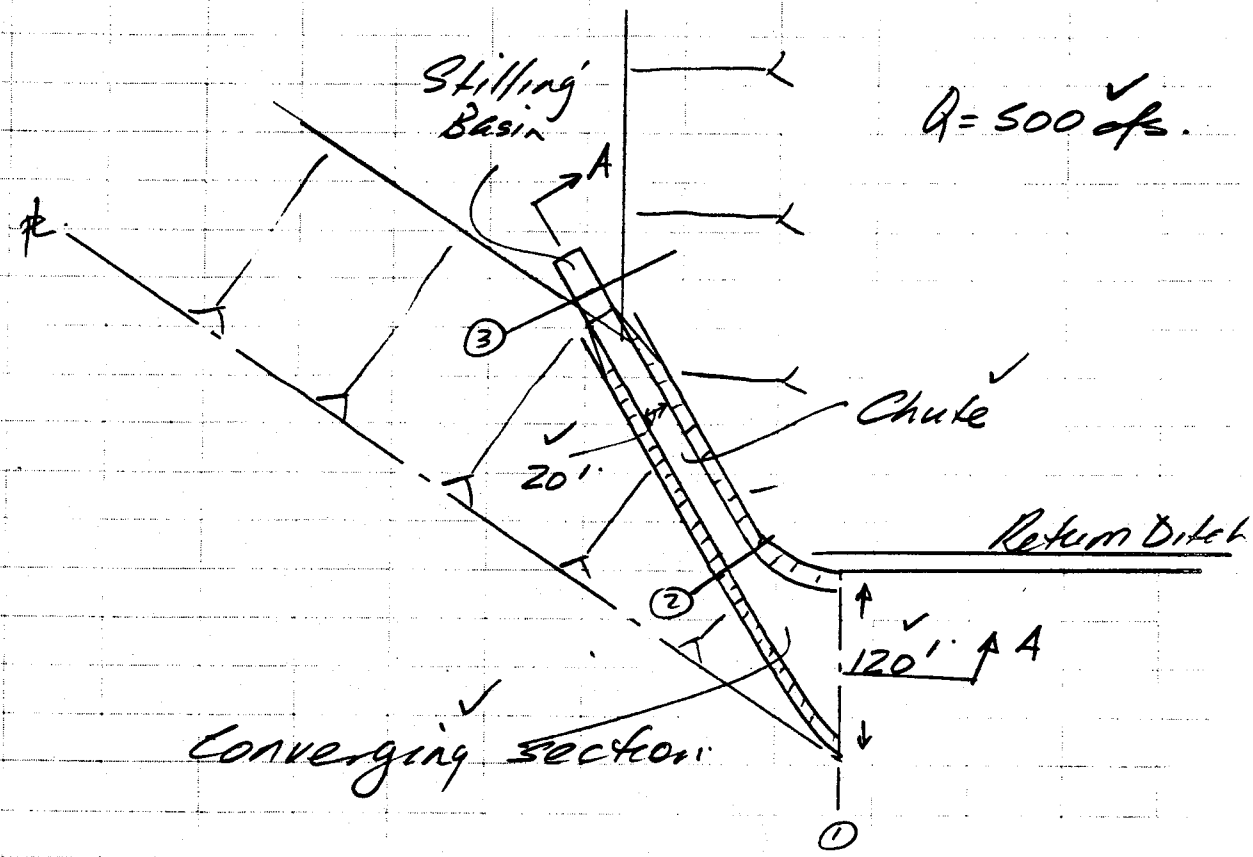

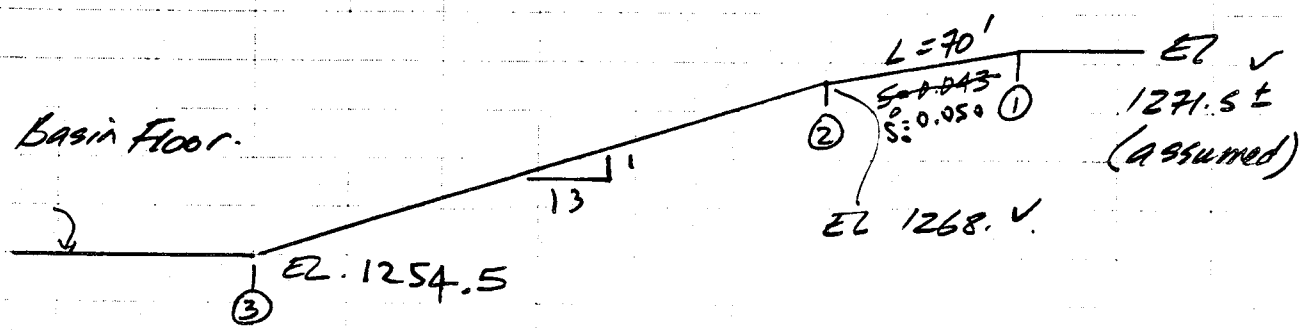

Approach

Will use energy equation and normal depth analysis to determine depth flow and flow velocity at points 1, 2 3 on inlet.

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Subject Cross Roads Basin
Inlet #3 Hydraulics
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At Point ①

$$E_{①} = z_{①} + y_{①} + \frac{q_{①}^2}{2g\,y_{①}^2}$$

$$E_{①} = 1271.5 + 1.28 + \frac{4.17^2}{64.4 \times 1.28^2}$$

$z_{①} = 1271.5'$

$y_{①} = 1.28'$

$q_{①} = \frac{500'}{120'} = 4.17$

Energy at ①

$$\boxed{E_{①} = 1272.94\ ft}$$

At Point ②

$$E_{②} = E_{①} - h_{L\,①\to②}$$

$$z_{②} + y_{②} + \frac{Q^2}{2g\,A_{②}^2} = 1272.94 - 0.21$$

$$y_{②} + \frac{3882}{(20y_{②} + 4y_{②}^2)^2} = 4.73$$

$$y_{②} \doteq 1.4'$$

Assume $\bar{y}$ 2.5

$\Rightarrow f = 0.113\left(\frac{d}{\bar{y}}\right)^{.33}$

$= 0.113 \times (1.33)^{.3}$

$= 0.058$

$h_L = \frac{f\,L\,\bar{q}^2}{8g\,\bar{y}^3}$

$h_L \doteq 0.21$

$z_{②} \doteq 1268$

$\bar{y} = \frac{1.28 + 1.4}{2} \Rightarrow 1.4 \pm \neq$ Assumed 2.5'

with $\bar{y} = 1.5' \Rightarrow f = 0.068 \Rightarrow h_L = 1.15'$

$$\Rightarrow y_{②} + \frac{3882}{(20y_{②} + 4y_{②}^2)^2} = 3.79$$

$$\Rightarrow \boxed{y_{②} = 1.6'}$$

$\bar{y} = \frac{1.28 + 1.6}{2} = 1.45$

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Inlet # 3 Hydraulics
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Checking on Normal Flow.

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}.$$

$$\Rightarrow AR^{2/3} = (20y_n + 4y_n^2)\left(\frac{20y_n + 4y_n^2}{20 + 8.24y_n}\right)^{2/3} = \frac{Qn}{1.49 S_o^{1/2}}$$

$$= \frac{500 \times .027}{1.49 \times (\frac{3}{70})^{1/2}}$$

$$43.77 = (20y_n + 4y_n^2)\left(\frac{20y_n + 4y_n^2}{20 + 8.24y_n}\right)^{2/3}$$

$\frac{1}{70.52}$

By trial & Error.

$$y_{n①} \doteq 1.5 \text{ ft}$$

$$y_{②} = 1.6' > y_{n①} = 1.5'$$

$$\boxed{\text{Use } y_{②} = 1.6'}$$ ✓

$$V = \frac{Q}{A} = \frac{500}{(20 \times 1.6 + 4 \times 1.6^2)}$$
$$= 11.8 \text{ fps}$$

With $y_{①} = 1.28'$ & $y_{②} = 1.6'$

and $\Delta \text{Elev}_{① \to ②} = 3'+$

Backwatering shouldn't be a problem Here ✓

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At Point ③

$$E_{③}' = E_{②}' - h_{f\,②\to③}$$

$$z_{③} + y_{③} + \frac{Q^2}{2gA_{③}^2} = z_{②} + y_{②} + \frac{Q^2}{2gA_{②}^2} - h_f$$

$$1254.5 + y_3 + \frac{3882}{(20y_3 + 4y_3^2)^2} = 1268 + 1.6 + \frac{3882}{42.24^2} - 3.90$$

$$y_{③} + \frac{3882}{(20y_3 + 4y_3^2)^2} = 13.37' = \phi$$

$$y_{③} = 0.76'$$

$\Rightarrow \bar{y} = \frac{1.6 + 0.76}{2} \doteq 1.2' <$ Assumed 1.6

with $\bar{y} = 1.0' \to f = 0.078 \to h_f = 7.41' \Rightarrow \phi = 9.9$

$$y_{③} + \frac{3882}{(20y_3 + 4y_3^2)^2} = 9.9$$

$$\Rightarrow y_3 \doteq 0.9\,ft \qquad \Rightarrow \bar{y} = \frac{1.6 + 0.9}{2} = 1.25$$

with $\bar{y} = 1.2' \to f = 0.073 \to h_f = 5.8 \Rightarrow \phi\ 11.48$

$$y_{③} + \frac{3882}{(20y_3 + 4y_3^2)^2} = 11.5$$

$$\boxed{\Rightarrow y_3 \doteq 0.82\,ft}$$

$\bar{y} = \frac{.82 + 1.6}{2} = 1.21'$ OK

for E②:
$y_{②} = 1.6'$
$z_{②} = 1268.5'$

for E③:
$z_{③} = 1254.5'$

for $h_{f\,②\to}$
$f \doteq .113\left(\frac{.33}{1.6}\right)^{.33}$
$f = 0.066$
$h_f = \frac{0.066 \times 175 \times 1}{.8 \times 32.2 \times 1.6}$
$h_f = 3.90'$

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Check against normal flow

$$AR^{2/3} = \frac{Qn}{1.49\, S_o^{1/2}} \qquad S_o = \frac{17}{220}$$

$$= \frac{500 \times 0.027}{1.49\,(17/220)^{1/2}} = 32.59$$

$$(20y_n + 4y_n^2)\left(\frac{20y_n + 4y_n^2}{20 + 8.24y_n}\right)^{2/3} = 32.59$$

By trial & error

$$y_n \doteq 1.3'$$

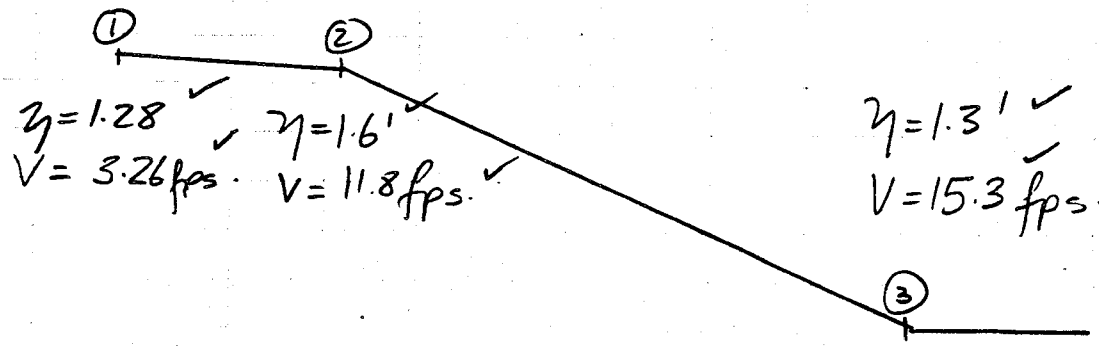
$$y_n = 1.3 > y_{③} = 0.82'$$

$$\boxed{\therefore\ y_{③} = 1.34}$$

$A = 32.76\ ft^2$

$\Rightarrow V = Q/A = \frac{500}{32.76} = 15.26$

Summary



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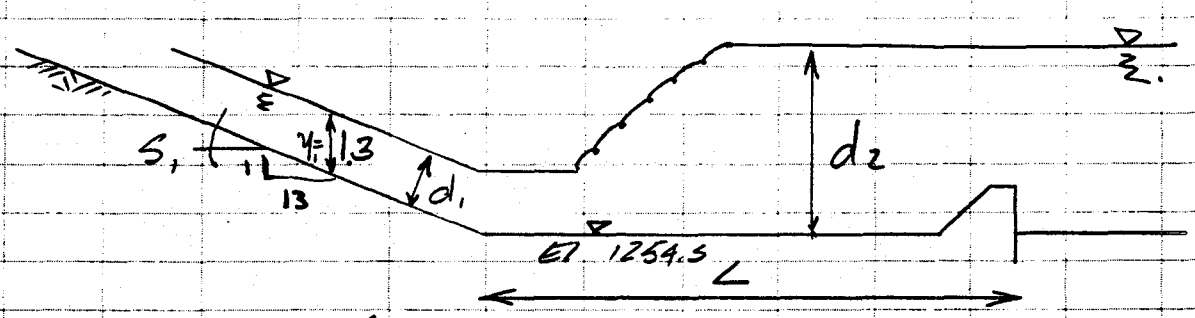
Job No. 10924 Page No. 67
Subject Cross Roads Basin
Final Hydraulics Design
Inlet #3
Sheet No. 1 of 3

INLET #3: STILLING BASIN DESIGN

Profile



From comps 3-7 91 pg 6.

$$y_1 = 1.3', \quad S_1 = 0.077 \ (\equiv 4.4°)$$

$$\Rightarrow d_1 = \cos 4.4° * y_1$$
$$= 0.999 \times 1.3$$

$$\boxed{d_1 = 1.3'}$$

$$\text{Froude \# @ ①} = \frac{V_1}{\sqrt{gA_1/B_1}} = \frac{Q/A_1}{\sqrt{gA_1/B_1}}$$

$$F_1 = \frac{500/(20 \times 1.3 + 4 \times 1.3^2)}{\sqrt{32.2 \times \frac{(20 \times 1.3 + 4 \times 1.3^2)}{20 + 8 \times 1.3}}}$$

$$\boxed{F_1 = 2.4}$$

$$d_2 = \frac{d_1}{2} + \sqrt{\frac{d_1^2}{4} + \frac{2V_1^2 d_1}{g}}$$

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Inlet #3
Final Hydraulics Design
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$$d_2 = \frac{1.3'}{2} + \sqrt{\frac{1.3^2}{4} + \frac{2'}{32.2} \times \frac{1.3' \times 500^2}{(\underbrace{20' \times 1.3' + 4' \times 1.3^2}_{32.67})}}$$

$$d_2 = 0.65' + (0.42 + \cancel{19.16}\ 18.81)^{\frac{1}{2}}$$

$$\boxed{d_2 = 5.05'}$$

$$V_1 = Q/A_1 = 500/(20 \times 0.1.3 + 4 \times 1.3^2)$$

$$\boxed{V_1 = 15 \text{ fps.}}$$

From Fig 265, pg 398 "Design Small Dams"

with $F_1 = 2.4$

$L/d_2 = 4.75 \Rightarrow L = 4.75 \times 5.05$

$L = 23.99'$

$$\boxed{\text{Use } L = 24'}$$

$T_w/d_2 = 1.1 \Rightarrow T_w = 1.1 \times 5.05 = 5.6'$

$$\boxed{\begin{array}{l}\text{Side Wall Elev} \approx +8' \\ \text{Allows } 2' + \text{Freeboard}\end{array}}$$

