COUNTY OF ORANGE

PUBLIC FACILITIES AND RESOURCES DEPARTMENT

SANTA ANA, CALIFORNIA

ORANGE COUNTY FLOOD CONTROL DISTRICT
DESIGN MANUAL

November 2000

Vicki L. Wilson
Director

Orange County Board of Supervisors

Charles V. Smith
First District

Todd Spitzer
Third District

James W. Silva
Second District

Cynthia P. Coad
Fourth District

Thomas W. Wilson
Fifth District

COUNTY OF ORANGE

PUBLIC FACILITIES AND RESOURCES DEPARTMENT

SANTA ANA, CALIFORNIA

ORANGE COUNTY FLOOD CONTROL DISTRICT
DESIGN MANUAL

November 2000

Prepared By: Mouhsen M. Habib, Project Engineer
PFRD/Flood Control Design

Submitted By: Hany S. Henein, Senior Civil Engineer
PFRD/Flood Control Design

Recommended By: _____
J. A. Miller, Chief, PFRD/Flood Control Design

Approved By: _____
V. P. Rosales, Manager, PFRD/Design Division

Concurred By: _____
Kenneth R. Smith
PFRD, Deputy Director/Chief Engineer

TABLE OF CONTENTS

ADDENDA

TABLE OF CONTENTS

CHAPTER I

CHANNEL HYDRAULICS

TABLE OF CONTENTS

CHAPTER II

STRUCTURES

Table of Contents
Chapter II - continued

STORM WATER PROTECTION GOALS

A. STRUCTURES

The goal is to provide 100-year protection for residences and other nonfloodproof structures pursuant to Public Services and Facilities Element of the General Plan. - (See attached Flood Protection Goals.) Regional flood control facilities shall be planned and designed to meet this goal where feasible.

B. STREETS

Street criteria for 100-year storm flow is shown on the attached Flood Protection Goals.

1. Arterial Highway

    -   One 12-foot traffic lane will be free from inundation in each direction in a 10-year storm.

    -   In a sump condition, one 12-foot traffic lane will be free from inundation in each direction in a 25-year storm.

2. Local Street

    -   Storm waters will not exceed top of curb in a 10-year storm.

    -   In a sump condition, storm waters will not exceed top of curb in a 25-year storm.

3. Arterial Highway and Local Street

    -   Depth times velocity cannot exceed six. This criteria applies to storms up to 25-year.
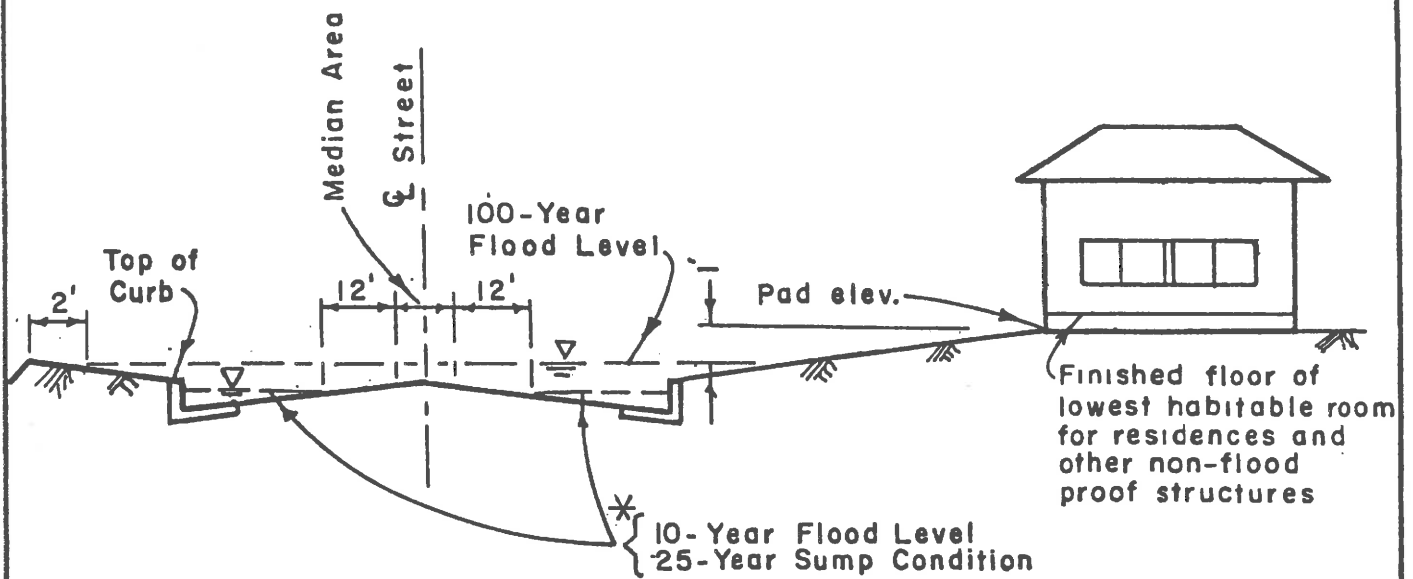
ARTERIAL HIGHWAY
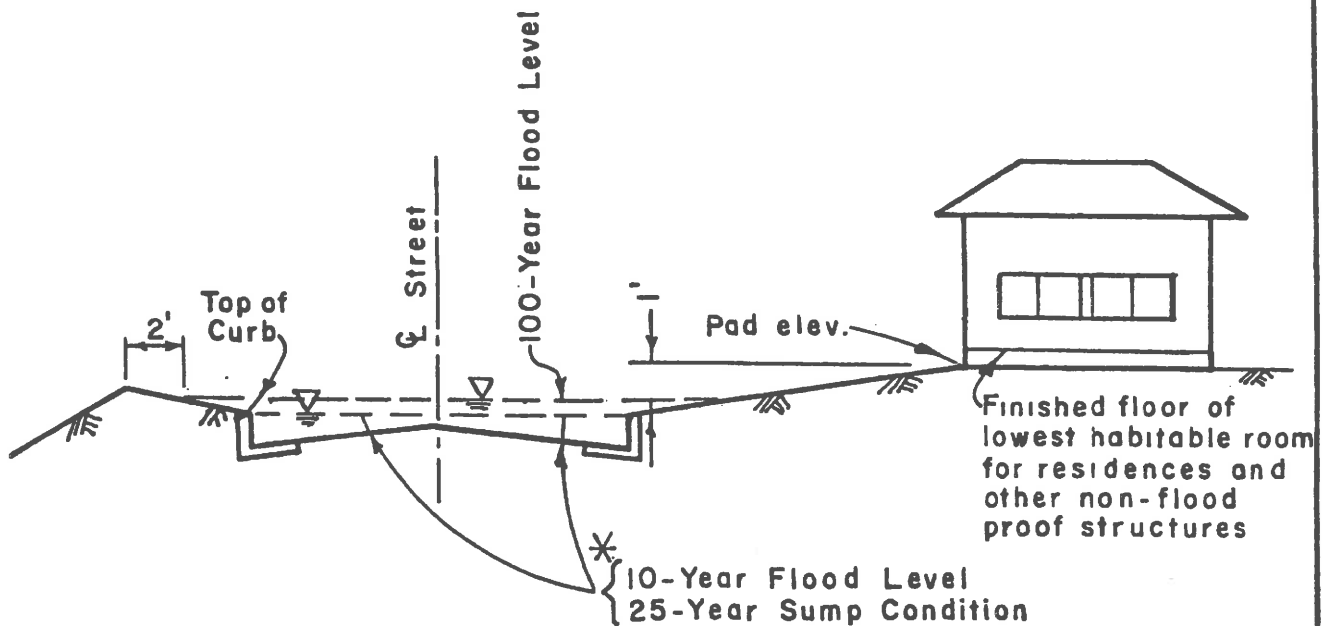
LOCAL STREET

