AEC #81059

Drainage Study

for

Apache Junction Junior High School

August 14, 1981

(Revised September 2, 1981)

(Revised September 25, 1981)

Library. It ties into the Powerline FRS Northern Diversion.

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Background Information

The proposed junior high school is located at the southwest corner of Southern Avenue and San Marcos Avenue. The unimproved site generally slopes from northeast to southwest at about 0.01 ft/ft, and is covered with medium-density desert brush. Several small washes cut across the site, but natural drainage is, for the most part, sheet flow. Project improvements will include paving, with curb and gutter, for both Southern and San Marcos, which will eliminate any concentrated flows entering the site via the small washes mentioned above.

Offsite Hydrology

Exhibit A shows what is estimated to be the maximum watershed contributing storm runoff to the site. Peak runoff values estimated by using this watershed area are probably quite conservative since Highway 60 actually prevents most runoff north of the highway from affecting the site. Exhibit B shows peak runoff values for that part of the total watershed which contributes flow to the dike crossing, which is discussed on page 2. Exhibit C shows peak runoff values entering the entire site along the northern and eastern boundaries.

The offsite runoff will be directed around the school buildings via Southern Avenue and a 2' deep by 90' wide drainage swale on the north, and via San Marcos and the parking lot on the east. Once past the buildings, the runoff will continue, as before, towards the Maricopa County flood control structure west of the school site. The streets, channels, and parking lot carry more than enough to protect the buildings from the 100-year storm.

Onsite Drainage Pattern and Retention

Positive drainage is provided around all buildings, and stormwater is then directed away from the building areas into the detention area via a system of drain pipes and drainage swales (see the grading and drainage plan). The required amount of onsite retention was calculated as follows:

Volume Req'd = 7200 ciA

= 95,040 cf

Where c = 0.30

i(50-yr, 24-hr) = 2.20 in/hr

A = 20 ac.

The onsite retention is provided in the play area south of the school buildings, and amounts to 102,500 cf. This area will be slowly drained at a rate of about 1.5 cfs, via an 8" pipe in the southwest corner of the basin. This will direct the detained water, at a slow flowrate, towards the county flood control channel in the direction of natural flow.

Special Item - Dike Crossing

A temporary walkway across the Maricopa County flood control structure west of the site, has been designed to provide access to the high school cafeteria. This is being coordinated with the County Flood District, and poses no flooding problem for the proposed school buildings.

The following is a summary of the hydraulic data and calculations for the proposed crossing of the drainage channel and diversion dike. The calculations follow the procedure as outlined in NEH, Section 4, Chap. 14 by S.C.S.