3004

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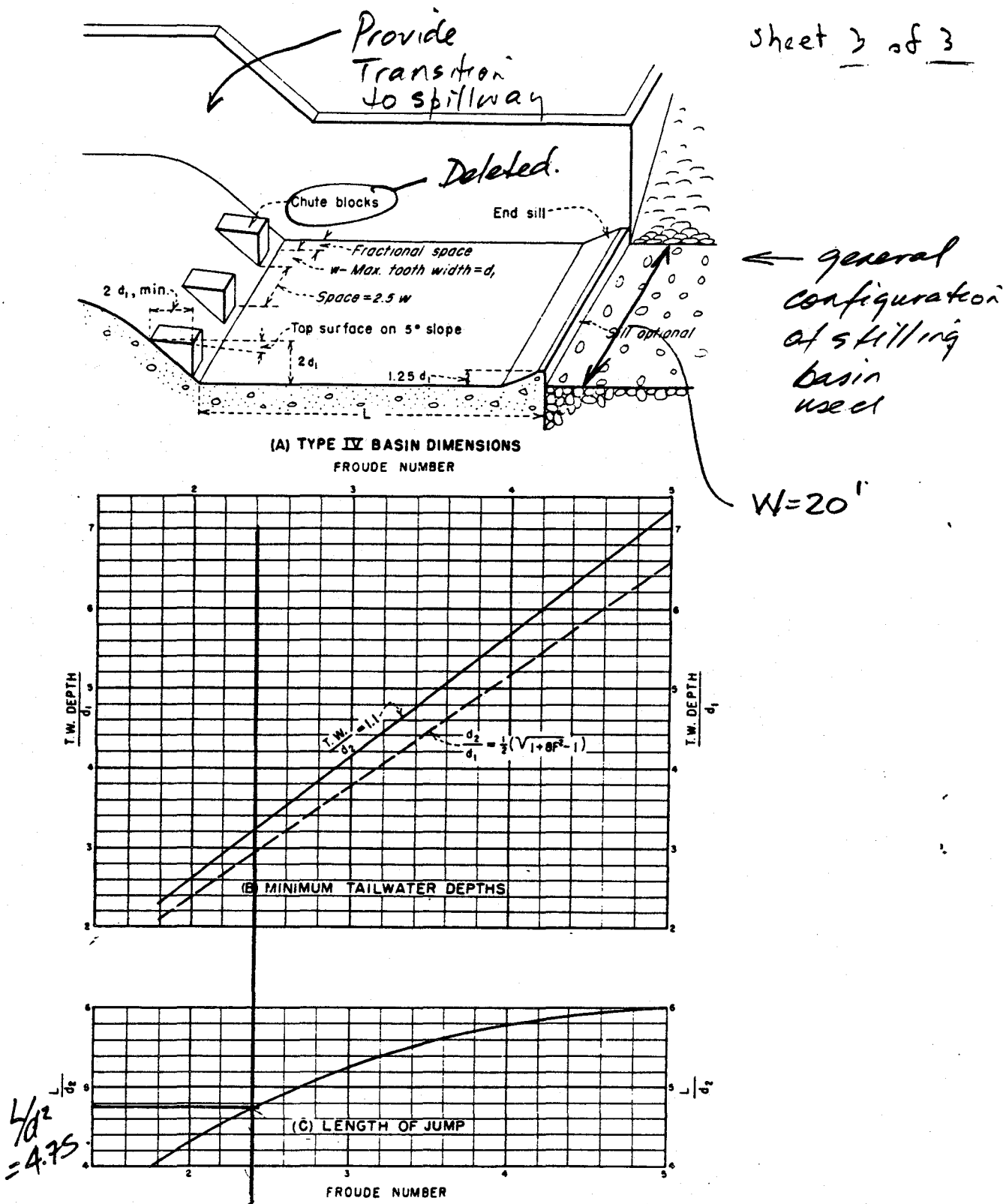

Figure 265. Stilling basin characteristics for Froude numbers between 2.5 and 4.5. 288—D—2425.

GROUTED RIPRAP DESIGN

Reference: "Design of Rip-rap Revetment"
US. DOT., FHA.
HEC # 11, FHWA-IP-89-016
Mar 1989.

1. Flow Velocity

(See following sheet)

$V \approx 15$ to 17 fps.

2. Grouted Riprap Armor Thickness

See Fig 57 pg 101 of reference (attach

$t = 21''$ ← sets max stone size:

Layer thickness

3. Riprap grading

See Tab 6, pg 104 ref. (attached).

For $t_{max} = 21''$ use Light Class.

1.75'	(1/4 ton)	0-5%
1.25'	(200 lb)	50-100% Retained
1.00'	(75 lb)	95-100%

Terminal Flow Velocities in Grouted Rip-rap Spillways

INLET #3

n=0.028
So=17/220

AR^2/3=Q.n/1.49/So^1/2
A=20.y+4.y.y
P=20+8.y R=A/P

Q (cfs)	AR^2/3 Formula	y (ft)	A (ft^2)	P (ft)	R (ft)	AR^2/3 Geometry	V (fps)
0	0.00	0	0	20	0.000	0.00	0
50	3.38	0.34	7.26	22.72	0.320	3.40	6.88
100	6.76	0.51	11.24	24.08	0.467	6.76	8.90
150	10.14	0.65	14.69	25.20	0.583	10.25	10.21
200	13.52	0.76	17.51	26.08	0.671	13.43	11.42
250	16.90	0.87	20.43	26.96	0.758	16.98	12.24
300	20.28	0.96	22.89	27.68	0.827	20.16	13.11
350	23.66	1.03	24.84	28.24	0.880	22.81	14.09
400	27.04	1.14	28.00	29.12	0.961	27.27	14.29
450	30.42	1.21	30.06	29.68	1.013	30.31	14.97
500	33.80	1.29	32.46	30.32	1.070	33.96	15.41

INLET #1

n=0.028
So=.25

AR^2/3=Q.n/1.49/So^1/2
A=100.y+4.y.y
P=100+8.y R=A/P

Q (cfs)	AR^2/3 Formula	y (ft)	A (ft^2)	P (ft)	R (ft)	AR^2/3 Geometry	V (fps)
0	0.00	0	0.00	100.00	0.000	0.00	0
100	3.76	0.14	14.08	101.12	0.139	3.78	7.10
200	7.52	0.21	21.18	101.68	0.208	7.44	9.44
300	11.28	0.27	27.29	102.16	0.267	11.32	10.99
400	15.03	0.32	32.41	102.56	0.316	15.04	12.34
500	18.79	0.37	37.55	102.96	0.365	19.17	13.32
600	22.55	0.41	41.67	103.28	0.403	22.75	14.40
700	26.31	0.45	45.81	103.60	0.442	26.59	15.28
800	30.07	0.48	48.92	103.84	0.471	29.62	16.35
900	33.83	0.52	53.08	104.16	0.510	33.87	16.96
1000	37.58	0.55	56.21	104.40	0.538	37.20	17.79

Computed by: I. McAlister Date: 9 Apr. 91
Checked by: Date:
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Approved by: BOR Date: 5-11-91

Job No. 1092 Page No. 72
Subject: Cross Roads Basin
Grouted Riprap Design
Sheet No. 3 of 3

4. Bedding layer

Ref: "Quarried Stone for Erosion & Sedement Control"
National Crushed Stone Association; June

Set $t = 4''$

From Table A. for Filter Stone use FS#3 with $D_{max} = 4''$

4"	0-5%
2½"	50-100% Retained
#16 Sieve	95-100%

*Note: Any crushed stone having this general gradation will be appropriate

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U.S. Department
of Transportation

**Federal Highway
Administration**

Hydraulic Engineering Circular No. 11

Publication No. FHWA-IP-89-016
March 1989

Design of Riprap Revetment

Research, Development, and Technology
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, Virginia 22101-2296

6.3 GROUTED ROCK

Grouted rock revetment consists of rock slope-protection having voids filled with concrete grout to form a monolithic armor. See section 2.5 for additional descriptive information and general performance characteristics for grouted rock. Sample specifications for components of grouted rock revetments are provided in appendix A.

6.3.1 Design Guidelines for Grouted Rock

Components of grouted rock riprap design include layout of a general scheme or concept, bank preparation, bank slope, rock size and blanket thickness, rock grading, rock quality, grout quality, edge treatment, filter design, and pressure relief.

General: Grouted riprap designs are rigid monolithic bank protection schemes. When complete, they form a continuous surface. A typical grouted riprap section is shown in figure 56.

Grouted riprap should extend from below the anticipated channel bed scour depth to the design high water level plus additional height for freeboard (see section 3.6.2). The longitudinal extent of protection should be as described in section 3.6.1.

During the design phase for a grouted riprap revetment, special attention needs to be paid to edge treatment, foundation design, and mechanisms for hydrostatic pressure relief. Each of these items is discussed below.

Bank and Foundation Preparation: The bank should be prepared by first clearing all trees and debris from the bank, and grading the bank surface to the desired slope. In general, the graded surface should not deviate from the specified slope line by more than six in (15.2 cm). However, local depressions larger than this can be accommodated since initial placement of filter material and/or rock for the revetment will fill these depressions.

Since grouted riprap is rigid but not extremely strong, support by the embankment must be maintained. To form a firm foundation, it is recommended that the bank surface be tamped or lightly compacted. Care must be taken during bank compaction to maintain a soil permeability similar to that of the natural, undisturbed bank material. The foundation for the grouted riprap revetment should have a bearing capacity sufficient to support either the dry weight of the revetment alone, or the submerged weight of the revetment plus the weight of the water in the wedge above the revetment for design conditions, whichever is greater.

Any large boulders or debris found buried near the edges of the revetment should be removed.

Bank Slope: Bank slopes for grouted riprap revetments should not exceed 1.5:1.

Rock Size and Blanket Thickness: Blanket thickness and rock size requirements for grouted riprap installations are interrelated. Figure 57 illustrates a relationship between the design velocity and the required riprap blanket thickness for grouted riprap designs. The median rock size in the revetment should not exceed 0.67 times the blanket thickness. The largest rock used in the revetment should not exceed the blanket thickness.

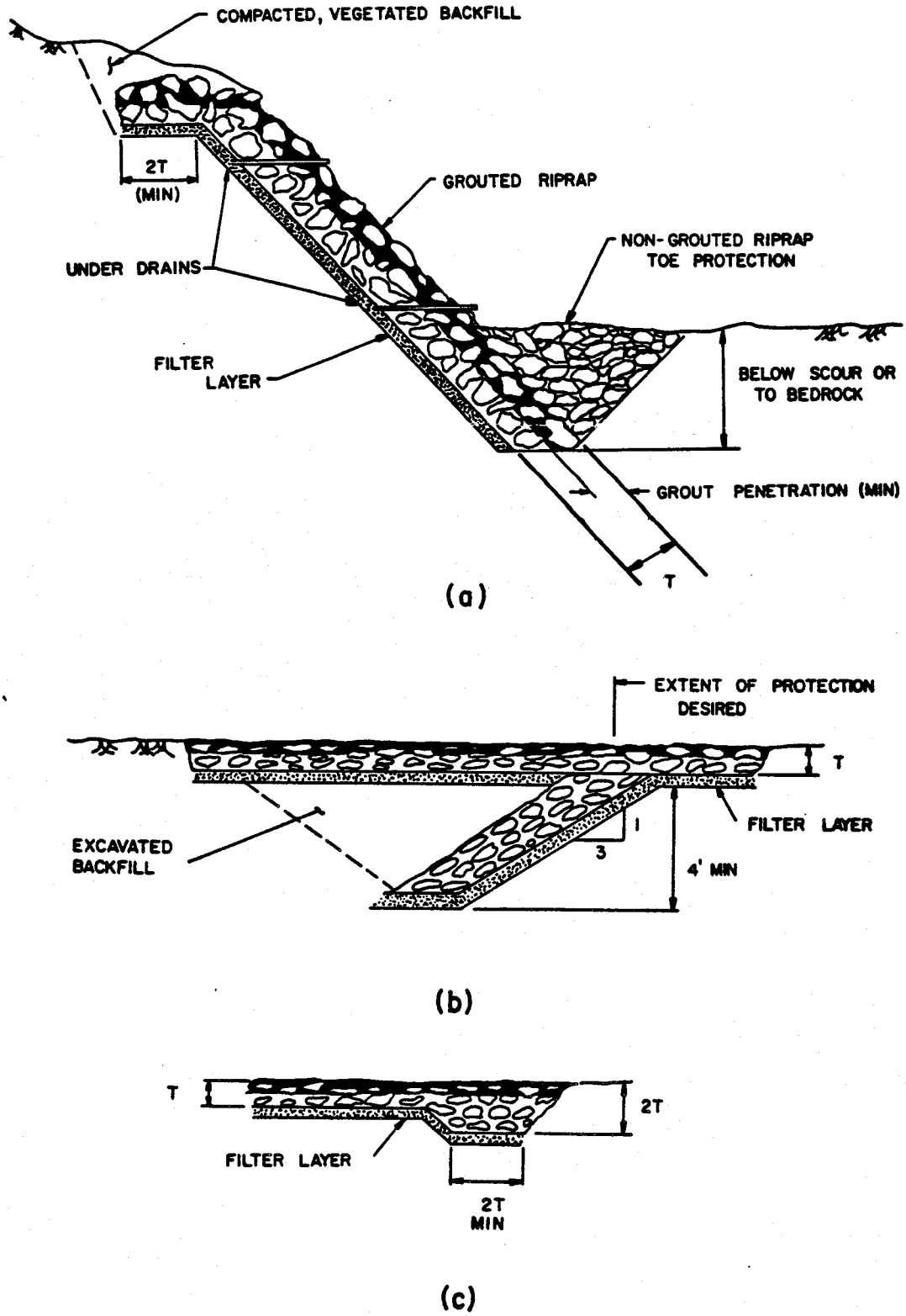

Figure 56. Grouted riprap sections: (a) section A-A; (b) section B-B; and (c) section C-C. (refer to figure 39 for section locations)