NOTE

* For Arterial Hwy and Local Street, depth times velocity cannot exceed six

ORANGE COUNTY E.M.A.

FLOOD PROTECTION GOALS

| 1986 | 1 OF 1 | |

CHAPTER I

CHANNEL HYDRAULICS

A. GENERAL

The district's criteria for design normally require calculation of hydraulic gradients over the entire project length for at least design flow and quite often for percentages above and below design flow. Hydraulic design should further include consideration of and accomodation for every inlet, curve, section and slope change, etc.

While deviation from the criteria will not be accepted as an expediency that compromises project quality the designer may deviate from the criteria listed provided all deviations are identified and justified.

B. SYMBOLS AND NOTATIONS

The symbols used in this manual were selected (with slight modification) from King[1] and Chow[2], the two references most used by the design engineers of the district.

A - Cross sectional area of water or structure (ft.^{2})

b - Width of invert (ft.)

C - Various coefficients

d - Diameter of circular conduits (ft.)

D - Depth of water (ft.)

D_c- Critical depth (ft.)

D_m- Mean Depth (ft.)

F_r- Froude   Number

g - Acceleration of gravity (ft./sec.^{2})

h - Water elev. change in junction (ft.)

h_L- Head loss (ft.)

Channel Hydraulics

h_v- Velocity head (ft.)

H - Height of box conduit (ft.)

K - Coefficient for head loss formulas

L - Length (ft.)

n - Coefficient of roughness (Manning's)

M - Momentum (ft.^{3})

P - Pressure (ft.^{3})

Q - Quantity of flow (ft.^{3}/sec)

R - Hydraulic radius (ft.) and
    Radius of curvature (ft.)

S - Slope of facility (ft./ft.)

S_f- Slope of energy gradient (ft./ft.)

T - Width of water surface (ft.)

V - Average velocity of flow (ft./sec.)

x - Length (ft.)

W - Unit weight of water (lbs./ft.^{3})

$\bar{y}$ - Vertical distance down to center of gravity (ft.)

z - Cotangent of sideslope (trap. sections)

Z - Difference in elevation (ft.)

C. FLOW STABILITY

At depths near critical, large depth changes occur with slight changes in total energy resulting in the creation of large waves and other disturbances without apparent cause.[2] This is the range of depths where the Froude number (F_r) is near one.