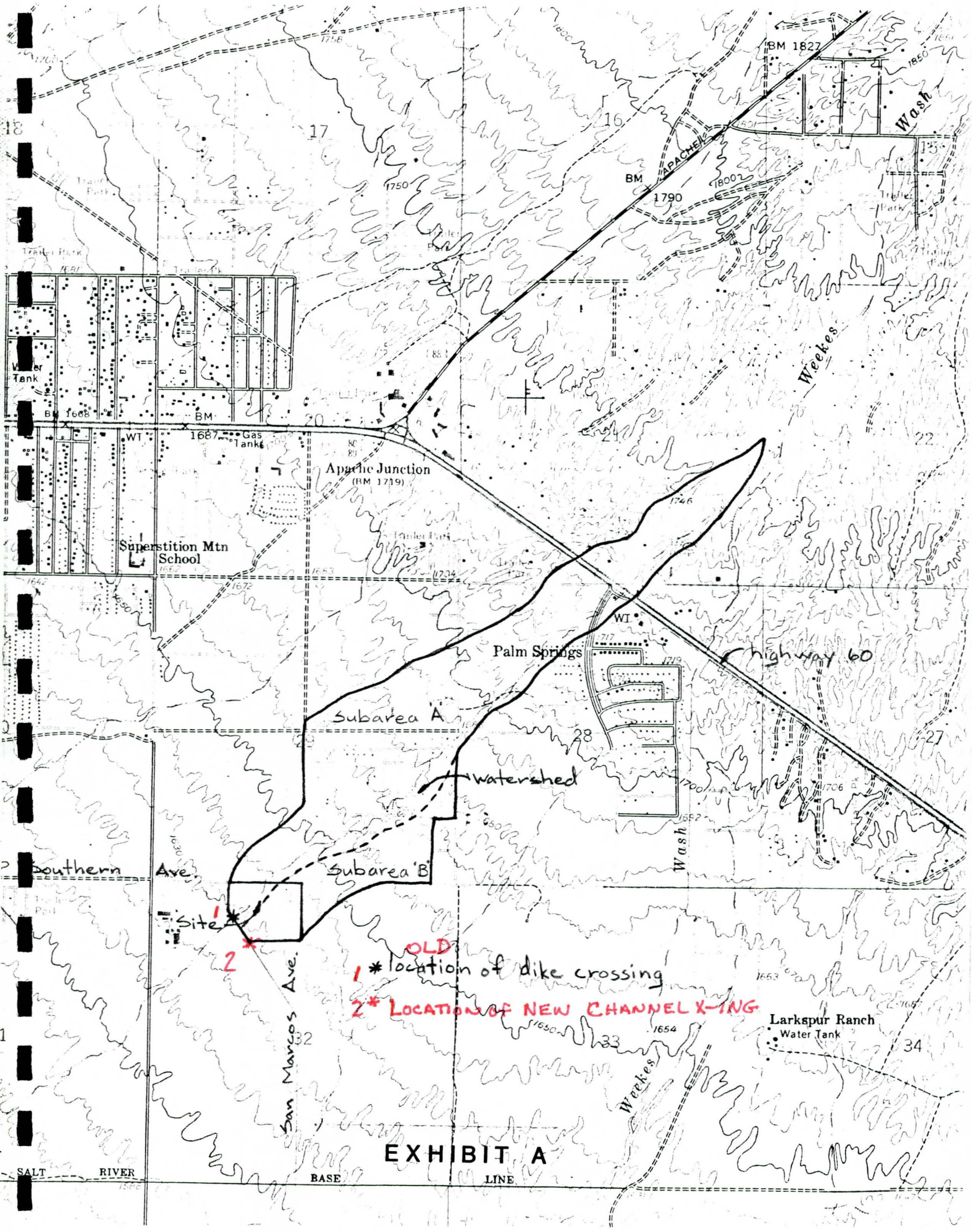
Figure 2 shows details of the dike crossing as contained in the construction drawings. Figure 6 gives a more detailed cross section of the crossing, which was obtained by running field levels.

Figure 3 gives the headwater computations for various discharges through the 18" C.M.P. These are plotted in Figure 5 as curves #4 and #5. Outlet control (curve #5) is seen to govern over the entire range plotted.

Discharges over the walkway are tabulated at various elevations in Figure 4. These are plotted in Figure 5 as curve #3.

Curve #2 in Figure 5 represents the combined discharge through the pipes and over the walkway. It was obtained by combining curves #3 and #5, at the point where water just begins to flow over the walkway.

As seen from curve #2 in Figure 5, the in-bank capacity at the crossing is 220 cfs. This exceeds both the 50-year flood (162 cfs) and the 100-year flood (201 cfs), as computed in Exhibit B.


BM 1827
Wash
APACHE
BM
1790
Trailer Park
Water Tank
BM 1668
BM
1687
WT
Gas Tanks
Apache Junction
(BM 1719)
Superstition Mtn
School
Palm Springs
WT
highway 60
Weekes
Subarea A
watershed
Subarea 'B'
Southern Ave.
Site
San Marcos Ave.
OLD
1 * location of dike crossing
2* LOCATION OF NEW CHANNEL X-ING
Larkspur Ranch
Water Tank
Wash
Weekes
SALT RIVER BASE LINE
EXHIBIT A

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET

S C S METHOD: PART I

LOCATION DATA:

~~Highway~~ North Diversion Dike County Pinal
Location Apache Junction Jr High School
Project No. ______ Station ______
Name of Stream Subarea 'A'

DESIGN DATA:

Design Frequency	10, 50, 100	years
Drainage Area	0.530	square miles
Drainage Length	12,400	feet
Elevation		
Top of Drainage Area	1763	feet
At Structure	1633	feet
Drainage Area Slope	1.048	%
Drainage Width	1,500	feet
Width factor W_f	1.0	
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	15	%
Soil Group	B	(from "General Soil Map" SCS)
Precipitation		
P = 6 hour =	2.02, 2.80, 3.16	inches
P = 24 hour =	2.42, 3.40, 3.86	inches

DESIGN COMPUTATION:

Precipitation	P = 1 hour =	1.57, 2.18, 2.45	inches
Curve Number		83	
Runoff	Q =	0.40, 0.82, 1.02	inches
Time of Concentration	Tc	1.30	hours
Time of Peak	Tp = (Tc)(W_f)	1.30	hours
Peak Discharge	$Qp = \frac{484\,AQ}{Tp}$ =	197.32 Q	
	=	79, 162, 201	cfs

Computed by M. Lloyd Date 9/25/81

EXHIBIT B

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION

HYDROLOGIC DESIGN DATA SHEET

S C S METHOD: PART I

LOCATION DATA:

Highway North Diversion Dike County Pinal
Location Apache Junction Jr High School
Project No. ______ Station ______
Name of Stream Total Watershed (Subareas 'A' and 'B')

DESIGN DATA:

Design Frequency	10, 50, 100	years
Drainage Area	0.631	square miles
Drainage Length	11,000	feet
Elevation		
Top of Drainage Area	1,763	feet
At Structure	1,643	feet
Drainage Area Slope	1.091	%
Drainage Width	1,500	feet
Width factor W_f	1.0	
Vegetative Cover Type	Desert Brush	
Vegetative Cover Density	15	%
Soil Group	B	(from "General Soil Map" SCS)
Precipitation		
P = 6 hour =	2.02, 2.80, 3.16	inches
P = 24 hour =	2.42, 3.40, 3.86	inches

DESIGN COMPUTATION:

Precipitation	P = 1 hour =	1.57, 2.18, 2.45	inches
Curve Number		83	
Runoff	Q =	0.40, 0.82, 1.02	inches
Time of Concentration	Tc	1.28	hours
Time of Peak	$Tp = (Tc)(W_f)$	1.28	hours