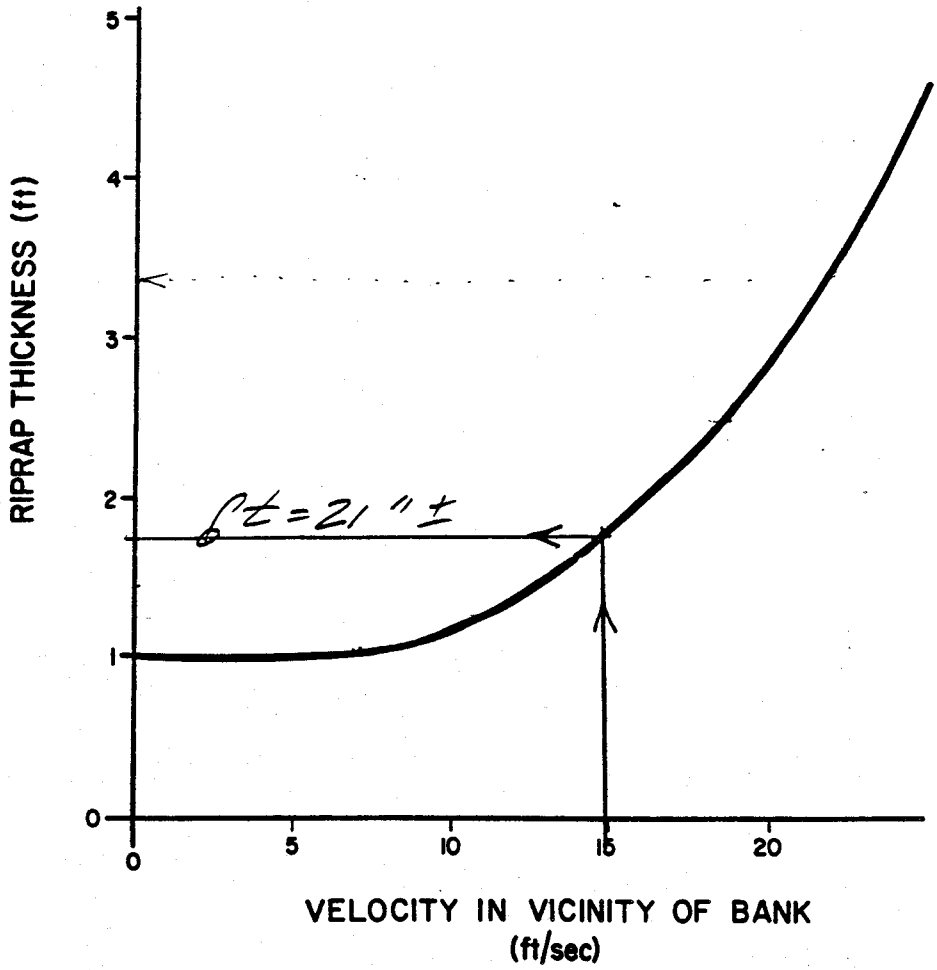

Figure 57. Required blanket thickness as a function of flow velocity.

Rock Grading: Table 6 provides guidelines for rock gradation in grouted riprap installations. Six size classes are listed.

Rock Quality: Rock used in grouted rock slope-protection is usually the same as that used in ordinary rock slope-protection. However, the specifications for specific gravity and hardness may be lowered if necessary as the rocks are protected by the surrounding grout.

In addition, the rock used in grouted riprap installations should be free of fines in order that penetration of grout may be achieved.

Grout Quality and Characteristics: Grout should consist of good strength concrete using a maximum aggregate size of 3/4 in and a slump of 3 to 4 in (7.6 to 10.2 cm). Sand mixes may be used where roughness of the grout surface is unnecessary, provided sufficient cement is added to give good strength and workability.

The volume of grout required will be that necessary to provide penetration to the depths shown in table 6.

The finished grout should leave face stones exposed for one-fourth to one-third their depth and the surface of the grout should expose a matrix of coarse aggregate.

Edge Treatment: The edges of grouted rock revetments (the head, toe, and flanks) require special treatment to prevent undermining. The revetment toe should extend to a depth below anticipated scour depths or to bedrock. The toe should be designed as illustrated in figure 56(a). After excavating to the desired depth, the riprap slope protection should be extended to the bottom of the trench and grouted. The remainder of the excavated area in the toe trench should be filled with ungrouted riprap. The ungrouted riprap provides extra protection against undermining at the bank toe.

To prevent outflanking of the revetment, various edge treatments are required. Recommended designs for these edge treatments are illustrated in figure 56, parts (a), (b), and (c).

Filter Design: Filters are required under all grouted riprap revetments to provide a zone of high permeability to carry off seepage water and prevent damage to the overlying structure from uplift pressures. A 6-in (15.4-cm) granular filter is required beneath the pavement to provide an adequate drainage zone. The filter can consist of well-graded granular material or uniformly-graded granular material with an underlying filter fabric. The filter should be designed to provide a high degree of permeability while preventing base material particles from penetrating the filter, thus causing clogging and failure of the protective filter layer, chapter 4 contains more specific information for filter design.

Pressure Relief: Weep holes should be provided in the revetment to relieve hydrostatic pressure build-up behind the grout surface [see figure 56(a)]. Weeps should extend through the grout surface to the interface with the gravel underdrain layer. Weeps should consist of 3-in (7.6-cm) diameter pipes having a maximum horizontal spacing of 6-ft (1.8 m) and a maximum vertical spacing of 10-ft (3.0 m). The buried end of the weep should be covered with wire screening or a fabric filter of a gage that will prevent passage of the gravel underlayer.

6.3.2 Construction

Construction details for grouted riprap revetments are illustrated in figure 40. The following construction procedures should be followed:

Step 1. Normal construction procedures include (a) bank clearing and grading; (b) development of foundation; (c) placement of the rock slope protection; (d) grouting of the interstices; (e) backfilling toe and flank trenches; and (f) vegetation of disturbed areas.

Step 2. The rock should be wet immediately prior to commencing the grouting operation.

Step 3. The grout may be transported to the place of final deposit by chutes, tubes, buckets, pneumatic equipment, or any other mechanical method which will control segregation and uniformity of the grout.

Step 4. Spading and rodding are necessary where penetration is achieved by gravity flow into the interstices.

Step 5. No loads should be allowed upon the revetment until good strength has been developed.

Table 6. Recommended grading of grouted rock slope protection.

Rock Sizes		Classes (Percent Larger Than Given Rock Size)					
Equivalent Diameter (ft)	Weight	1 Ton	1/2 Ton	1/4 Ton	Light	Facing	Cobble
3.5	2-Ton	0-5					
2.75	1-Ton	50-100	0-5				
2.25	1/2-Ton	---	50-100	0-5			
1.75	1/4-Ton	95-100	---	50-100	0-5		
1.25	200-Lb.		95-100	---	50-100	0-5	
1.00	75-Lb.			95-100	95-100	50-100	0-5
0.50	25-Lb.					95-100	95-100
Minimum Penetration of Grout (in)		24	18	14	10	8	6

t=21"

Table 7. Dimensions for concrete slab

REVETMENT CLASS	DIMENSION											
	A	B	C	D	E	F	G	H	I	J	K	L
LIGHT DUTY	4"	0"	9"	1'-10"	1'-10"	6"	4"	4-5'	2'-3'	1-6"	15'-20'	6"
HEAVY DUTY	6"	9"	9"	1'9"	2'-0"	9"	6"	4'-5'	2'-3'	1'-6"	25'-30'	9"

Computed by: L. McAlister Date: 9 Apr 91
Checked by: Date:
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Approved by: BDR Date: 5/11/91

Job No. 10924 Page No. 79
Subject: Cross Roads Basin Riprap Design
Sheet No. 1 of 1

Plain Riprap. (Channel Lining d/s Stilling Basin, Inlet #3

1. Flow Velocity:

at exit to stilling basin
$q = 25$ cfs, $d = 5$ ft $\Rightarrow V \simeq 5$ fps

Further along the stream $S_o \simeq 0.01$

By normal depth:

$$y_n = \left(\frac{qn}{1.49 S_o^{1/2}}\right)^{0.6} = \left(\frac{25 \times .027}{1.49 \times 0.1}\right)^{0.6}$$

$$y_n = 2.48 \text{ ft}$$

$$\Rightarrow V = q/y = 25/2.48 \simeq 10 \text{ fps}$$

← more critical

2. Riprap Grading

From "Quarried Stone for Erosion & Sediment Control" National Stone Association June 1979, Table A pg 29

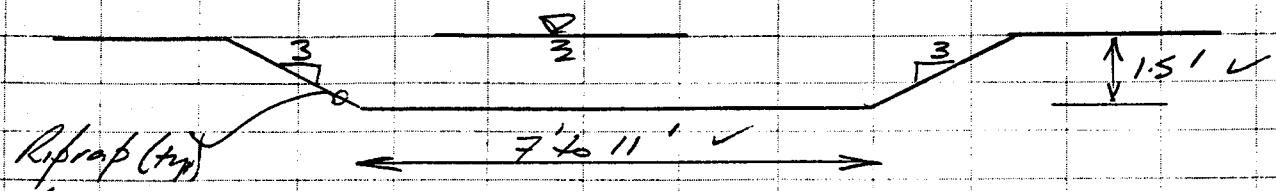
Use R-5 (V=10 fps)	18"	0-5%	retained
	9"	50-100%	
	5"	95%-100%	

Use same bedding as for Grouted Riprap ✓

Computed by J McAlister Date 3 July 91
Checked by BDR Date 7/3/91
Reviewed by Date
Approved by BDR Date 7/3/91

Job No. 10924.02 Page No. 79-A
Subject Crossroads Detention Bas. Channel Riprap design
Sheet No. 1 of 2

CHANNEL RIPRAP DESIGN - Low flow channel across bottom of basin.



Assume $n = .017$

Calculate Brim Full Velocities

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

Let $n = 0.017$

$S = \frac{\Delta H}{\Delta L} = \frac{51-47}{860} = 0.0046$

$V = \frac{1.49}{.017} R^{2/3} .0046^{1/2} = 5.98 R^{2/3}$

$R = \frac{A}{P} = \frac{By + zy^2}{B + 6.3y}$ $y = 1.5'$, $z = 3$

$R = \frac{1.5B + 6.75}{B + 9.45}$

For $B = 7'$

$R = \frac{1.5 \times 7 + 6.75}{7 + 9.45}$

$R = 1.05'$

$V = 5.98 \times 1.05^{2/3}$

$V = 6.2$ fps

For $B = 11'$

$R = \frac{1.5 \times 11 + 6.75}{11 + 9.45}$

$R = 1.14$

$V = 5.98 \times 1.14^{2/3}$

USE ⇒ $V = 6.4$ fps.

6.5

Computed by J. McAlister Date 3 July 91
Checked by BDR Date 7/3/91
Reviewed by Date
Approved by BDR Date 7/3/91

Job No. 10924.02 Page No. 79.B
Subject Crossroads Detention Des.
Channel Riprap
Sheet No. 2 of 2

Rip Rap Design

Ref: "Quarried Stone for Erosion & Sediment Control", No 2.
National Crushed Stone Association
June, 1978

From Table A, pg 29 ✓
with V = 6.4 (6.5) fps. ✓

Use: R-3 Stone: ✓

ie Dmax 6" ✓ (12 lb±) Thickness
Dmean 3" ✓ (1.5 lb±) = 6" ✓
Dmin 2" (0.4 lb±)

Filter Stone:

Dmax 2" ✓ Thickness
Dmean #4 ✓ = 3" ✓
Dmin #100 ✓

Computed by SCSmith Date 4-4-91
Checked by TURNER Date 4/12/91
Reviewed by Date
Approved by Date 5-7-91

Lateral earth pressures

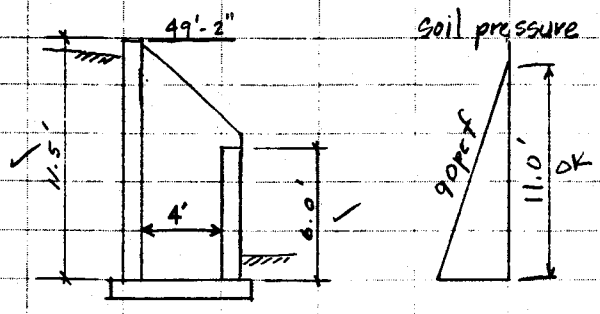
Use At-rest pressure coefficient of 0.5 (no rotation)
$\gamma_{sat} \cong 120$ pcf

Flood water EL is assumed at top of pump station (EL 73'-0") per J. Fink

$\gamma_{design} = 0.5(120-62.4) + 62.4 = 91.2$ **use $\gamma = 90$ pcf**

When considering load case with H_2O in tanks, neglect the surrounding soil.

Intake Structure



PCA Table III
$b/a = 20/11.5 = 1.74$

Horizontal

Corner $M_H = .052(.09)(11)^3 = 6.2^{1-k/1}$ OF

Midspan $M_H = .025(.09)(11)^3 = 3.0^{1-k/1}$ IF

Vertical

Bottom $M_V = .074(.09)(11)^3 = 8.9^{1-k/1}$ OF

• Conserv. to assume 90pcf for full height of wall

Shear

$V \cong \frac{1}{2}(.09)(11)^2 = 5.4^{k/1}$

For water inside, factor above values by $\frac{.0624}{.09} = 0.69$:

Horizontal

Corner $M_H = 4.3^{1-k/1}$ IF

Midspan $M_H = 2.1^{1-k/1}$ OF

Vertical

Bottom $M_V = 6.2^{1-k/1}$ IF

Use allowable stress design to select reinf.

Intake Structure (cont'd)

Try a 12" thick wall

$d_H \cong 12 - 2.0 - \frac{0.75}{2} = 9.6''$

$d_V \cong 12 - 2.0 - 0.75 - \frac{0.875}{2} = 8.8''$

Shear capacity $= \frac{1.1\sqrt{4000}}{1000}(12)(8.8) = 7.4^{k/'} > 5.4^{k/'}$ OK

Horizontal steel — Use:

Corner	OF	$A_s = 6.2/1.76(9.6) = 0.37\ in^2/'$		#6@12
"	IF	$A_s = 4.3/1.76(\ "\ ) = 0.25$		#5@12
Midspan	OF	$A_s = 2.1/1.76(\ "\ ) = 0.12$		#4@12
"	IF	$A_s = 3.0/1.76(\ "\ ) = 0.18$		#4@12

Vertical steel

OF	$A_s = 8.9/1.76(8.8) = 0.57\ in^2/'$	#6@9
IF	$A_s = 6.2/1.76(\ "\ ) = 0.40$	#5@9

Check for Uplift:

DL

$11.5 \times 22 \times 1$	$= 253\ ft^3$	Side Wall
$6.0 \times 20 \times 1$	$= 120$	"
$8.75 \times 5 \times 2 \times 1$	$= 87.5$	End Wall
$1.25 \times 23 \times 7$	$= 201.3$	Base Slab
	$661.8 \times 0.15\ KCF = 99.3^k$	(Neglects soil on l.p)

Uplift Force $= 22 \times 6 \times 0.0624 \times 7' \pm^{OK} = 57.7^k$ OK

Sliding — Extend base slab heel 3' beyond wall to pick up enough weight to provide a F.S. of about 1.5 against sliding.