Except for special cases where it is more economical to provide sufficient height to confine the waves than modify slopes, stable depths shall be provided. Stability for district projects is defined as: the range of depths

where the Froude number is below 0.9 for subcritical velocities and above 1.2 for supercritical velocities.

$$0.9 > F_r > 1.2 \qquad \text{where} \qquad F_r = \frac{V}{\sqrt{gD_m}}$$

D. LOSSES

(1) Friction Losses

While many methods are available for computing friction head losses, Manning's equation is by virtue of its simplicity and ease of application the most popular equation currently in use. Therefore, even though it is recognized that other perhaps more accurate formulations may be preferred by some engineers, the district has adopted Manning's equation for determining friction losses.[3]

Friction slopes for steady flow shall be determined by Manning's equation:

$$S_f = \left[\frac{Qn}{1.486 \, AR^{\frac{2}{3}}} \right]^2$$

Although some disagreement exists it is generally recognized that Manning's "n" is not independent of hydraulic radius and that increased hydraulic radius is accompanied by increased "n" value.

While most district projects fall within a relatively narrow range of hydraulic radii and adjustment of "n" is not usually required, for large improved channels the values of "n" tabled below may be inadequate. In channels where the hydraulic radius is 5 or greater the tabled value should be increased[4]

Channel Hydraulics

approximately 15 percent and rounded to the nearest thousandth.

SEE ADDENDUM #1

MANNING'S EQUATION
Coefficient of Roughness

DESCRIPTION	MANNING'S "n"
Concrete Sections	
Rectangular, flowing full[1] and precast pipe	.013
Rectangular, open flow[1]	.014
Trapezoidal and Cast-in-place pipe	.015
Asphalt Concrete Sections	.017
Asbestos Cement Pipe	.011
Corrugated Steel Pipe[2]	.024
Engineered Earth Channels	
Fine sand and silt	
size determination	.030
scour determination	.020
River sand and gravel[5]	.025
Coarse gravel mixed with boulders	.035
Greenbelt Channels	
Maintained turf[1]	.030
Heavily weeded, no brush	.040
Heavily weeded, moderate shrubs[1]	.050
Some weeds, heavy brush	.060
Rock Slope Protection	
Levee riprap	.035
Flush Grouted Riprap	.020
Sacked Concrete	.025

Channel Hydraulics

DESCRIPTION	MANNING'S "n"
Wire Revetment - - - - - - - - - - - - - - - -	.035
Natural Streams[6]	
Regular section- - - - - - - - - - - - - -	.045
Irregular section- - - - - - - - - - - -	.060
Mountain - - - - - - - - - - - - - - - - -	.055
Flood Plains[6]	
Pasture or cultivated- - - - - - - - -	.040
Heavy weeds, light brush - - - - - - -	.050
Medium to dense brush- - - - - - - - -	.090
Willows- - - - - - - - - - - - - - - - - -	.170

In cases where the wetted perimeter is composed of sections of varying roughness an equivalent value for the entire perimeter may be obtained by the following equation:[2]

$$n = \left(\frac{P_1 n_1^{\frac{3}{2}} + P_2 n_2^{\frac{3}{2}} + P_N n_N^{\frac{3}{2}}}{P} \right)^{\frac{2}{3}}$$

P and n with subscripts are the wetted perimeter and coefficient of roughness, respectively, for the various sections of roughness within the entire perimeter, P.

(2) Confluences

Junctions should be analyzed by the specific force

(pressure plus momentum, P+M) method if the incremental increase in flow is more than 10 percent of the flow in the main channel or if the incremental increase, regardless of magnitude, could adversely affect the system. Structures flowing at slightly supercritical velocities are especially susceptible to adverse affects from side inflows.

The P+M method used for district projects (based on Newton's second law of motion) has been expanded from the Corps of Engineers open channel analysis[7] to include all junctions.

The general equilibrium equation is:

$$P_2 + M_2 = P_1 + M_1 + M_3 \cos \Theta + P_{li} + P_w - P_f$$

Where P_1 = hydrostatic pressure on section 1

P_2 = hydrostatic pressure on section 2

P_i = horizontal component of hydrostatic pressure on invert

P_s = horizontal component of hydrostatic pressure on soffit

P_w = axial component of hydrostatic pressure on walls

P_f = retardation force of friction

M_1 = momentum of moving mass of water entering junction at section 1

M_2 = momentum of moving mass of water leaving junction at section 2

$M_3 \cos \Theta$ = axial component of momentum of the moving mass of water entering the junction at section 3

Channel Hydraulics

The expression for pressure acting on an area is

$$P = wA\overline{y} \qquad \text{(pounds)}$$

and for momentum per unit time is

$$M = wQV/g \qquad \text{(pounds)}$$

However, since the unit weight of water (w) appears in all terms of the general equilibrium equation it may be omitted and the dimension for P + M becomes feet to the third power.

Since most applications of junction analysis involve relatively small elevation changes simplifying assumption have been made that cosines of the invert slope equal unity and tangents and sines of the friction slope are equal.

The designer should recognize that components of wall and invert pressures may be either positive or negative and should be used accordingly.

Often when a confluence is within a transition from trapezoidal to rectangular shape (or reverse), a portion of the invert and wall pressures are of negative sign. These can be measured by superimposing the end areas of the sections over each other and developing a graphical representation of the negative areas. By adding algebraically the component $A\overline{y}$'s, a reasonable approximation of the wall and invert pressures is obtained.