Peak Discharge $Qp = \frac{484\ AQ}{Tp} =$ 238.60 Q

= 95, 196, 243 cfs

Computed by M. Lloyd Date 9/25/81

EXHIBIT C

Project No. Apache Junction Jr. High School Station ______

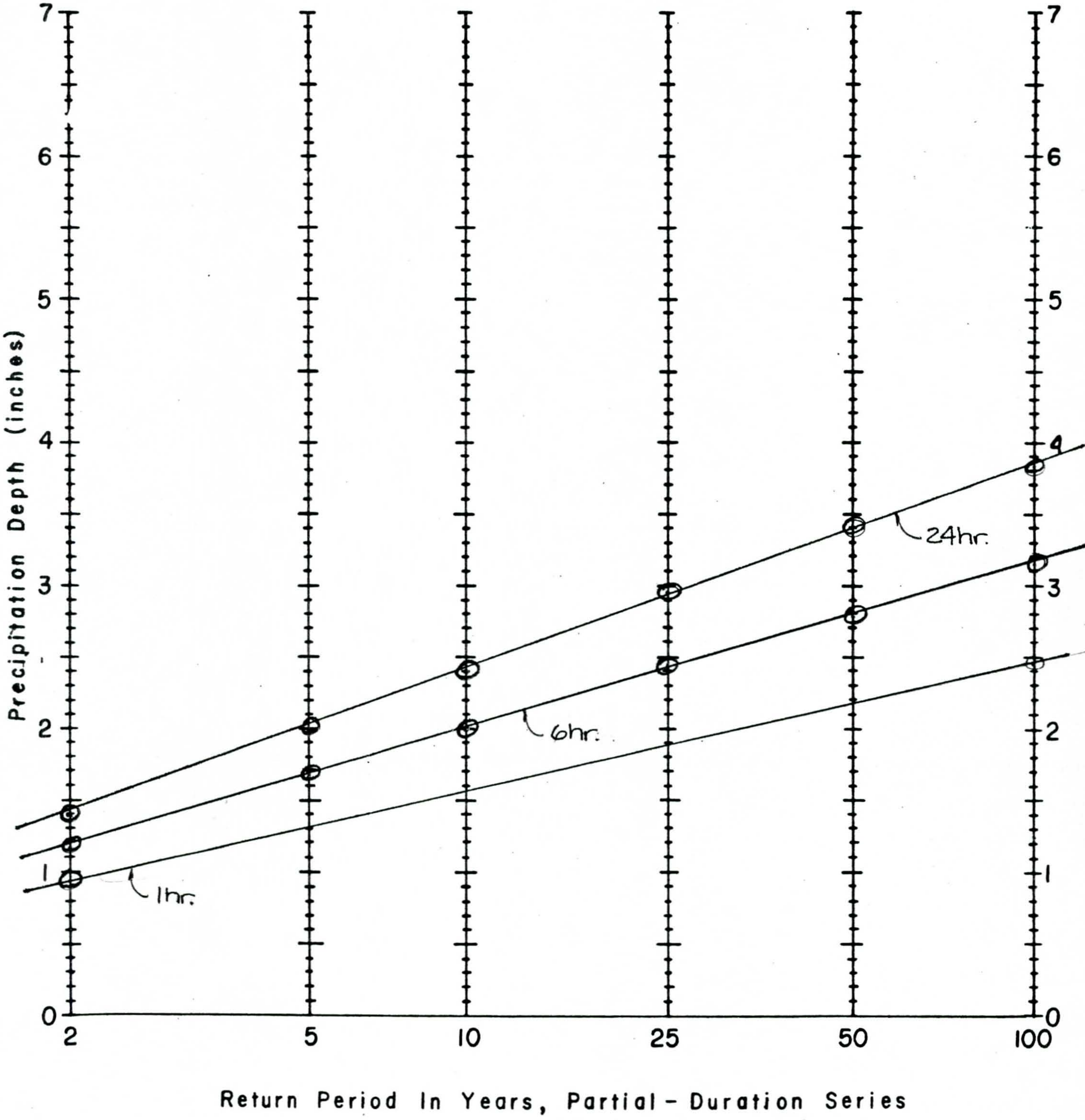


Figure I Precipitation Depth Versus Return Period for Partial - Duration Series

Apache Junction
Jr. High School

ADDENDUM to "HYDROLOGIC DESIGN FOR HIGHWAY DRAINAGE IN ARIZONA" April 1975

Steps to be used to determine precipitation values for various durations and return periods.

STEP 1. From the precipitation maps in the manual "Hydrologic Design for Highway Drainage in Arizona", determine the precipitation values for the 6 and 24 hour duration storms for return periods of 2, 5, 10, 25, 50 and 100 years. Tabulate these values in Table 1 in the column headed 'Map Values'

TABLE 1

Return Period (Years)	Precipitation Values (inches)			
	6 hour duration		24 hour duration	
	Map Value	Corrected Value	Map Value	Corrected Value
2	1.17	1.20	1.40	1.43
5	1.67	1.69	2.00	2.03
10	2.00	2.02	2.40	2.42
25	2.43	2.42	2.95	2.93
50	2.78	2.80	3.40	3.40
100	3.13	3.16	3.82	3.86