Overturning — OK by inspection with 3' slab extension

Revised

Job No. 10924 Page No. 82

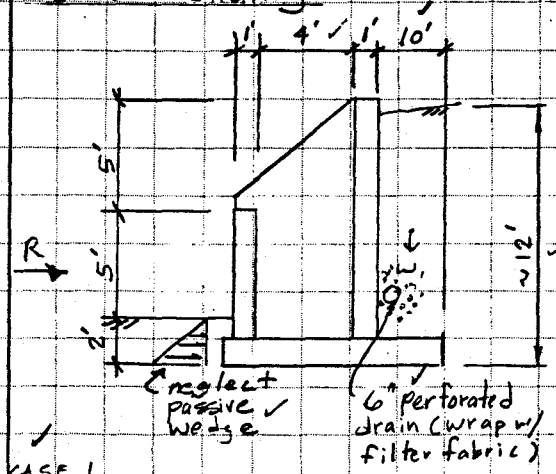
Computed by SC Smith Date 5-10-91 7-18-91 Subject Structural

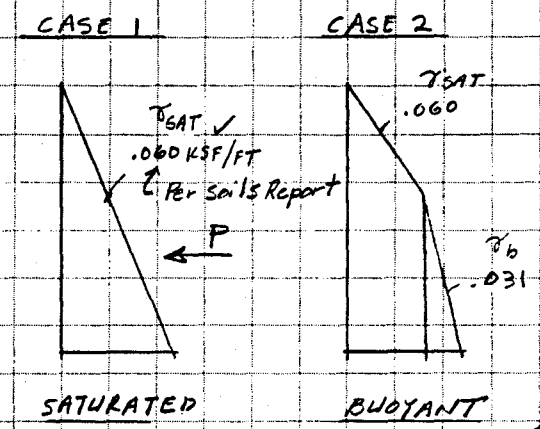
Checked by TURNER, Ray Date 5/10/91 7/18/91

Reviewed by Date Intake Structure

Approved by Rew Date 5/[illegible] 7/18/91 Sheet No. 3 of 5

Check Sliding





$\frac{62.6}{(125-62.4)0.5}$ / 1000 = 0.031

(H_2O pressure balances on opp. walls)

CASE 1

W_1 10 × 23' × 0.125 × 11' = 316.3

99.3 + 9.75(23)(1.25)(0.15) (sh. 2)

R = (316.3 + 141.3) × 0.4 = 183^k

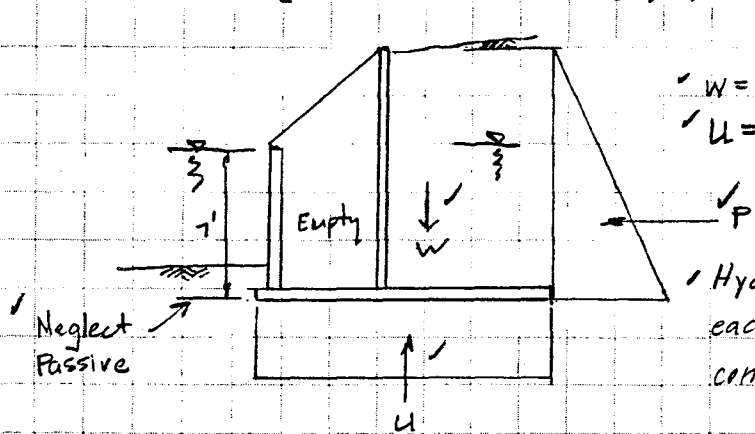
Friction ↑ per soils report

$P = 0.060(12)^2(\frac{1}{2})(22') = 95.0^k$

$F.S = \frac{183^k}{95.0} = 1.93$ ok - See case 2

CASE 2

$P = [0.06(5)^2(\frac{1}{2}) + .06(5)(7) + .031(7)^2(\frac{1}{2})] \times 22' = 79.4^k$



$W = 10(23)(0.125)(11) + 141.3^k = 458^k$ (γ_{SAT})

$U = 7'(16.25')(0.0624)(23') = 163^k$

Hydrostatic pressure is balanced on each side of structure and does not contribute to sliding.

$R = (W - U) \times 0.4 = (458 - 163)\,0.4 = 118^k$

$F.S. = \frac{118^k}{79.4^k} = 1.49 \approx 1.5$ OK ; Use 10' Heel

Job No. 10924 Page No. 83

Computed by SCSmith Date 4-4-91
Checked by TURNER Date 4/12/91
Reviewed by Date
Approved by Date 5-4-91

Subject Structural

Intake structure

Sheet No. 4 of 5

Intake structure (cont'd)

(Trash on Screen

Base slab

Total DL — Base slab DL — 5' Head

$$\text{Design pressure} = \frac{99.3^{k} - 201.3(0.15) + 5(.0624)(20)(4)}{6.5' \times 22.5'}$$

$$= \frac{94.1^{k}}{146.25} = 0.64 \text{ KSF}$$

$$M \cong 0.64(5)^2/10 = 1.6 \text{ '-k/'}$$

From wall

$$M_{TOTAL} = 1.6 + 8.9 = 10.5 \text{ '-k/'} \quad : \text{Try } 15'' \text{ slab}$$

$$d = 15'' - 3 - \frac{.875}{2} \cong 11.5''$$

$$A_s = \frac{10.5}{1.76(11.5)} = 0.52 \text{ in}^2/'$$

#6@9 Bottom SW
#5@12 Top SW

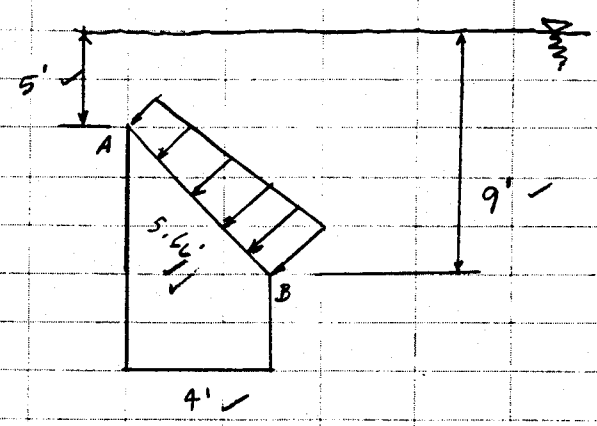
$$A_{s\,T+S} = 0.0018(15)(12)(\tfrac{1}{2}) = 0.16 \text{ in}^2/'$$

EF
#4@12 T+B LW

Bar Screen 1/2" x 2" bars @ 2 1/2" o.c.

Design for 5' head assuming fully plugged ✓

For one bar:



$b = \frac{2.5''}{12} = 0.2083'$

$w_A = 0.2083(62.4)(5) = 65\ \#/'$

$w_B = \quad " \quad " \quad (9) = 117\ \#/'$

$F = \frac{65+117}{2} \times 5.66 = 515.1\ \#$

$M = \frac{515.1\#(5.66')}{7.93} = 367.6$ ft-lb Ind. Check

← Rogers "Tables for fixed end moments"

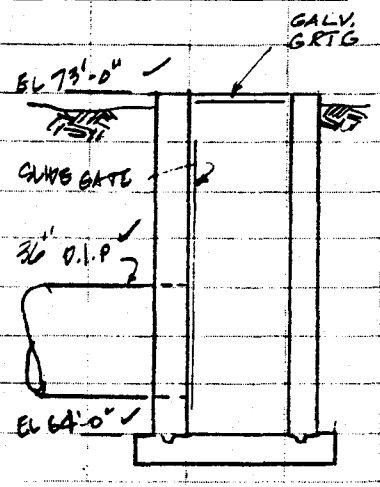
$f_b = \frac{367.6(12)(6)}{0.5(2)^2\ 1000} = 13.2$ ksi OK

Shear OK

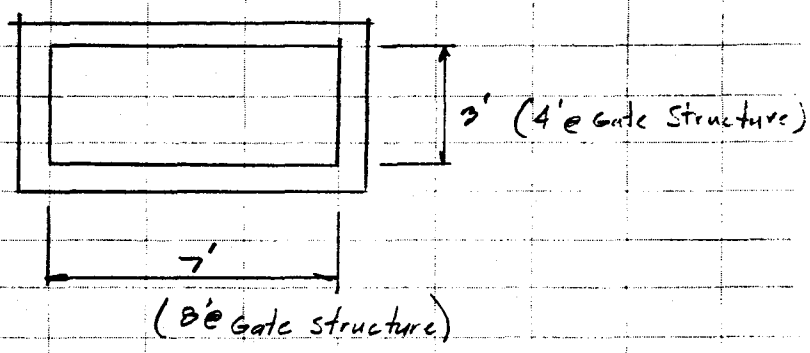
Computed by SGSmith Date 4-4-91
Checked by TURNER Date 4/12/91
Reviewed by Date
Approved by Date

Job No. 10924 Page No. 85
Subject Structural
Outfall Structure
Sheet No. 1 of 1

Outfall structure



Use same design results for the Gate Structure which is very similar



Side wall $p = 90 psf$ $b/a = \frac{7}{9} = 0.78$
Table III PCA

$M_V = 0.024(.090)(9.0)^3 = 1.6$ '-k/'

Corner $M_H = 0.017$ (") " $= 1.1$ '-k/'

Midspan $M_H = 0.010$ (") " $= 0.66$ '-k/'

Try 12" wall: $d_v = 8.5"$ shear: OK
$d_h = 9.5"$

$A_{sv} = \frac{1.6}{1.76(8.5)} = 0.11$ in²/' USE #4 @ 12 FOR ALL BARS.

Base slab (12" thick)

WALL DL = 9' × 24' × 1.0' × 0.15 pcf = 32.4^k
SLAB DL = 9.5 × 5.5 × 0.15 = 7.8^k
40.2^k

Uplift = 8.67 × LOW 4.67 × 9' × .0624 = 22.7^k No net Uplift

Design Pressure: $\frac{32.4^k + 4.2^k}{9.5 \times 5.5}$ = 0.70 KSF (200 psf LL) : Use #4 @ 12 T&B EW

Job No. 10924 Page No. 86

Computed by SCSmith Date 4-10-91
Checked by W E Bragg Date 5-9-91
Reviewed by Date
Approved by Date 5-9-91

Subject Structural

Screenwall

Sheet No. 1 of 2

Masonry Screenwall

Wind per UBC 1988 Ed. (Seismic will not control by inspection)

Basic wind speed = 70 mph

Exposure C

$p = C_e C_q q_s I$

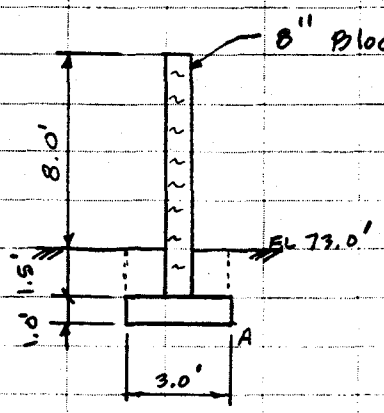
$q_s = 13 psf$ Table 23-F

$C_e = 1.2$ Table 23-G

$C_q = 1.3$ Table 23-H

$I = 1.0$

$\therefore p = 1.2(1.3)(13 psf)(1.0) = 20.3 psf \approx \boxed{21 psf}$



Neglect soil passive resistance

Overturning $M = 21(8)(5.5') = 924^{1-16/1}$ (block)

Stability $M_A = 21(8)(6.5) = 1092^{1-16/1}$

			w (plf)	× Arm	M_A
Block	9.5'(65 psf) OK	=	618	1.5	927
Ftg	1.0'(150 pcf)(3.0')	=	450	1.5	675
Soil	1.5'(100 pcf)(2.33')	=	350	1.5	525
			1418 plf		2127 1-16/1

F.S. overturning $= \frac{2127}{1092} = 1.95 > 1.5$ ok

F.S. Sliding $= \frac{0.3(1418)}{21(8')} = 2.53 > 1.5$ ok

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Computed by SCSmith Date 4-10-91
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Subject Structural

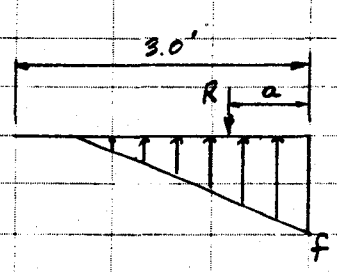
Screenwall

Sheet No. 2 of 2

Screenwall (continued)

Bearing pressure: $e = \frac{1092^{1\text{-}lb/1}}{1418\ plf} = 0.77' > \frac{3.0}{6}$

$a = 1.5 - 0.77 = 0.73 < \frac{3.0}{3}$



$f = \frac{2(1418)}{3(.73)(1.0')} = 1295\ psf$

Per soils report, actual allowable ≅ 2750 psf ok

For 1500 psf capacity, $f_{allow} = 1500 \times 1.33 = 1995\ psf$ ok

1995 psf > 1295 . ok

Reinforcing steel: Ref. SCI Masonry Memo
8" CMU w/o inspection
$f'_m = 1500\ psi$

By inspection of interaction diagram, 8" block is not adequate for a moment = $924 \times 0.75 = 695^{1\text{-}lb/1}$
allow 1/3 incr in allow. stress

Bring conc. fdn up to grade level and recompute block moment:

$M = 21(8)^2/2 = 672 \times 0.75 = 504^{1\text{-}lb/1}$

For 8" CMU, $P = 65\ psf \times 8' = 520\ plf \times 0.75 = 390^{1\text{-}lb/1}$ ok

Then from interaction chart, A_s req'd ≅ $0.10\ in^2/1$
#5 @ 32" o.c. : $A_s = 0.12\ in^2/1$
ok

Use 8" Block w/ #5 @ 32" o.c.