The following are examples of those cases most often encountered:

Channel Hydraulics

CONFLUENCES

OPEN TRAPEZOIDAL CHANNEL

$$b_2 \geqq b_1$$

$$P_1 = \frac{D_1^2}{6}(3b_1 + 2z_1 D_1) \qquad\qquad P_2 = \frac{D_2^2}{6}(3b_2 + 2z_2 D_2)$$

$$M_1 = \frac{Q_1^2}{(b_1 + z_1 D_1)gD_1} \qquad\qquad M_2 = \frac{Q_2^2}{(b_2 + z_2 D_2)gD_2}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3 g}\cos\theta \qquad \text{Where } A_3 = \text{water area at section 3}$$

$$P_i = \left(\frac{b_1 + b_2}{2}\right) z \left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)} \right]$$

$$P_w = \frac{D_1 + D_2}{4} \left[\frac{b_1 + b_2}{2}(D_1 - D_2) + h(z_1 D_1 + z_2 D_2) + (b_2 + z_2 D_2)D_2 - (b_1 + z_1 D_1)D_1 \right]$$

$$P_f = \frac{L(s_1 + s_2)}{4} \left[(b_1 + z_1 D_1)D_1 + (b_2 + z_2 D_2)D_2 \right] \quad \text{where } s = S_f$$

CONFLUENCES

OPEN RECTANGULAR CHANNEL

$b_2 \geq b_1$
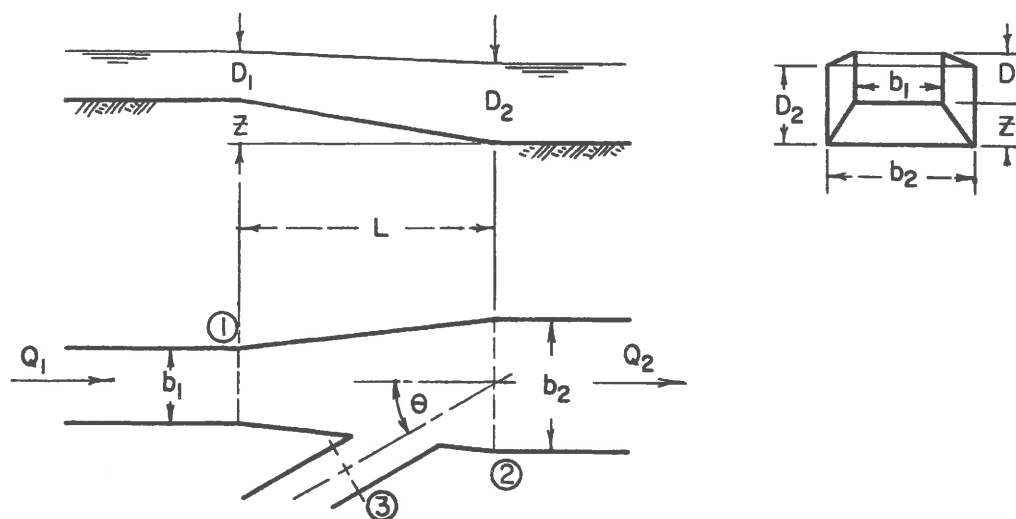
$$P_1 = \frac{b_1 D_1^2}{2} \qquad\qquad P_2 = \frac{b_2 D_2^2}{2}$$

$$M_1 = \frac{Q_1^2}{b_1 D_1 g} \qquad\qquad M_2 = \frac{Q_2^2}{b_2 D_2 g}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{A_3 g}(\cos\theta) \qquad \text{Where } A_3 = \text{water area at section 3}$$

$$P_i = \left(\frac{b_1 + b_2}{2}\right) Z \left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)} \right]$$

$$P_w = \frac{D_1 + D_2}{4}(b_2 - b_1) \left[D_1 + \frac{(D_2 - D_1)(D_1 + 2D_2)}{3(D_1 + D_2)} \right]$$