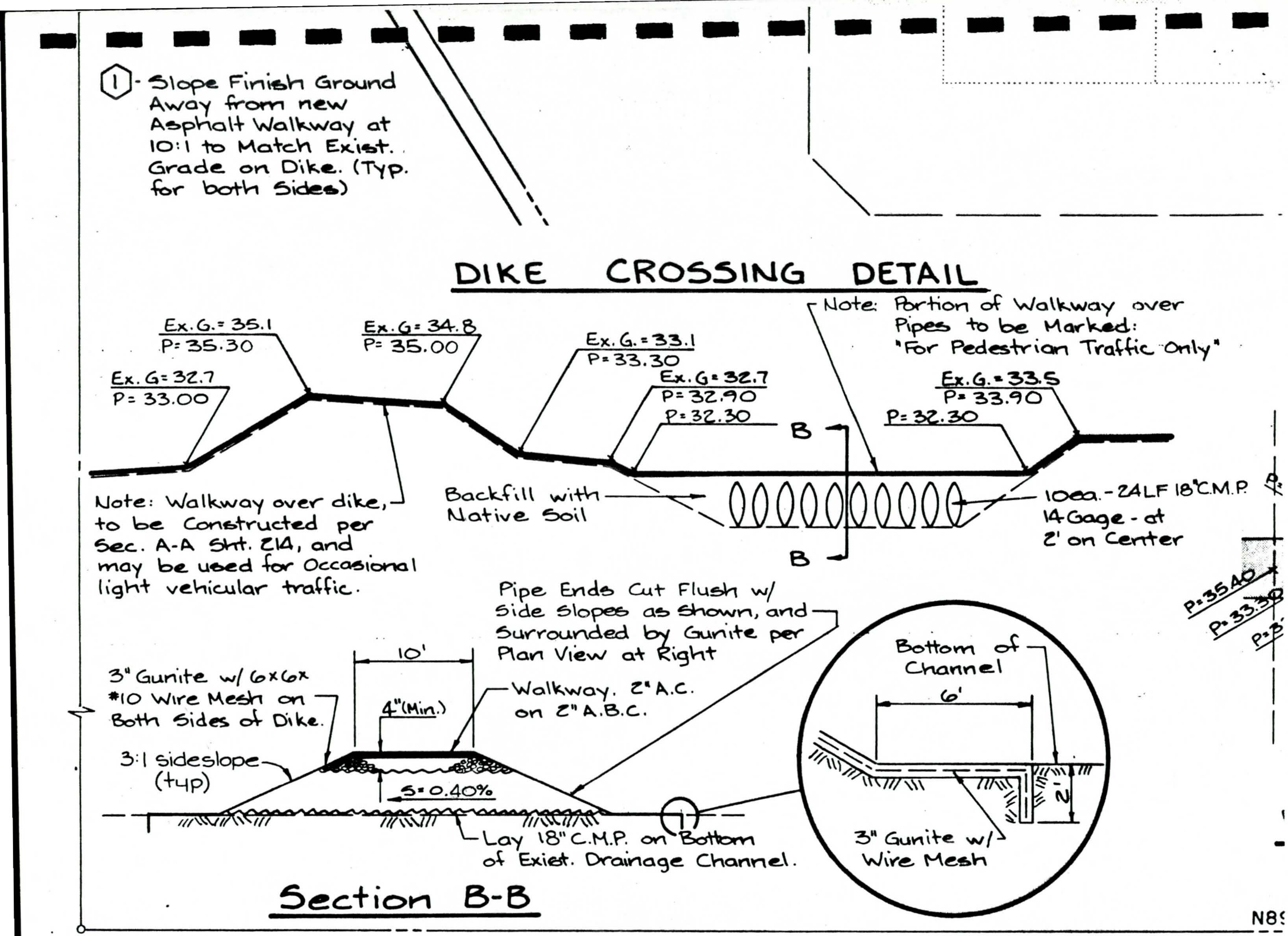
NOTE: There is a possibility of making an error while reading the maps because, (1) a site is not easy to locate precisely on a series of 12 maps, (2) there may be some slight registration differences in printing, and (3) precise interpolation between isolines is difficult. In order to minimize any errors in reading the maps, these values should be plotted on the diagram "Precipitation Depth versus Return Period" Fig. 1.

1 hr.:

$Y_2 = -0.011 + 0.942\,(1.20^2/1.43) = 0.938$ in.

$Y_{100}\ \ 0.494 + 0.755\,(3.16^2/3.86) = 2.447$ in.

50 yr - 2 hr: $Y_{50} = 0.341\,(2.80) + 0.659\,(2.18) = 2.391$ in.