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Computed by S.S. Smith Date 4-4-91
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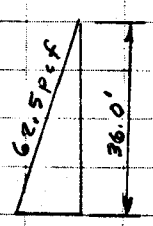
Subject Structural
Pump Station
Sheet No. 1 of 7

Ref. Intake structure for lateral earth pressure comps
$\gamma = 62.5$ pcf (H_2O)
Design for backfill w/o top slab in place
Structure inside dimensions : 18' L x 10' W x 36' D

18' Long side Wall — Ref PCA Table II or III
$b/a = 0.5$

Water or Soil Loading — $.0625(36)^3 = 2916$



Horizontal Moments — See below

				d	As
Corner	$M_{27} = .014 \times 2916$	=	40.8 k'/'	23.5"	0.99 in²/'
	$M_{18} = .010$	=	29.2	15.5	#7@6 1.07
	$M_9 = .005$	=	14.6	9.5	0.87
Midspan	$M_{27} = .008$	=	23.3	23.5	0.56
	$M_{18} = .006$	=	17.5	15.5	0.64
	$M_9 = .005$	=	14.6	9.5	0.87

Vertical Moments

$M_{27} = .007 \times 2916$	=	20.4 k'/'	22.5"		0.52 in²/'
$M_{18} = .002$	=	5.8	14.5	#5@12	0.23
$M_{36} = .015$	=	43.8	22.5	#8@6	1.11
$M_{27} = .004$ / .001	=	11.7 / 2.9	22.5		small 0.2
$M_{18} = .002$	=	5.8	14.5	#5@12	0.23

See base slab

Shear

$V_{Bottom} \doteq 0.141 \times 0.0625 \times (36)^2 = 11.42$ k/'

$V_{side} \doteq 0.1919 \times$ " $\times$ " $= 15.55$ k/' (9' From bottom)

$d \geq \dfrac{15.55(1000)}{1.1\sqrt{4000}\,(12)} = 18.6''$ (MIN) $h = d +$ cover

use 26" thick (Bot. 12') $\left[\begin{array}{l} d_H \doteq 23.5'' \\ d_V \doteq 22.5'' \end{array}\right]$

$V_{side} \doteq 0.128 \times 0.0625 \times (36)^2 = 10.4$ k/' $\Rightarrow d \geq 12.4''$ (min) (18' from bottom)

use 18" thick (Middle 12' section) $\left[\begin{array}{l} d_H = 15.5'' \\ d_V = 14.5'' \end{array}\right]$

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Subject Structural
Pump Station
Sheet No. 2 of 7

For upper 12' of walls use 12" thick walls.

$$\left[\begin{array}{l} d_h = 9.5" \\ d_v = 8.5" \end{array}\right]$$

Use same reinf. on each face of walls.
By inspection, reinf. selected will satisfy load case with top slab in place.

Base Slab

DL Walls $12(2.17)(22.3+10)(2) = 1682\ ft^3$
$12(1.5)(21.0+10)(2) = 1116$
$12(1.0)(20+10)(2) = 720$
$3518 \times .15 = 527.7^k$

Top Slab $18(1.25)(5.0) = 112.5\ ft^3$
$18(0.83)(5.0) = 75.0$
(Beam) $0.42(0.83)(5.0) = 1.7$
$189.2 \times .15 = 28.4^k$

Base Slab $23.3 \times 15.3 \times 2.17 = 774\ ft^3 \times .15 = 116.0^k$
672.1^k

See sh. 6

LL 0.2^{OK} ksf $\times$ 20' x 12' $= 48.0^k$

Uplift force $= 0.0624\ kcf[\ 36.0(18)(10) + 3518 + 22.3(14.3)(2.17)\ ] = 667.2^k$
(interior walls, Base slab)

$F.S. = \frac{672.1}{667.2} = 1.01$

Consider weight of soil on 6" base slab lip
$Wt = 0.5'(34')(14+22)(2)(0.1) = 122.4^k$

$F.S = \frac{672.1 + 122.4}{667.2} = 1.19$ OK - soil wt. will increase due to stepped wall

Uplift should never become a problem with this structure since water level inside should be at least equal to the ground water level.

Base slab design pressure $= \frac{527.4 + 28.4 + 48.0}{23.3 \times 15.3} = 1.7$ KSF

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Subject Structural

Pump Station

Sheet No. 4 of 7

Base Slab (cont'd)

Ref. ACI 318-63 $\frac{A}{B} = \frac{10}{18} = 0.55$

				As
$+M_{SHORT} = .088(1.7)(10)^2 =$	15.0 '-k/'	d = 23.5	#6 @12	0.36 in²/'
$+M_{LONG} = .008(1.7)(18)^2 =$	4.4	d = 22.5	#6 @12	0.11 in²/'
$-M_S = .084(1.7)(10)^2 =$	14.3 *	d = 22.5	#8 @6	1.47
$-M_L = .007(1.7)(18)^2 =$	3.9 *	d = 21.5	Use #8 @6 near wall, #6 @12 mid span.	

* Add wall moment to -M:

$-M_S = 14.3 + 43.8 = 58.1$ '-k/'

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Base Slab

Long direction: Vertical wall moments are small
M from soil pressure on base slab ≅ 4.0 '-k/'

Use #6 @ 12 by inspection for top bars

Lap #6@12 W / #8 @ 6 O.F. wall bars for bottom bars

T & S steel: $A_s = .0018(12)(30)(\frac{1}{2}) = 0.32$ in²/' EF OK

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Subject Structural

Top slab - Pump Station

Sheet No. 6 of 7

Slab Design

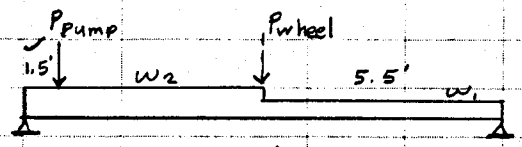
$LL = 200^{OK}$ psf or $10^{k\ OK}$ wheel load ✓

Distribute wheel load over a width $= \frac{2}{3}\ell$ ✓

$= 0.67(11.0') = 7.33'$

Since 7.33' > wheel spacing, use 6' width ✓

Wheel load will control slab design.



$\ell = 11.0'$

R_L	R_R
0.28	0.70 ✓
1.07	0.36 ✓
5.01	0.79 ✓
1.85	1.85 ✓
8.16^k	3.7^k ✓

Design as a one-way slab:

$W_{1DL} = 0.83(0.15)(1.4) = 0.17^{k/'}$ 10" slab

$W_{2DL} = 1.25(0.15)(1.4) = 0.26^{k/'}$ 15" slab @ Pumps

$P_{pump} = 3.4(1.7) = 5.8^k$ ✓

$P_{wheel} = 10(1.3)(1.7)/6 = 3.7^k$ ✓ ✓

$M_u = 3.7(5.5) - 0.17(5.5)^2(\frac{1}{2}) = 17.8^{1-k/'}$

$d = 10 - 1.5 - 0.5 = 8"$ ✓

$A_{s\,req'd} = 0.52\ in^2/'$ > $A_{min} = \frac{6d}{300}$ ✓ ok

Try #5@6 $A_s = 0.62\ in^2/'$

Crack Control:

$d_c = 1.5" + \frac{0.625}{2} = 1.81"$

$A = 2(1.81)(6) = 21.75\ in^2$

ACI commentary R10.6.4

$z = 36\sqrt[3]{21.75(1.81)} = 123 < 145\left(\frac{1.2}{1.35}\right) = 129$ ok ✓ Use #5@6 Bottom

Use #4@9 Top

T&S steel

$A_s = .0018(12)(10) = 0.216(\frac{1}{2}) = 0.11\ in^2/'$ EF

$= \quad " \quad " \quad (15) = 0.324(\frac{1}{2}) = 0.16\ in^2/'$ EF

Use #4@12 T&B

Slab Design continued

(wheel load at edge)

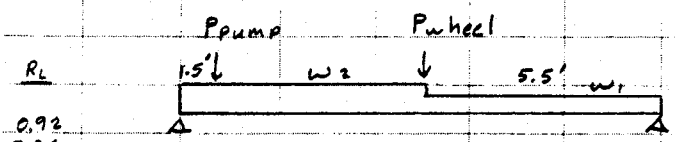
$\phi V_c = 0.85(2)\sqrt{4000}\,(12)(8")/1000 = 10.3^{k/'} > 3.7 + 1.85$ ok

$\phi V_c =$ " " $(13")/1000 = 16.8^{k/'} > 8.16 + 1.85$ ok

Shear is ok

Beam Design

Hatch wt ≅ 650 # (High) OK



R_L	R_R
0.92	2.72
5.94	1.98
3.38	0.53
3.69	3.69
13.93 k	8.92 k

slab Bm

$W_{1DL} = [0.83(3.0) + 0.83(0.42)](0.15)(1.4) = 0.60^{k/'}$ OK

$W_{1DL} = 0.65 \times \frac{1}{2} / 5.0$ (Hatch) $= 0.07^{k/'}$ } $0.67^{k/'}$

$W_{2DL} = 1.25\left(\frac{5.0}{2} + 3.0\right)(0.15)(1.4) = 1.44^{k/'}$

$P_{pump} = 2.3(1.7) = 3.91^{k}$

$P_{wheel} = 10^{k}(1.3)(1.7)/3' = 7.37^{k}$

$M_u = 8.92(5.5) - 0.67(5.5)^2(\frac{1}{2}) = 39^{'k}$

$d = 15 - 1.5 - 0.5 - \frac{.75}{2} = 12.6"$

$b = 10"$

A_s req'd $= 0.73\ in^2 > A_{s\,min} = \frac{bd}{300}$ ok

Use 3 - #5 Bottom w/ #3 @ 6 ties

$M_{CAP} = 48^{'k}$
$V_{CAP} = 37^{k}$

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Computed by G.J.Fink Date 4-8-91
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Subject Pump Station Pump Weights
Sheet No. 1 of 1

Irrigation Pumps

Use Fairbanks-Morse 8M 7000 as a guide
3600 rpm

Bowl Wt (1st)	82	lb.
" (2nd)	31	
Column (20#/ft)(35')	700	
Shaft (5#/f)(35)	175	
Discharge Head	350	
Motor	500	
	1838	

Use 2000# equipment.

Include 300# of water

Flood Pumps

Use Fairbanks Morse 10" 8312 as a guide
1800 rpm.

Bowl Wt.	100
Column 10" (31#)(35')	1085
Shaft + Enclosing tube (15#)(35')	525
Discharge Head	400
Motor	500
	2610

Use 2600# equipment

Include 800# of water

3004

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Computed by SC Smith Date 5-8-91

Checked by W. E. [illegible] Date 5-9-91

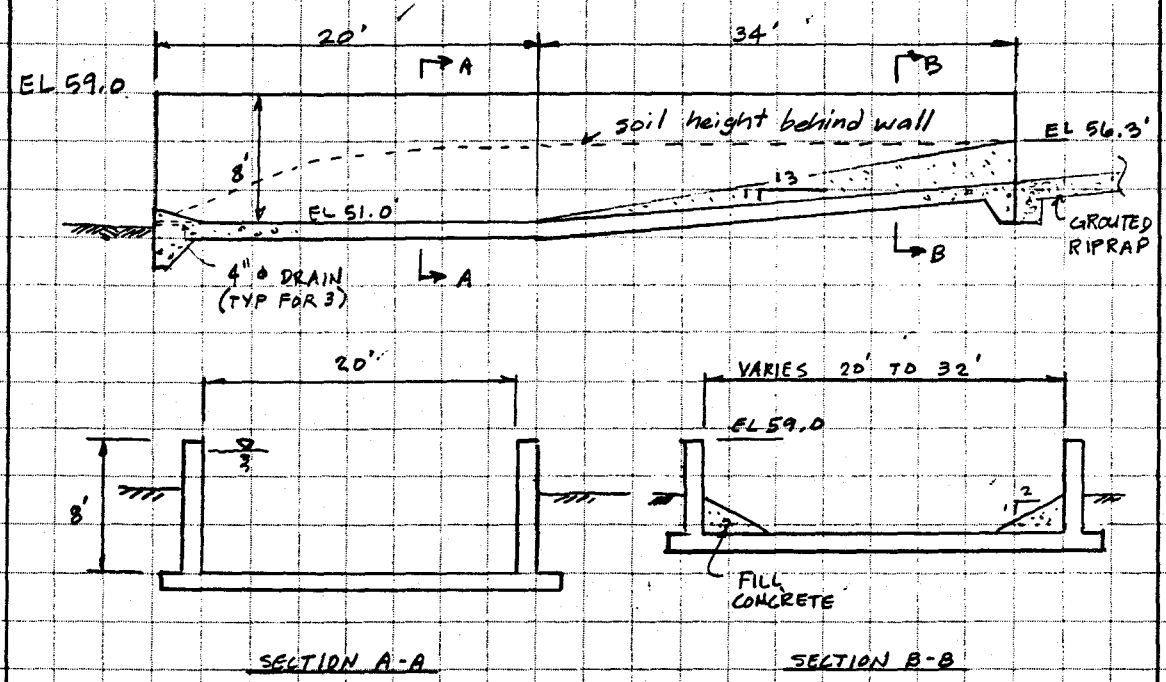
Reviewed by [illegible] Date

Approved by W. E. [illegible] Date 5-9-91

Subject Structural

Inlet structure No. 3

Sheet No. 1 of 2



Lateral earth pressure

$p \cong 0.33 \times 110\ pcf = 36.6$ Use 35 psf/ft as recommended by soils report.

active coeff.

Soil height will be approximately 3' to 3.5' below top of inlet walls.

Water inside could reach full depth with no water on outside of walls during heavy rains

(~ 2' soil)

Include a surcharge soil load of 70psf for trucks or equipment near wall and to account for compaction forces during backfill operations.

Pressure due to saturated backfill:

$p = 0.33\,(120 - 62.4) + 62.4 = 81\ psf/ft$

Do not combine this load with surcharge load

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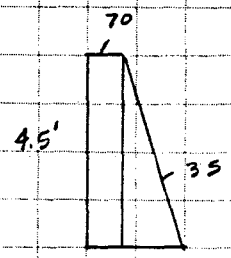
Subject Structural
Inlet structure No. 3
Sheet No. 2 of 2

Compute equivalent loading due to soil + surcharge



$70(4.5) + 35(4.5)^2(\frac{1}{2}) = w(4.5)^2(\frac{1}{2})$

$w = 66$ psf/ft < 81

Use 81 psf/ft for soil pressure w/o H_2O inside channel

Stem moments

$M_{soil} = 0.081(5.0)^3/6 \times 1.7 = 2.9$ '-k/'

$M_{H_2O} = 0.0624(8.0)^3/6 \times 1.7 = 9.1$ '-k/' (Neglect passive soil pressure)

$V_{max} = 1.7(0.0624)(8.0)^2(\frac{1}{2}) = 3.4$ k/'

Try 10" walls:

$\phi V_c = 0.85(2)(\frac{\sqrt{4000}}{1000})(12)(7.5) = 9.7^k > 3.4$ ok

Vertical reinforcing

			Use:
OF	A_s req'd = 0.09 in²/'	(M = 2.9 '-k/')	#4@12 OF
IF	A_s req'd = 0.28 in²/'	(M = 9.1 '-k/')	#5@12 IF

Base Slab

$M_{top} = 9.1$ '-k/' max

For 12" slab, $d_{top} \cong 9.5"$

A_s req'd = 0.22 in²/' Use: #5@12 Top
#4@12 Bottom

T&S steel: $.0018(12)(12)(\frac{1}{2}) = 0.13$ in²/' EF

Use: #4@12 T&B Longitudinal

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Computed by SCSmith Date 5-8-91