$$P_f = \frac{L(s_1 + s_2)}{4}(b_1 D_1 + b_2 D_2)$$

CONFLUENCES

RECTANGULAR BOX UNDER PRESSURE

$$b_2 \geq b_1$$
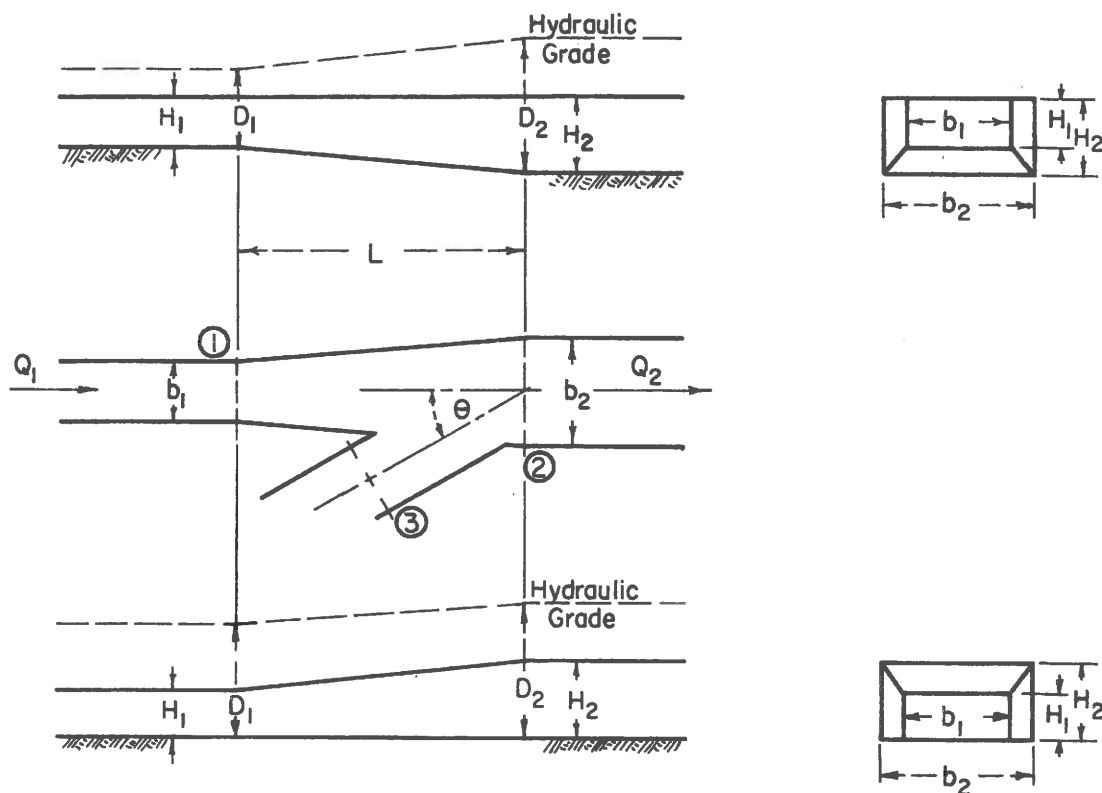
$$P_1 = b_1 H_1 \left(D_1 - \frac{H_1}{2} \right) \qquad\qquad P_2 = b_2 H_2 \left(D_2 - \frac{H_2}{2} \right)$$

$$M_1 = \frac{Q_1^2}{b_1 H_1 g} \qquad\qquad M_2 = \frac{Q_2^2}{b_2 H_2 g}$$

$$M_3 \cos \Theta = \frac{(Q_2 - Q_1)^2}{A_3 g} (\cos \Theta) \qquad \text{Where } A_3 = \text{water area at section 3}$$

$$P_i = \frac{b_1 + b_2}{2} (H_2 - H_1) \left[D_1 + \frac{(D_2 - D_1)(b_1 + 2b_2)}{3(b_1 + b_2)} \right] \left[\text{Level Soffit} \right]$$

$$P_S = \frac{b_1 + b_2}{2} (H_2 - H_1) \left[D_2 - H_2 + (D_1 - D_2 + H_2 - H_1) \left(\frac{2b_1 + b_2}{3(b_1 + b_2)} \right) \right] \left[\text{Level Invert} \right]$$

$$P_w = \frac{H_1 + H_2}{4} (b_2 - b_1) \left[D_1 + D_2 - \frac{H_1 + H_2}{2} \right]$$

$$P_f = \frac{L(s_1 + s_2)}{4} (b_1 H_1 + b_2 H_2) \qquad \text{Where } s = \left[\frac{Qn(b+H)^{2/3}}{.936(bH)^{5/3}} \right]^2$$

Hydraulics

CONFLUENCES

CIRCULAR CONDUIT UNDER PRESSURE, PIPE INLET
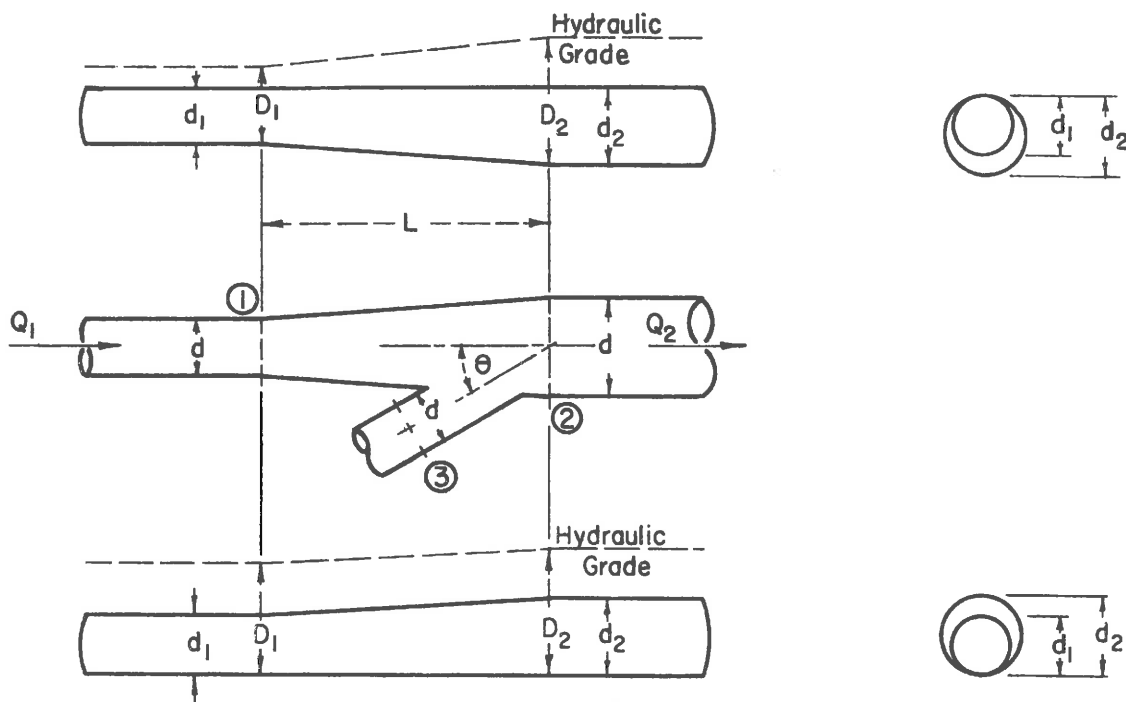
$$P_1 = .785\, d_1^2 \left(D_1 - \frac{d_1}{2}\right) \qquad P_2 = .785\, d_2^2 \left(D_2 - \frac{d_2}{2}\right)$$