1 - Slope Finish Ground Away from new Asphalt Walkway at 10:1 to Match Exist. Grade on Dike. (Typ. for both Sides)
DIKE CROSSING DETAIL
Ex. G. = 35.1
P: 35.30
Ex. G = 34.8
P: 35.00
Ex. G. = 33.1
P = 33.30
Ex. G = 32.7
P = 33.00
Ex. G = 32.7
P = 32.90
P = 32.30
Ex. G. = 33.5
P = 33.90
P = 32.30
Note: Portion of Walkway over Pipes to be Marked: "For Pedestrian Traffic Only"
B
B
Note: Walkway over dike, to be Constructed per Sec. A-A Sht. 214, and may be used for Occasional light vehicular traffic.
Backfill with Native Soil
10ea. - 24LF 18" C.M.P. 14 Gage - at 2' on Center
Pipe Ends Cut Flush w/ Side Slopes as Shown, and Surrounded by Gunite per Plan View at Right
10'
3" Gunite w/ 6x6x #10 Wire Mesh on Both Sides of Dike.
4" (Min.)
Walkway. 2" A.C. on 2" A.B.C.
3:1 sideslope (typ)
S = 0.40%
Lay 18" C.M.P. on Bottom of Exist. Drainage Channel.
Bottom of Channel
6'
2'
3" Gunite w/ Wire Mesh
Section B-B
P=35.40
P=33.30
N8

FIGURE 2

FIGURE 3

Headwater Computations for 10 - 18" C.M.P.

total Q cfs	Q/pipe cfs	V (full) fps	Inlet Control			Outlet Control							
			$\frac{HW}{D}$	HW	HW_I	Ke	H	d_c	$\frac{d_c+D}{2}$	h_o elev.	TW elev.	LS_o	HW_o elev.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
25	2.5	1.415	0.57	0.86	31.36	0.7	0.099	0.60	1.05	31.45	31.3	0.10	31.45
50	5.0	2.830	0.85	1.28	31.78	0.7	0.395	0.85	1.175	31.575	31.7	0.10	32.00
100	10.0	5.659	1.70	2.55	33.05	0.7	1.580	1.25	1.375	31.775	32.2	0.10	33.68
150	15.0	8.489	3.00	4.50	35.00	0.7	3.556	1.45	1.475	31.875	32.6	0.10	36.06
200	20.0	11.319	4.50	6.75	37.25	0.7	6.322	1.50	1.50	31.90	32.9	0.10	39.12
225	22.5	12.733	5.20	7.80	38.30	0.70	8.000	1.50	1.50	31.90	33.1	0.10	41.00

Notes:
1) pipes parallel with channel flow
2) pipes have mitered edges
3) HW_I = HW + invert elev. @ entrance
4) $h_o = \frac{d_c + D}{2}$ + invert elev. @ exit.
5) $HW_o = H + TW - LS_o$ or $H + h_o - LS_o$, whichever is greater
6) $H = \left(1 + K_e + \frac{29 n^2 L}{R^{1.33}}\right)\frac{V^2}{2g}$, where n = 0.024 & L = 24'
7) procedure from NEH, SEC 4, chap. 14, 1972