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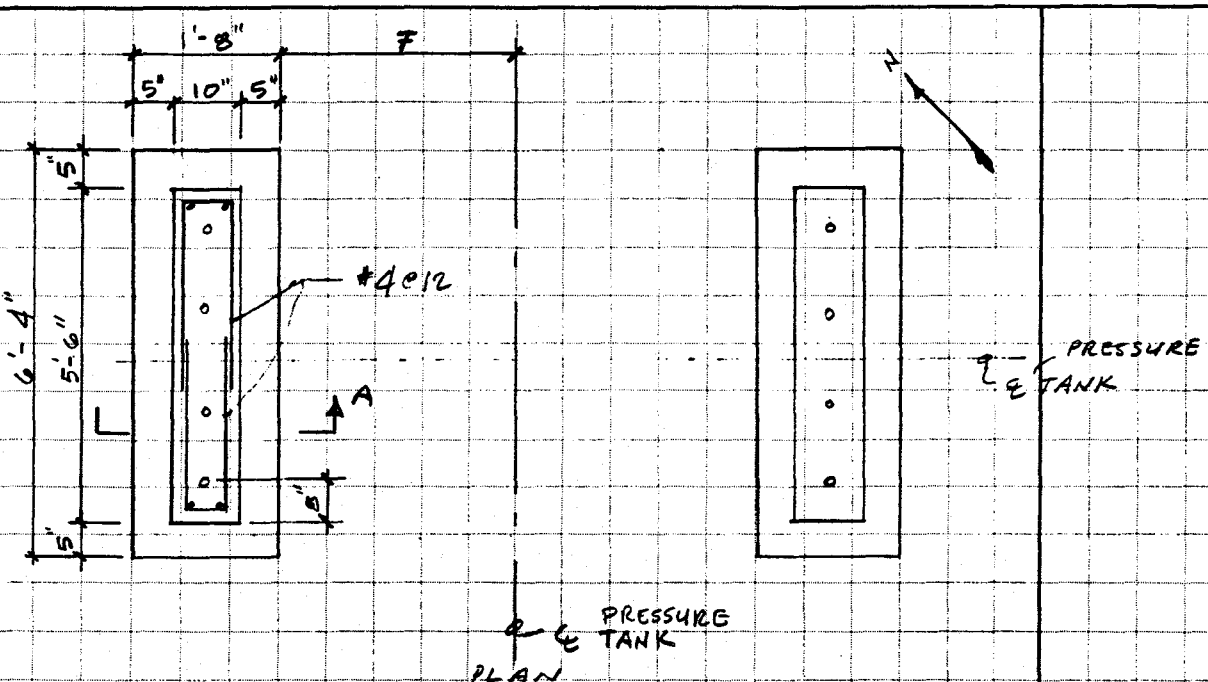
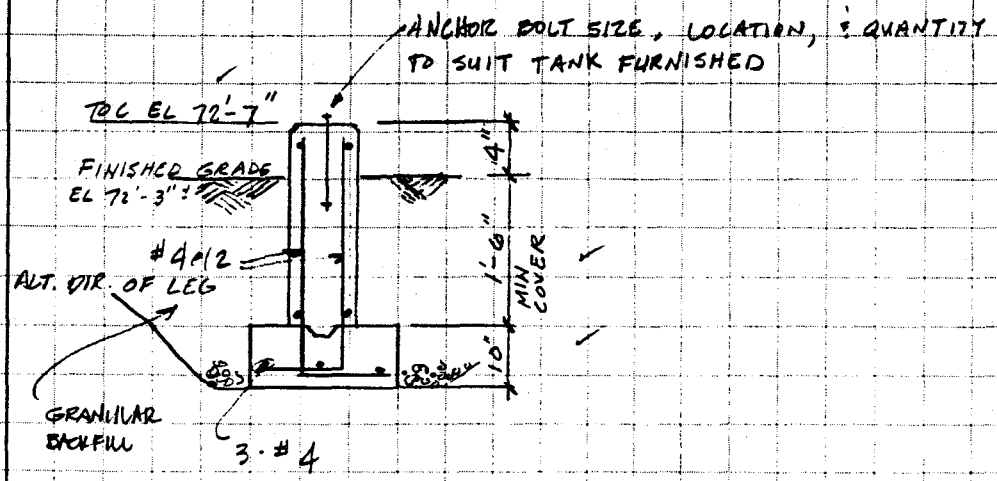
Subject structural

Pressure Tank Fdn

Sheet No. 1 of 1



PLAN



SECTION A

Tank: 5'φ x 15' long

Est wt: 1/4" steel plate assumed

Body: $5\pi(12)(3.4)(15') = 2400^{\#}$

Ends: $\pi(2.5)^2(1/4")(\frac{1}{12})(490) = 200 \times 2 = 400^{\#}$

Total - use $3500^{\#}$: Brg pressure OK by Inspection.

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Computed by D. GOETZ Date 5/10/91
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Subject GILBERT-DETENTION BASIN SIZING FEEDER BREAK AND FEEDER CIRCUITS

Sheet No. of

USE 75°C THWN

FOR FIELD #1,2 LIGHTS 22 FIXTURES TOTAL LOAD 37.4 KW

$\frac{37.4\ KW}{480\sqrt{3}} = 44.98(1.25) = 56.23\ A$

CIRCUIT BREAKER SIZE 100 ~~60~~ A 100

CONDUCTOR SIZE 4 ~~3~~ # ~~6~~ 2 1½ ~~3/4~~ "C

FOR FIELD #3,4 LIGHTS 50 FIXTURES TOTAL LOAD 85 KW

$\frac{85\ KW}{480\sqrt{3}} = 102.23(1.25) = 127.79\ A$

CIRCUIT BREAKER SIZE : 150 ~~125~~ A

CONDUCTOR SIZE : 4 ~~7~~ - # 1/0, 1½ "C

PARKING LOT LIGHTS 33 FIXTURES TOTAL LOAD = 9.9 KW

$\frac{9.9\ KW}{480} = 20.6(1.25) = 25.78\ A$

CIRCUIT BREAKER SIZE : 100 ~~30~~ A 100

CONDUCTOR SIZE 4 ~~7~~ - # ~~10~~ 2, ½ "C

MINI POWER ZONES 1R, 3R (2 WITHIN 20 FT OF MAIN PANELS)

$\frac{25\ KVA}{480} = 52.08(1.25) = 65.1\ A$ 15 KVA use 40A & 2 - #8, 3/4

CIRCUIT BREAKER SIZE 60 ~~65~~ A

CONDUCTOR SIZE - 2 ~~7~~ - #6, 3/4 "C

MINI POWER ZONE 2R (WITH 1040 FT VOLTAGE DROP)

$\frac{25\ KVA}{480}$ USE 3 - #3, 1 "C

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Job No. 10924 Page No. 100
Subject SILBERT-DETENTION BASIN
SIZING FEEDER BREAKER
AND FEEDER CIRCUITS
Sheet No. of

USE 75°C THWN

FOR FLOOD PUMPS 30HP = 22.38 KW

$\frac{22.38\,KW}{480\sqrt{3}} = 26.9A(1.75) = 47.12\,A$

~~CIRCUIT~~ fuse BREAKER SIZE: ~~50~~ 70 A

CONDUCTOR SIZE: 4~~3~~ - #~~8~~ 6 AWG, 3/4" C

FOR IRRIGATION PUMPS 40 HP = 29.84 KW

$\frac{29.84}{480\sqrt{3}} = 35.9(1.75) = 62.81\,A$

~~CIRCUIT~~ fuse BREAKER SIZE ~~70~~ 90 A

CONDUCTOR SIZE: 4~~3~~ - #~~6~~ 4 AWG, ~~3/4~~ 1" C

FOR AERATOR BLOWER 15HP = 11.19 KW

$\frac{11.19\,KW}{480\sqrt{3}} = 13.46(1.75) = 23.55\,A$

~~CIRCUIT~~ fuse BREAKER SIZE: ~~25~~ 35 A

CONDUCTOR SIZE: 4~~3~~ - #~~12~~ 10, ~~1/2~~ 3/4" C

FOR COMPRESSOR 5HP = 3.73 KW

$\frac{3.73\,KW}{480\sqrt{3}} = 4.49(1.75) = 7.85\,A$

~~CIRCUIT~~ fuse BREAKER SIZE: ~~20~~ 15 A

CONDUCTOR SIZE: 4~~3~~ - #12, ~~1/2~~ 3/4"

FOR JOCKEY PUMP 3HP = 2.24 KW

$\frac{2.24\,KW}{480\sqrt{3}} = 2.69(1.75) = 4.71\,A$

~~CIRCUIT~~ fuse BREAKER SIZE: ~~20~~ 10 A

CONDUCTOR SIZE: 4~~3~~ - #12, ~~1/2~~ 3/4" C

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Job No. 10924 Page No. 101
Subject GILBERT-DETENTION BASIN
SIZING FEEDER BREAKERS
AND FEEDER CIRCUITS
Sheet No. of

PANEL 1R

RECEPTACLES 20A 3-#12

FLAGPOLE LIGHTING 2 @ 350W = $\frac{700W}{120}$ = 5.83A

CIRCUIT BREAKER 20 A

CONDUCTOR 3-#12, 1/2"C (NOT INCLUDING VOLTAGE DROP)

BALLFIELD SCOREBOARD $\frac{1KW}{120}$ = 8.3A

CIRCUIT BREAKER 20 A

CONDUCTOR 3-#12, 1/2"C

SPRINKLER SYSTEM $\frac{2KW}{120}$ = 16.6A

CIRCUIT BREAKER

CONDUCTOR 3-#12, 1/2"C (NOT INCLUDING VOLTAGE DROP)

LIGHTS FOR LIGHTING EQUIPMENT AREA 150W HIGH PRESSURE SODIUM

2 FIXTURES @ 200W = $\frac{400W}{120}$ = 3.33A

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Job No. 10924 Page No. 102
Subject GILBERT-DETENTION BASIN
SIZING FEEDER BREAKS
AND FEEDER CIRCUITS
Sheet No. of

PANEL 2R

RECEPTACLES 20A 3-#12 ½"C

AMPITHEATER LIGHTING 2 @ 500W = 1KW

$\frac{1KW}{240} = 4.2A$

CIRCUIT BREAKER 20A

CONDUCTOR 3-#12 ½"C (NOT INCLUDING VOLTAGE DROP

Job No. 10924 Page No. 103

Computed by D. GOETZ Date 5/10/91
Checked by V McAnsen Date 5/17/91
Reviewed by DP [illegible] Date 5/20/91
Approved by Date

Subject GILBERT -- DETENTION BASIN LOAD COMPUTATION FOR LIGHTING TRANSFORMER

Sheet No. 1 of 1

LIGHTING TRANSFORMER LOADS:

1. LITTLE LEAGUE FIELD LIGHTING	2@ 35.2 KW	70.4 KW
2. BABE RUTH LEAGUE FIELD LIGHTING	2@ 80 KW	160.0 KW
3. PARKING LOT LIGHTING		15 KW
4. FLAGPOLE AND SCOREBOARDS		5 KW
5. AMPHITHEATER		20 KW
6. CONTROLS FOR WATER METERING		1 KVA
7. SPRINKLER SYSTEM		2 KW
8. CONCESSION AND STORAGE BLDGS.		20 KW
9. SOCCER FIELD LIGHTING		231 KW
TOTAL KW LOAD		524.4 KW

WITH SECONDARY 480V ⇒ 631.5 FLA

CHOOSE 500 KVA TRANSFORMER ⇒ 600 FLA x 1.25 = 750 FLA

Job No. 10774 Page No. 104

Computed by D. GAFTZ Date 5/10/91
Checked by [illegible] Date 5/17/91
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Approved by Date

Subject GILBERT DETENTION BASIN
PUMP STATION TRANSFORMER
LOAD COMPUTATION

Sheet No. 1 of 1

PUMP STATION TRANSFORMER LOAD

1.	FLOOD PUMPS	2 @ 30HP	45 KW
2.	IRRIGATION PUMPS	2 @ 40HP	60 KW
3.	AERATOR BLOWER	15 HP	11.2KW
4.	COMPRESSOR	5HP	3.7KW
5.	JOCKEY PUMP	3HP	2.3 KW
6.	CONTROL AND MISCELLANEOUS POWER	5KVA	5KVA

TOTAL KW LOAD $\boxed{127.2\ KW}$

WITH SECONDARY 480V ⇒ 153.2 FLA

CHOOSE 150 KVA ⇒ 180 FLA × 1.25 = $\boxed{225\ FLA}$

Job No. 10924 Page No. 105

Computed by D.K. GOETZ Date 5/7/91
Checked by V.M. ALLISON Date 5/17/91
Reviewed by DP [signature] Date 5/20/91
Approved by _____ Date _____

Subject ELECTRICAL SHORT CIRCUIT STUDY

Sheet No. 1 of 1

1) PUMP STATION

SERVICE TRANSFORMER: 300 KVA, Z = 5.75%

UTILITY S.C. CAPACITY: ASSUME INFINITE

TRANSFORMER S.C. CONTRIBUTION: $\frac{300,000}{480 \times 1.73 \times .0575} = 6,276A$

MOTOR LOAD: ASSUME 100%

MOTOR CONTRIBUTION: $\frac{300A}{.25} = 1,200$ A

TOTAL S.C. CONTRIBUTION 7476 A

USE MINIMUM 20,000 A INTEGRATED S.C. RATING

2) LIGHTING

SERVICE XFMR: 500 KVA Z = 5.75%

UTILITY S.C. CAPACITY: ASSUME INFINITE

TRANSFORER SC. CONTRIBUTION $\frac{500,000}{480 \times 1.73 \times .0575} = 10,460 A$

LIGHTING LOAD. ASSUME 100% 605 A

TOTAL SC CONTRIBUTION 10,460 A

USE MINIMUM 15,000 A INTEGRATED S.C. RATING.