$$M_1 = \frac{Q_1^2}{25.2\, d_1^2} \qquad M_2 = \frac{Q_2^2}{25.2\, d_2^2}$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{25.2\, d_3^2}\,(\cos\theta)$$

$$P_i = 0$$

$$P_w = .393\left[d_2^3 - 2d_2 d_1^2 + d_1^3 + (d_2^2 - d_1^2)(D_1 + D_2 - 2d_2)\right]\ \left[\text{Level Invert}\right]$$

$$P_w = .393\left[(d_2^3 - d_1^3) + (d_2^2 - d_1^2)(D_1 + D_2 - d_1 - d_2)\right]\ \left[\text{Level Soffit}\right]$$

$$P_f = .196\, L(s_1 + s_2)(d_1^2 + d_2^2) \qquad \text{Where}\ s = \left(\frac{Qn}{.463\, d^{8/3}}\right)^2$$

CONFLUENCES

CIRCULAR CONDUIT FLOWING PARTIALLY FULL, PIPE INLET
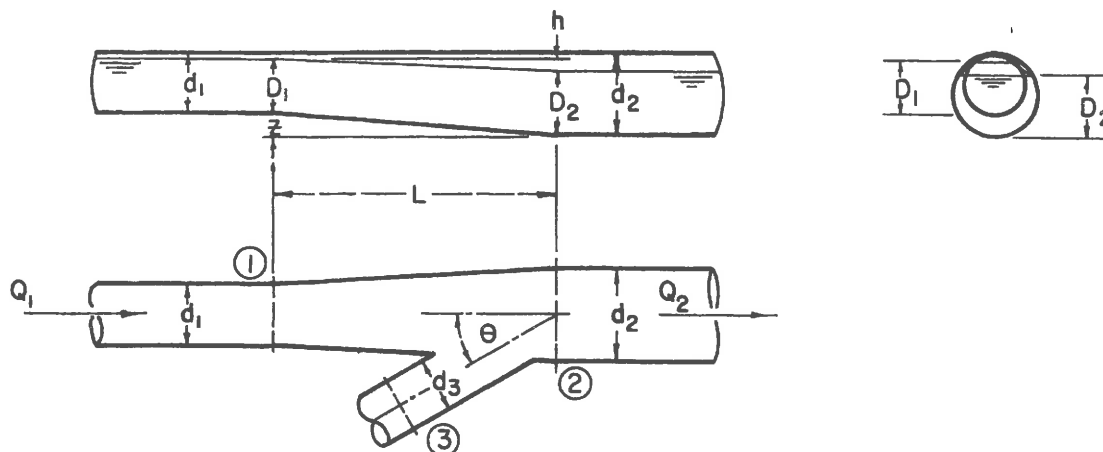
$$P_1 = C_1 d_1^3 \qquad\qquad P_2 = C_2 d_2^3$$

$$M_1 = K_1 \left(\frac{Q_1}{d_1}\right)^2 \qquad\qquad M_2 = K_2 \left(\frac{Q_2}{d_2}\right)^2$$

$$M_3 \cos\theta = \frac{(Q_2 - Q_1)^2}{25.2 d_3^2} (\cos\theta)$$

$$P_i = 0$$

$$* P_w = A_2 \bar{y}_2 - A_1 \bar{y}_1 + \frac{h}{2}(A_2 + A_1) + \frac{(h)^2}{12}(T_2 - T_1)$$

$$P_f = \frac{L(s_1 + s_2)}{4}(A_1 + A_2)$$

For tabulated values of C and K, see Page 13.

See King "Handbook of Hydraulics", for $A_1, \bar{y}$ and T.

$*$ Where $h = \bar{z} + D_1 - D_2$, The term $\frac{(h)^2}{12}(T_2 - T_1)$ is usually negligible.