FIGURE 4

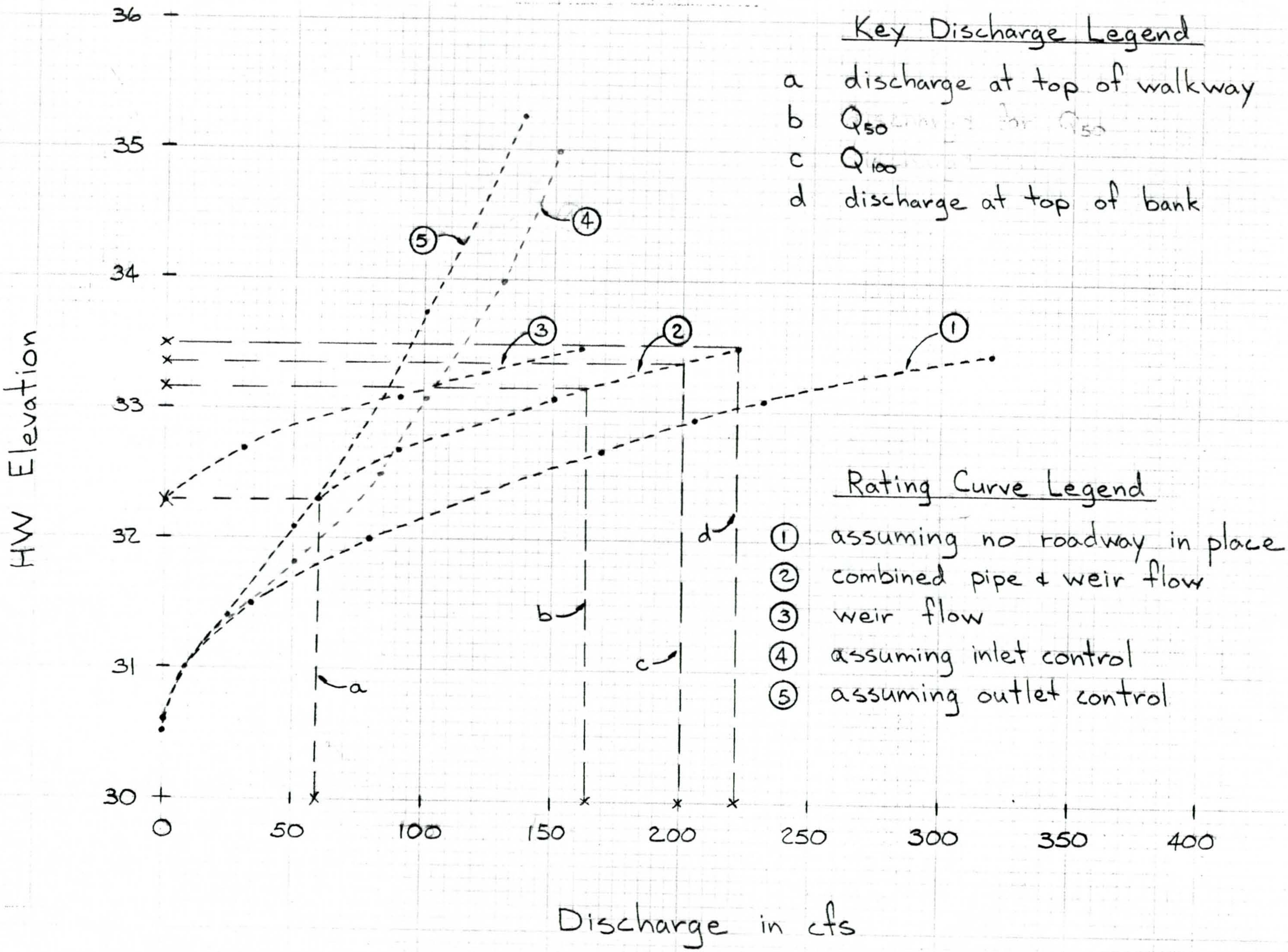
Stage Discharge Over Walkway

Elev.	h over road ft.	$h^{1/2}$	A over road ft^2	A up-stream ft^2	Q thru pipes cfs	Q est. over bridge cfs	Q est. total cfs	V fps	$\frac{V^2}{2g}$	c'	Q over road cfs
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
32.3	0	0	0	43.9	55.0	0	55	1.25	0.024	0	0
32.7	0.4	0.632	14.5	58.4	70.0	30	100	1.71	0.046	3.17	30
33.1	0.8	0.894	31.6	76.1	85.0	90	175	2.30	0.082	3.13	90
33.5	1.2	1.095	47.5	95.0	95.0	160	255	2.68	0.112	3.09	160

Notes:

1) velocity head = $\frac{V^2}{2g}$
2) $c' = c \left[\frac{1}{\left(\frac{h}{h + \frac{V^2}{2g}} \right)^{3/2}} \right]$, where $c = 2.7$
3) $Q = c' A h^{1/2}$
4) procedure from NEH, sec. 4, chap. 14, 1972.