Job No. 10924 Page No. 106

Computed by D GOETZ Date 5/10/91
Checked by [illegible] Date 5/19/91
Reviewed by [illegible] Date 5/20/91
Approved by Date

Subject: VOLTAGE DROP CALCULATIONS FOR BALLFIELD LIGHTING

Sheet No. 1 of 4

1) FOR POLE A1, 2 CIRCUITS ASSUME 6 LUMINARES PER CIRCUIT

NUMBER X (WATTS PER LUMINAIRE) = TOTAL WATTS

$6 \times 1700W = 10.2KW$

$\frac{10.2KW}{480\sqrt{3}} = 12.3A$

$V = \frac{2K(I)(L) \times .866}{d^2}$

#6 AWG WIRE IS GOOD

ASSUME USING #6 WIRE

$V = \frac{2(11)(12.3)(340)(.866)}{26240} = 3.04V$

VOLTAGE DROP $= \frac{3.04}{480} = 0.63\%$

2) FOR POLE A2 2 CIRCUITS ASSUME 6 LUMINARES PER CIRCUIT

$V = \frac{2(11)(12.3)(300)(.866)}{26240} = 2.7V$

VOLTAGE DROP $\frac{2.7}{480} = 0.56\%$

#6 AWG WIRE IS GOOD

3) FOR POLES A3, A4 2 CIRCUITS PER POLE ASSUME 6 LUMINAIRES PER CIRCUIT

$V = \frac{2(11)(12.3)(450)(.866)}{26240} = 4.02V$

VOLTAGE DROP $= \frac{4.02}{480V} = 0.84\%$

#6 AWG WIRE IS GOOD

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Job No. 10924 Page No. 107

Computed by D. GOETZ Date 5/10/91
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Approved by Date

Subject GILBERT - DETENTION BASIN
VOLTAGE DROP CALCULATIONS
FOR BALLFIELD LIGHTING

Sheet No. 2 of 4

1) FOR POLE B1 1 CIRCUIT 12 FIXTURES PER POLE

$12(1700W) = 20.4KW$

$\frac{20.4KW}{480\sqrt{3}} = 24.5A$

$V = \frac{2(11)(24.5)(150)(.866)}{26240} = 2.7V$

#6 AWG WIRE IS GOOD

VOLTAGE DROP $= \frac{2.7}{480} = 0.56\%$

2) FOR POLE B2, 2 CIRCUITS 12 FIXTURES PER POLE

$V = \frac{2(11)(24.5)(560)(.866)}{26240} = 9.96V$

#6 AWG WIRE IS GOOD

VOLTAGE DROP $= \frac{9.96}{480} = 2.08\%$

3) FOR POLE B3, 1 CIRCUIT 12 FIXTURES

$V = \frac{2(11)(24.5)(580)(.866)}{26240} = 10.32V$

#6 AWG WIRE IS GOOD

VOLTAGE DROP $= \frac{10.32}{480} = 2.15\%$

2004

Computed by D. GOETZ Date 5/10/91
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Job No. 10724 Page No. 108
Subject GILBERT DETENTION BASIN
VOLTAGE DROP CALCULATIONS
FOR PARKING LIGHTING
Sheet No. 3 of 4

1) FOR POLE C1, 1 CIRCUIT 6 FIXTURES

$6(1700W) = 10.2kW$

$\frac{10.2kW}{480\sqrt{3}} = 12.3A$

$V = \frac{2(11)(12.3)(450)(.866)}{26240} = 4.02V$

#6 AWG WIRE IS GOOD

VOLTAGE DROP $= \frac{4.02}{480} = 0.84\%$

2) FOR POLE C2 - 1 CIRCUIT 6 FIXTURES

$V = \frac{2(11)(12.3)(80)(.866)}{26240} = 0.714V$

#6 AWG WIRE IS GOOD

VOLTAGE DROP $= \frac{0.714}{480} = 0.15\%$

3) FOR POLE C3, 1 CIRCUIT, ASSUME 9 FIXTURES

$9(1700W) = 15.3kW$

$\frac{15.3kW}{480\sqrt{3}} = 18.4A$

$V = \frac{2(11)(18.4)(250)(.866)}{26240} = 3.34V$

VOLTAGE DROP $= \frac{3.34}{480} = 0.7\%$

4) FOR POLE C4, 1 CIRCUIT, ASSUME 9 FIXTURES

$V = \frac{2(11)(18.4)(450)(.866)}{26240} = 6.01V$

VOLTAGE DROP $= \frac{6.01}{480} = 1.25\%$

Job No. 10921 Page No. 109

Computed by D. GOETZ Date 5/10/91
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Subject GILBERT - DETENTION BASIN
VOLTAGE DROP CALCULATIONS
FOR BALLFIELD LIGHTING

Sheet No. 4 of 4

1) FOR POLE C5, 1 CIRCUIT, ASSUME 9 FIXTURES

$$V = \frac{2(11)(18.4)(780)(.866)}{26240} = 10.42$$

#6 AWG WIRE IS GOOD

$$\text{VOLTAGE DROP} = \frac{10.42}{480} = 2.17\%$$

2) FOR POLE C6, 1 CIRCUIT, ASSUME 9 FIXTURES

$$V = \frac{2(11)(18.4)(800)(.866)}{26240} = 10.68\ V$$

#6 AWG WIRE IS GOOD

$$\text{VOLTAGE DROP} = \frac{10.68}{480} = 2.23\%$$

3) FOR POLE C7, 1 CIRCUIT, 6 FIXTURES

$$V = \frac{2(11)(12.3)(640)(.866)}{26240} = 5.72\ V$$

#6 AWG WIRE IS GOOD

$$\text{VOLTAGE DROP} = \frac{5.72}{480} = 1.2\ \%$$

4) FOR POLE C8, 1 CIRCUIT, 6 FIXTURES

$$V = \frac{2(11)(12.3)(520)(.866)}{26240} = 4.64\ V$$

#6 AWG WIRE IS GOOD

$$\text{VOLTAGE DROP} = \frac{4.64}{480} = 0.97\ \%$$

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Job No. 10724 Page No. 110

Computed by DGoetz Date 5/10/91
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Approved by ..... Date .....

Subject GILBERT - DETENTION BASIN
VOLTAGE DROP CALCULATION
FOR PARKING LOT [illegible]

Sheet No. ..... of .....

1) CIRCUIT 1P-1 13 @ 250 W

13(300) = 3.9 KW

$\frac{3.9\text{ KW}}{480\text{ V}} = 8.125\text{A}$

$V = \frac{2K(L)(I)}{d^2} = \frac{2(11)(1300)(8.125)}{26240} = 8.86\text{V}$

VOLTAGE DROP = $\frac{8.86}{480}$ = 1.84%

#6 AWG WIRE IS GOOD

2) CIRCUIT 1P-2 10 FIXTURES

10(300) = 3 KW $\frac{3}{480}$ = 6.25A

$V = \frac{2(11)(920)(6.25)}{26240} = 4.82\text{V}$

VOLTAGE DROP = $\frac{4.82}{480}$ = 1%

#6 AWG WIRE IS GOOD

3) CIRCUIT 1P-3 8 FIXTURES

8(300) = 2.4 KW $\frac{2.4}{480}$ = 5.0 A

$V = \frac{2(11)(1,100)(5.0)}{26240} = 4.61\text{ V}$

VOLTAGE DROP = $\frac{4.61}{480}$ = 0.96%

#6 AWG WIRE IS GOOD

4) CIRCUIT 1P-4 L = 1040 ft MAXIMUM LOAD ~~1.25~~(25 KVA) = ~~31.25 KV~~
(TO MINIPOWER ZONE AT AMPHITHEATER) 25 KVA / .48 = 52.1

$\frac{31.25\text{ KW}}{480\text{ V}}$ = ~~37.6~~A 65A

$V = \frac{2(K)(I)(L)(.866)}{d^2} = \frac{2(11)(\cancel{37.6}\ 52.1)(1040)(.866)}{52620} = \cancel{14.16}\text{V}$ 19.62

USE #2

VOLTAGE DROP = $\frac{14.16}{480}$ = 2.95%

#2 AWG WIRE IS GOOD

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Reviewed by: [illegible] Date: 5/20/91
Approved by: Date:

Job No.: Page No. 111
Subject: VOLTAGE DROP FOR SCOREBOARDS
Sheet No.: of:

1) SCOREBOARD FIELD #1 240 ft $\frac{1kW}{120} = 8.33\ A$

$$V = \frac{2(11)(240)(8.33)}{26240} = 1.7V$$

$$\text{VOLTAGE DROP} = \frac{1.7V}{120\ V} = 1.4\%$$

#6 AWG WIRE IS GOOD

2) SCOREBOARD FIELD #2 880 ft

$$V = \frac{2(11)(880)(8.33)}{41740} = 3.51$$

$$\text{VOLTAGE DROP} = \frac{3.51}{120V} = 2.93\ \%$$

#4 AWG WIRE IS GOOD

3) SCOREBOARD FIELD #3 940 ft

$$V = \frac{2(11)(940)(8.33)}{52620} = 3.3\ V$$

$$\text{VOLTAGE DROP} = \frac{3.3}{120} = 2.7\%$$

#3 AWG WIRE IS GOOD

4) SCOREBOARD FIELD #4 380'

$$V = \frac{2(11)(380)(8.33)}{26240} = 2.65$$

$$\text{VOLTAGE DROP} = \frac{2.65}{120} = 2.2\ \%$$

#6 AWG WIRE IS GOOD

5) FLOODLIGHTS 100 ft 700 W $\frac{700W}{120V} = 5.83$

$$V = \frac{2(11)(100)(5.83)}{26240} = 0.5\ V$$

$$\text{VOLTAGE DROP} = \frac{0.5}{120} = 0.42\ \%$$

#6 AWG WIRE IS GOOD

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Job No. _____ Page No. 112
Subject VOLTAGE DROP
AMPLIHEATER LIGHTS
Sheet No. _____ of _____

AMPLIHEATER 2 @ 500 W (INCLUDING BALLAST LOSS)

$$\frac{1\,KW}{120} = 8.33\,A$$

$$V = \frac{2(11)(8.33)(205\,ft)}{26240} = 1.4\,V$$

#6 AWG WIRE IS GOOD

$$\text{VOLTAGE DROP} = \frac{1.4}{120} = 1.16\%$$

Computed by TDC Date 4/11/91
Checked by AL Date 5-21-91
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Job No. 10924 Page No. 113
Subject Crossroads Park
Sheet No. 1 of 5

Determine runoff from Park Entrance Road

Roadway length ≈ 950 LF
Highpoint is ±60' west of Greenfield Rd
⇒ Drainage Length ≈ 890'
⇒ width = 35' PC to FC

Area = 31,150 SF = 0.72 Acres

R/W width = 65'

Area of R/W = (65'-35') × 890' = 26,700 SF = 0.61 Acres

Determine Weighted "C" Value

Asphalt C ≈ 0.95
Landscape C ≈ 0.70

$$C_w = \frac{(0.95)(0.72) + (0.70)(0.61)}{1.33} = 0.835$$

Use $C_w \approx 0.85$

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Job No. 10724 Page No. 114

Computed by TDC Date 4/11/91

Checked by AC Date 5-21-91

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Subject Crossroads Park

Sheet No. 2 of 5

Determine T_c

Travel Length $\approx 890'$

Assume Curb Velocity $= 2'/s$

$890' \div 2'/s \div 60 = 7.4$ min. $\Rightarrow$ Use 10 min.

Local / Collector Streets Must Maintain Flow Within The Curbs

Using Phoenix Storm Drain Manual

Pg 24, Using $T_c = 10$ min, $i_{10} = 3.85$ in/hr

Calculate Runoff From Rational Method

$$Q = CIA$$

$$Q = (0.85)(1.33)(3.85)$$

$$Q_{10} = \underline{4.35 \text{ CFS}}$$

3004

Job No. 10924 Page No. 115

Computed by TPC Date 4/11/91

Checked by AC Date 5-21-91

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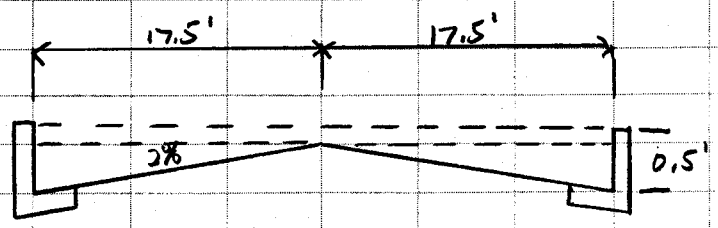
Subject Crossroads Park

Sheet No. 3 of 5

Determine Capacity of Local / Collector Street @ T.O. 6's Minimum Slope of 0.0025'/'



$$Area = (2)(\tfrac{1}{2})(.35)(17.5) + (35)(.15) = 11.38\ SF$$

$$P = (2)(.5) + (2)\sqrt{(17.5)^2 + (.35)^2} \approx 36'$$

$$R = A/P = 11.38 / 36 = 0.32$$

Use Mannings Formula

$$Q = \frac{1.49}{N}(R)^{2/3}(A)(S)^{1/2}$$, Use N = 0.016 - Asphalt

$$Q = \frac{1.49}{.016}(0.32)^{2/3}(11.38)(.0025)^{1/2}$$

$$Q_{CAP} = 24.78\ CFS > 4.35\ \underline{OK}$$

3004

Job No. 10924 Page No. 116

Computed by TDC Date 4/11/91

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Subject Crossroads Park

Sheet No. 4 of 5

From Pg 40, City of Phoenix Storm Drain Manual, The Capacity of a M-1, L=0' Catch Basin with a sump of 0.5' is ± 2.4 CFS. > 2.2 CFS OK

The City of Phoenix has essentially quit using M1, L=0 catch basins. For a very small amount of additional money the M1, L=3' catch basin can be constructed allowing greater capacity and additional clogging protection.

We are using M.A.G.S.D. catch basins. The comprable MAG catch basin to a C.O.P, M1, L=3 is the M.A.G.S.D. 531, Type B, which has an inlet opening of 5'-6".

Use M.A.G.S.D. 531, Type B Catch Basins

3004

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Looking On

Station

z / n = So =

Preliminary INLET NO. Pg. PLANS No.

	Total Area Ac.
	Runoff C
	A imp. Ac.
	Street S. ft./ft.
	Max Dist. Ft.
	T_c Min
	i-.2 in/hr.
From Separate Calcs Q_{TOT} = 4.35	Q = .9 x (i-.2) x Ai cfs.
Q (each C.B.) = 4.35 ÷ 2 ≈ 2.2 CFS	Flow by Q 0 cfs.
☐ Find depth required to operate ☐ grate only	Total Q 2.2 cfs.
☐ See Chart Pg.	Inlet Type MAG. SD. 531 5'-6" CB TYPE B
	O in cfs.
By Design, Create 0.5' Sump Just East of Parking Lot	O Flow by cfs.
	To inlet No.
From Pg 26, S.D. Manual (DEPTH)	Y 0.28 ft.
0.28 ÷ .02 (SPREAD)	T on: 14' ft.
(SPREAD)	T on: ft.
EL. HGL: EL. AT LIP:	Available Head ft.
Q = ☐ Manning Equation	Length Conn. Pipe ft.
S = ☐ By Inspection	
V =	Dia. Concrete Conn. Pipe in.
From Pg 40, S.D. Manual M-1, L=3' = 6' Opening ≈ 5'-6" County C.B. Capacity of M-1, L=3' @ 0.5' Depth (Sump) ≈ 4.9 CFS, use 5-6" Opening	Depth of Catch Basin ft.
	Design By: TRC Check by: AL
	Date 4-1-91 Pg. 5 of 5

Job No. 10924 Page No. 117

Computed by TOC Date 5/14/91 Subject Cross Creek Park

Checked by AC Date 5-21-91

Reviewed by Date

Approved by Date Sheet No. 1 of 1

Determine runoff from parking lot for each of the two proposed catch basins

Area of contributing area for each inlet is 1/2 of the parking lot.