$\frac{D}{d}$	K	C	F	$\frac{D}{d}$	K	C	F
.01	23.919	.0000	9188.	.51	.0773	.0873	.0958
.02	8.403	.0000	1134.	.52	.0753	.0914	.0912
.03	4.507	.0001	326.	.53	.0736	.0956	.0869
.04	2.961	.0002	140.9	.54	.0719	.0998	.0829
.05	2.115	.0003	71.9	.55	.0703	.1042	.0793
.06	1.620	.0005	42.1	.56	.0687	.1087	.0758
.07	1.285	.0007	26.5	.57	.0672	.1133	.0726
.08	1.058	.0010	17.97	.58	.0658	.1179	.0696
.09	0.888	.0013	12.68	.59	.0645	.1227	.0668
.10	0.760	.0017	9.28	.60	.0632	.1276	.0641
.11	0.662	.0021	7.03	.61	.0620	.1326	.0617
.12	0.582	.0026	5.45	.62	.0608	.1376	.0594
.13	0.518	.0032	4.31	.63	.0597	.1428	.0572
.14	0.466	.0038	3.48	.64	.0586	.1428	.0551
.15	0.421	.0045	2.84	.65	.0575	.1534	.0532
.16	0.383	.0053	2.36	.66	.0565	.1589	.0514
.17	0.351	.0061	1.982	.67	.0559	.1644	.0496
.18	0.324	.0070	1.681	.68	.0547	.1700	.0480
.19	0.299	.0080	1.438	.69	.0538	.1758	.0465
.20	0.278	.0091	1.242	.70	.0530	.1816	.0450
.21	0.259	.0103	1.080	.71	.0521	.1875	.0437
.22	0.243	.0115	0.946	.72	.0514	.1935	.0424
.23	0.228	.0128	0.833	.73	.0506	.1996	.0411
.24	0.215	.0143	0.740	.74	.0499	.2058	.0400
.25	.2026	.0157	0.659	.75	.0492	.2121	.0389
.26	.1916	.0173	0.589	.76	.0485	.2185	.0379
.27	.1817	.0190	0.530	.77	.0479	.2249	.0369
.28	.1727	.0207	0.479	.78	.0473	.2314	.0359
.29	.1645	.0226	0.435	.79	.0467	.2380	.0351
.30	.1569	.0255	0.395	.80	.0462	.2447	.0342
.31	.1493	.0266	0.361	.81	.0456	.2515	.0334
.32	.1435	.0287	0.331	.82	.0451	.2584	.0327
.33	.1376	.0309	0.304	.83	.0446	.2653	.0320
.34	.1320	.0332	0.280	.84	.0441	.2723	.0313
.35	.1269	.0356	0.259	.85	.0437	.2794	.0307
.36	.1221	.0381	0.240	.86	.0433	.2865	.0301
.37	.1177	.0407	0.222	.87	.0429	.2938	.0295
.38	.1135	.0434	0.207	.88	.0425	.3011	.0290
.39	.1096	.0462	.1931	.89	.0421	.3084	.0285
.40	.1060	.0491	.1804	.90	.0418	.3158	.0280
.41	.1026	.0520	.1689	.91	.0414	.3233	.0276
.42	.0993	.0551	.1585	.92	.0411	.3308	.0272
.43	.0963	.0583	.1489	.93	.0408	.3384	.0266
.44	.0934	.0616	.1402	.94	.0406	.3460	.0265
.45	.0907	.0650	.1321	.95	.0403	.3537	.0261
.46	.0882	.0684	.1248	.96	.0401	.3615	.0259
.47	.0857	.0720	.1180	.97	.0399	.3692	.0256
.48	.0834	.0757	.1118	.98	.0398	.3770	.0254
.49	.0813	.0795	.1060	.99	.0397	.3848	.0253
.50	.0792	.0833	.1007	1.00	.0396	.3927	.0252

Tabulated Values

$$M = k(Q/d)^2 \qquad P = C \cdot d^3$$

$$h_v = F(Q/d^2)^2$$

ORANGE COUNTY FLOOD CONTROL DISTRICT

PRESSURE PLUS MOMENTUM FACTORS FOR PARTIALLY FULL CIRCULAR CONDUITS

(3)   Transitions

Most references equate losses in transitions
directly to change in velocity head through use of
coefficients. This method has recently become
somewhat suspect and research tacitly indicates a
future general adoption of specific force (pressure
plus momentum) principles for determining transi-
tion losses. However, unless or until the P + M
method is further documented, energy coefficients
probably represent the most valid criteria for
analysis.[2] Therefore, losses in transitions for
district projects should be determined by:

$$h = K_i \ \frac{V_2^2 - V_1^2}{2g}$$ velocity increase
in transition

$$h = K_o \ \frac{V_1^2 - V_2^2}{2g}$$ velocity decrease
in transition

(a)   Subcritical Velocities

Stable subcritical flows tend to generate
only minor waves[8] and transition design
is usually governed by available head.
The table on the following page should be
used to select transition shape and deter-
mine head losses:

SHAPE	K_i	K_o
Abrupt (square)	0.30	0.80
Straight Line*		
10°	0.10	0.20
15°	0.10	0.30
20°	0.10	0.40
30°	0.10	0.70
Warped Design**	0.10	0.20

- -

*   Angle given is maximum water boundary angle relative to transition centerline.

**   Reversing curves with maximum convergent angle of 12°30' for K_i and maximum divergent angle of 5°45' for K_o.

The values of K_i and K_o listed for determining losses do not include frictional losses except that in well designed curved sections the coefficients are usually adequate to account for both impact and friction. (If impact loss in a well designed curved transition is less than friction alone the friction loss should be used and the impact loss may be ignored.)