FIGURE 5


Key Discharge Legend
a discharge at top of walkway
b Q_{50}
c Q_{100}
d discharge at top of bank
Rating Curve Legend
① assuming no roadway in place
② combined pipe & weir flow
③ weir flow
④ assuming inlet control
⑤ assuming outlet control
HW Elevation
36
35
34
33
32
31
30
0
50
100
150
200
250
300
350
400
Discharge in cfs
a
b
c
d

CHANNEL CROSS SECTION

CRITICAL WATER LEVEL SUMMARY

top of bank elevation = 33.5
high water level for Q_{50} (HWL_{50}) = 33.2
high water level for Q_{100} (HWL_{100}) = 33.4
top of walkway elevation = 32.3

LEGEND

(35.0) ----- finish pavement
34.8 - - - - - natural ground

SCALE

vert.: 1" = 4'
horz.: 1" = 10'

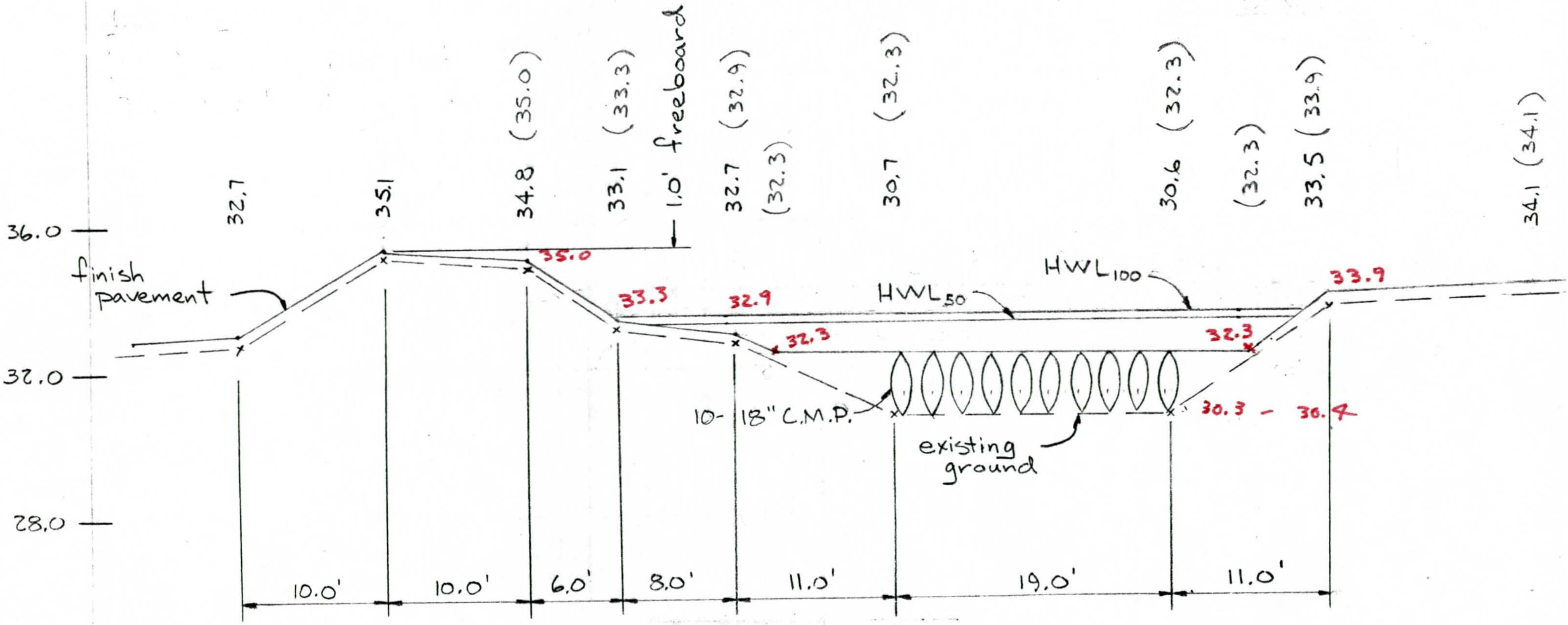


FIGURE 6