By measure 1/2 of the parking lot is

$$150 \times 370 = 55,500$$
$$+ 135 \times 290 = 39,150$$
$$+ [(440 \times 440) - (\pi (220)^2](1/4) = 10,327$$
$$105,077 \text{ ft}^2$$
$$= 2.41 \text{ Acres}$$

From previous roadway drainage calcs $i = 3.85$ "/hr

Calculate Runoff By Rational Method

$$Q = CIA$$

$$Q = (0.95)(3.85)(2.41)$$
$$= 8.8 \text{ CFS each catch basin}$$

FROM COP STORM DRAIN MANUAL THE CAPACITY OF A GRATED INLET IN 0.5 FT SUMP IS 3.5 CFS/SF

AREA REQUIRED ASSUMING 50% CLOGGING

$$A = \frac{8.8}{3.5 \times 0.5} = 5.0 \text{ Ft}^2$$

USE CATCH BASIN MAG DET. 537, TYPE G, DOUBLE GRAT

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Job No. 10924 Page No. 118

Computed by TDC Date 5/18/91

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Subject Crossroads Park

Sheet No. 1 of 3

Determine Storm Drain Pipe Requirements

Pipe From North C.B. on entrance road to South C.B.

Use 15" pipe minimum size

For 15" pipe, $A = 1.227\ ft^2$

Design pipe to flow @ 5'/s flowing full

$Q = AV$

$Q = 1.227(5)$

$Q = 6.1\ CFS > 2.2\ CFS$ OK

Determine Minimum Slope Req'd

From Fig 2, TOG Public Works Procedure Manual

$K = 65$, where $S_f = (Q/K)^2$

$S_{min} = \left(\frac{6.1}{65}\right)^2 = \underline{\underline{0.0088'/ft}}$

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Job No. 10924 Page No. 119

Computed by TDC Date 5/18/91

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Subject Crossroads Park

Sheet No. 2 of 3

Pipe From South C.B. to East Parking Lot C.B.

Q = Runoff From Entrance Road = 4.4 CFS

From Calcs on Pg 2,

15" Pipe @ 0.0088' Capacity = 6.1 CFS

6.1 CFS > 4.4 ⇒ Use 15" pipe

Pipe From East Parking Lot CB to West Parking lot CB

Because of Short Run In Pipe, Ignore Travel Time In Pipe and Sum Flows Calculated To Size Catch Basins

Q = 4.4 (Roadway) + 8.8 (East Parking Lot) = 13.2 CFS

For 24" Pipe, A = 3.142 ft²

Design Pipe to Flow @ 5'/s min flowing full

Q = AV

Q = 3.142(5)

Q = 15.7 CFS > 13.2 CFS OK

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Job No. 10924 Page No. 120

Subject Crossroads Park

Computed by TDC Date 5/18/91

Checked by AC Date 5-21-91

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Sheet No. 3 of 3

Determine Minimum Slope Req'd

From Fig. 2, For 24" pipe K = 226

$$S_{min.} = \left(\frac{15.7}{226}\right)^2 = \underline{\underline{0.0048'/ft}}$$

Pipe From West Parking Lot CB to Lake Outfall

Again Ignore Pipe Travel Time and Sum Flows @ Catch Basins

Q = 4.4 + 8.8 + 8.8 (West Parking Lot) = 22 CFS

Determine slope necessary for a 24" pipe to convey 22 CFS.

From Fig. 2, For 24" pipe K = 226

$$S_{min} = \left(\frac{22}{226}\right)^2 = \underline{\underline{0.0095'/ft}}$$

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FIGURE 1

FRICTION SLOPE

$$\text{Friction Slope} = s_f = \left(\frac{Q}{K}\right)^2$$

Q in cfs
A in Square Feet
P in Feet

$$K = \frac{1.486}{n} \cdot A \cdot R^{2/3}$$

$$R = \frac{\text{Area}}{\text{Wetted Perimeter}} = \frac{A}{P}$$

Pipe Diameter (inches)	Conveyance Factor (K)	
	n = 0.011	n = 0.013
8	14	12
10	26	22
12	42	36
15	76	65
18	124	105
21	188	158
24	267	226
27	367	310
30	485	410
33	625	529
36	789	666
39	977	825
42	1189	1006
45	1429	1209
48	1698	1436

Job No. 10924 Page No. 122

Computed by RCG Date 5/20

Checked by Date

Reviewed by Date

Approved by Date

Subject Crossroads Park

Sheet No. 1 of 2

Determine Volume of Detention Basin

Procedure - Planimeter Each Contour

- Determine volume between each contour by average end area method

- Sum all volumes for total

3004

Job No. 10924 Page No. 123

Computed by B.C.O. Date 5/20/91

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Subject Crossroads Park

Sheet No. 2 of 2

CONTOUR	TRIAL 1 ft	TRIAL 2 ft	TRIAL 3 ft	AVG. ft	ft³
47	185845.37	185690.37	185380.37	185638.70	
					237202.14
48	288455.57	288920.57	288920.57	288765.57	
					679004.61
50	390290.78	390290.57	390135.78	390239.04	
					875235.01
52	485305.97	485150.97	484530.96	484995.97	
					1069450.46
54	584506.16	584506.16	584351.16	584454.49	
					1217114.08
56	632556.26	632866.26	632556.26	632659.59	
					1307582.6
58	674716.34	674871.34	675181.35	674923.01	
					1396346.12
BASIN 60	721371.44	721681.44	721216.44	721423.11	
SACCERFIELD 60	241180.48	241490.48	241335.48	241342.15	1980292.26
62	1018662.00	1018817.00	1018662.00	1017527.00	
					2128310.87
64	1110887.20	1110577.20	1110887.20	1110783.87	
					2537613.34
66	1426932.8	1426777.8	1426777.8	1426829.47	
68	843821.68	843511.68	843511.68	843615.01	
SPLIT AREA				+	3013722.63
68	743071.48	743226.48	743536.48	743278.15	
70	883191.76	882726.76	883036.76	88[illegible]165.09	
SPLIT AREA				+	3238834.78
70	768956.53	768956.53	768956.53	768956.53	
				cu. ft.	19680708.9
					451.81 Ac-ft

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Job No. 10924 Page No. 124

Computed by B.C.O. Date 5/21/91

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Approved by Date

Subject Crossroads Park

Pipe Class Determination

Sheet No. 1 of 3

SHEET C2

PIPE	HIGH FILL	LOW FILL	(A) D LOAD HIGH FILL	(A) D LOAD LOW FILL	(B) PIPE CLASS
(11) -24"	5.1 FT	3.5 FT.	1000	1000	II
(13) -24"	5.4 FT	3.5 FT	1000	1000	II
(14) -24"	4.5 FT	0.0 FT	1000	2250	II
(10) -24"	1.0 FT.	1.0 FT.	2250	2250	II

(A) CITY OF PHOENIX D LOAD CHART

(B) ASTM C76 CHART

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Job No. 10921 Page No. 125

Computed by B.C.O Date 5/21/91

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Subject Crossroads Park

Pipe Class Determination

Sheet No. 2 of 3

SHEET C3

PIPE	HIGH FILL	LOW FILL	(A) D LOAD HIGH FILL	(A) D LOAD LOW FILL	(B) PIPE CLASS
(2) - 15"	5.7 FT.	2.5 FT.	1000	1250	II
(4) - 24"	5.9 FT.	1.8 FT.	1000	1250	II
(7) - 24"	7.1 FT	3.3 FT.	1000	1000	II
(8)-(3) - 15"	2.5 FT.	0.5 FT.	1250	2250	II

(A) CITY OF PHOENIX D LOAD CHART

(B) ASTM C76 CHART

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Job No. 109211 Page No. 126

Computed by B.C.O. Date 5/21/91

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Approved by Date

Subject Crossroads Park

Pipe Class Determination

Sheet No. 3 of 3

SHEET C6

PIPE	HIGH FILL	LOW FILL	(A) D LOAD HIGH FILL	(A) DLOAD LOW FILL	(B) PIPE CLASS
(15) - 18"	1.8	0.4 FT	1250	2250	II
(14) - 15"	5.9 FT.	2.0 FT	1000	1250	II
(13) - 15"	2.0 FT.	1.3 FT.	1250	1750	II
(5) - 18"	1.0 FT	1.0 FT	2250	2250	II

(A) CITY OF PHOENIX D LOAD CHART

(B) ASTM C76 CHART

3004

REQUIRED "D" LOAD FOR REINFORCED CONCRETE PIPE
POSITIVE PROJECTION CONDITION
(140 P.C.F. EARTH LOAD)

PIPE SIZE	H (COVER IN FEET) 1	1.25	1.5	1.75	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
12	-	2000	1700	1500				1200					1400	1700									
15		1750																				2750	
18	2250												1350										
21																							
24		1500					1000				1250		1500		1750		2000		2250		2500		
27	2000			1250																			
30	2000																						
33	1750																						
36	1750																						
39	1500	1200	1100																				
42	1400	1200	1100																				
45	1300	1100																					
48	1200										1100			1400		1700	1800	1900		2200	2300	2400	2500
51	1100										1100			1400		1700	1800	1900		2200	2300	2400	2500
54	1100																						
57																							
60												1200											
63	900																		2000				
66	850												1300		1500				2000				
69				800																	2250		
72																					2250		
75																							
78									850	950	1000			1350		1600	1750	1850		2100		2350	2450
81									850	950	1000			1350		1600	1750	1850		2100		2350	2450
84																							
87												1150											
90												1150									2200		
93																			1950				
96																							

STRUCTURAL NOTES:

1. Ku = 0.1924
2. FOR H ≤ 10, P=1.0, He/Bc = 1.7
3. FOR H > 10, P=0.5, He/Bc = 1.4
4. DEAD LOAD FACTOR = 1.9
5. SAFETY FACTOR = 1.0
6. EARTH LOADING = 140 pcf (MARSTON'S FORMULA)
7. LIVE LOADING: H-20, S-16 TRUCK, AASHO IMPACT

EXAMPLES			
CONN. PIPE SIZE	DEPTH TO TOP MIN	DEPTH TO TOP MAX	"D" LOAD
15"	2'	13'	1750
15"	2.1'	13.1	2000

NEW, NOVEMBER, 1982

TABLE 12. Supporting strength of concrete pipe per linear foot of pipe in thousands of pounds (kips)

ASTM spec. no.	Reinforced concrete culvert, storm drain, and sewer pipe C76 Safety factor = 1.0 Based on 0.01" crack																			
	Class I				Class II				Class III				Class IV				Class V			
Bedding class →	D	C	B	A	D	C	B	A	D	C	B	A	D	C	B	A	D	C	B	A
Load factor →	1.1	1.5	1.9	3.0	1.1	1.5	1.9	3.0	1.1	1.5	1.9	3.0	1.1	1.5	1.9	3.0	1.1	1.5	1.9	3.0
Internal diameter of pipe (in inches)																				
12					1.1	1.5	1.9	3.0	1.5	2.0	2.6	4.0	2.2	3.0	3.8	6.0	3.3	4.5	5.7	9.0
15					1.4	1.9	2.4	3.8	1.8	2.5	3.2	5.0	2.8	3.7	4.8	7.5	4.1	5.6	7.1	11.3
18					1.6	2.2	2.8	4.5	2.2	3.0	3.8	6.1	3.3	4.5	5.7	9.0	5.0	6.8	8.6	13.5
21					1.9	2.6	3.3	5.2	2.6	3.5	4.5	7.1	3.9	5.3	6.7	10.5	5.8	7.9	10.0	15.8
24					2.2	3.0	3.8	6.0	3.0	4.0	5.1	8.1	4.4	6.0	7.6	12.0	6.6	9.0	11.4	18.0
27					2.5	3.4	4.3	6.8	3.3	4.6	5.8	9.1	4.9	6.7	8.5	13.5	7.4	10.1	12.9	20.2
30					2.7	3.7	4.7	7.5	3.7	5.1	6.4	10.1	5.5	7.5	9.5	15.0	8.2	11.2	14.2	22.5
33					3.0	4.1	5.2	8.2	4.0	5.6	7.0	11.1	6.0	8.2	10.4	16.5	9.0	12.4	15.7	25.0
36					3.3	4.5	5.7	9.0	4.4	6.1	7.7	12.2	6.6	9.0	11.4	18.0	9.9	13.5	17.1	27.0
42					3.8	5.2	6.6	10.5	5.2	7.1	8.9	14.2	7.7	10.5	13.3	21.0	11.5	15.7	20.0	31.5
48					4.4	6.0	7.5	12.0	6.0	8.1	10.2	16.2	8.8	12.0	15.2	24.0	13.2	18.0	22.8	36.0
54					4.9	6.7	8.5	13.5	6.7	9.1	11.5	18.2	9.9	13.5	17.1	27.0	14.8	20.2	25.7	40.5
60	4.4	6.0	7.6	12.0	5.5	7.5	9.5	15.0	7.4	10.1	12.8	20.2	11.0	15.0	19.0	30.0	16.5	22.5	28.5	45.0
66	4.8	6.6	8.3	13.2	6.0	8.2	10.4	16.5	8.1	11.1	14.1	22.3	12.1	16.5	21.0	33.0	18.1	24.8	31.3	49.5
72	5.3	7.2	9.1	14.4	6.6	9.0	11.4	18.0	8.9	12.1	15.4	24.3	13.2	18.0	22.8	36.0	19.8	27.0	34.2	54.0
78	5.7	7.8	9.9	15.6	7.1	9.7	12.3	19.5	9.6	13.2	16.7	26.3	14.3	19.5	24.7	39.0				
84	6.1	8.4	10.6	16.8	7.7	10.5	13.3	21.0	10.4	14.2	18.0	28.4	15.4	21.0	26.6	42.0				
90	6.6	9.0	11.4	18.0	8.2	11.2	14.2	22.5	11.1	15.2	19.2	30.4								
96	7.0	9.6	12.2	19.2	8.3	12.0	15.2	24.0	11.9	16.2	20.5	32.4								
102	7.5	10.2	12.9	20.4	9.3	12.7	16.1	25.5	12.6	17.2	21.8	34.4								
108	7.9	10.8	13.7	21.6	9.9	13.5	17.1	27.0	13.4	18.2	23.1	37.0								

*Supporting strengths shown in table are for concrete pipe meeting ASTM specifications (3-edge bearing test) and include safety and bedding factors as indicated.

TABLE 13. Values of *N* for various classes of bedding

Method of bedding	Value of *N*
Class A, Concrete cradle	.505
Class B, First class	.707
Class C, Ordinary	.840
Class D, Impermissible	1.310

TABLE 14. Values of *x* for various projection ratios *p*

Projection ratio *p*	Values of *x*	Values of *x'*
0.0	0	0.150
0.3	0.217	0.743
0.5	0.423	0.856
0.7	0.594	0.811
0.9	0.655	0.678
1.0	0.638	0.638

It should be emphasized that this is a construction method for high fills. In order to prevent settlement of the surface of the embankment over the pipe, the height of the embankment over the pipe must be greater than the height of the theoretical "plane of equal settlement." Generally, it will not be economical to go to this type of construction unless the fill heights exceed those that can be supported with standard classes of pipe and usual methods of installation. This automatically reserves the imperfect trench method for high fills.

This method also can be used to reduce the backfill