(b)   <u>Supercritical Velocities</u>

Most supercritical channels within the district have normal velocities such that the Froude Number is between 1.0 and 2.0. While a portion of this range has been classified as   stable for district projects, authorities generally agree that some degree of instability may exist over the entire range.

Channel Hydraulics

The probability of developing adverse effects from unstable flow is greatest in converging wall transitions (contractions) and this transition should be avoided if possible.

For Froude numbers less than 2.5 the length of contractions, if a contraction is necessary, and the length of expansions shall be determined from the following.[9]

Approach Channel Velocity (fps)	Maximum Wall Deflection (degrees)
10-15	6
15-30	4
30-45	2

The loss in the transition should be determined from:

$$K_i = 0.10$$

$$K_o = 0.20$$

In the rare event of a Froude number exceeding 2.5, transition design should be based on the criteria outlined by Ippen,[8,10] Rouse,[11] et.al. in the ASCE Transactions.

Supercritical transitions should have straight walls since cross-wave height varies directly

with wall deflection angle and for any given
length wall deflection angle is a minimum
with straight connection. Where appearance
is an important consideration, it may be desir-
able to use curved rather than angular changes.

The very limited research has been restricted
to rectangular sections and the given criteria
are directly applicable only to that shape.
However, the criteria will yield reasonable
designs with other sections if maximum water
direction change is substituted for wall deflec-
tion.

(4)   Curves and Angle Points

Curve and angle point losses for district projects
are limited to pressure flow situations since the
losses evident in properly designed open channels
are extremely minor and the treatment of open
channel superelevation wave action is covered under
separate heading.

Curve and angle point losses are additive to fric-
tional losses and are usually equated with velocity
head in the form of the following general equation.

$$h_L = K \left(\frac{V^2}{2g} \right)$$

(a)   Curve Losses

Curve losses shall be calculated by the
formula

$$h_L = .25 \, K_b \left(\frac{V^2}{2g} \right)$$

Channel Hydraulics

where $K_b = \sqrt{\dfrac{\triangle}{90}}$ and $\triangle$ is

the central bend angle in degrees.

(b) <u>Angle Point Losses</u>

Angle point losses shall be calculated by
the formula

$$h = K_a \left(\frac{V^2}{2g} \right)$$

where $K_a = 0.02$ for a deflection

angle of 6 degrees and K_a varies

uniformly to zero with diminishing

angles.[12]

Deflection angles greater than 6 degrees
should not be used except in connector pipes
and then only if a lesser angle is not econo-
mically feasible.

(5) <u>Bridge Piers</u>

(a) <u>Introduction</u>

Any facility having an obstruction must be
designed to permit flow through the obstructed
section without reducing the capacity of the
system. Treatment herein of obstructions is

limited to open channel bridge piers since
pressure flow losses from obstructions occurs
infrequently.

Three classes of flow that may exist at a
section constricted by piers are defined
below and illustrated on page 23.

Class A

Subcritical flow exists upstream
from the obstruction, around the
obstruction and downstream.

Class B

Subcritical flow exists upstream
from the obstruction, critical at
the obstruction, and subcritical
or supercritical downstream.

Class C

Supercritical flow exists upstream
from the obstruction, around the
obstruction and downstream.

Two principal methods of analysis have evolved
from research of flow past piers. One is the
energy method where characteristics of flow
are stated in terms of velocity head and the
other is the specific force method where analy-
sis is based on P+M techniques.

Either the energy method (usually referred to
as "Yarnell's Method") or the P+M method may
be used to analyze Class A flow situations.
On the other hand, only P+M should be used
for Class B and Class C determinations.

(b) Class A Bridges

In the solution of problems by Yarnell's Method,

Channel Hydraulics

the upstream depth may be determined by the following equation.

$$d_1 = 2K(K + 10\omega - 0.6)(\propto + 15 \propto^4)\,\frac{V_3^2}{2g} + d_3$$

ω is velocity-headwater-depth ratio of the downstream section of channel.

$\propto$ is the contraction ratio and is the cross sectional pier area divided by the cross sectional flow area.

K is selected from the following table.

PIER SHAPE	VALUE OF K
Semicircular nose and tail	0.90
Twin-cylinder piers with connecting diaphragm	0.95
Twin-cylinder piers without diaphragm	1.05
90 degree triangular nose and tail	1.05
Square nose and tail	1.25

(c) Class B and Class C Bridges

The determination of pier losses and water surface profiles for Class B and Class C flows requires use of the pressure plus momentum principles. Total momentum force, referred to herein as specific force, includes static (pressure) force (P) and momentum force (M) and may be written:

$$F = A\bar{y} + \frac{QV}{g} = A\bar{y} + \frac{Q^2}{gA}$$

Experimenters have determined that a short flat obstruction such as a bridge pier causes a total kinetic loss approximately equal to $(A_0/A_1)(Q^2/gA_1)$ where A_0 is area of obstruction and A_1 is water area in upstream unobstructed channel.

By letting subscripts 1, 2, and 3 represent conditions upstream, within and downstream, respectively, of the constricted area and equating the force at these sections the general momentum equation can be written:

$$\text{I} \qquad A_1\bar{y}_1 - A_0\bar{y}_0 + \frac{(A_1 - A_0)Q^2}{gA_1^2} =$$

$$\text{II} \qquad A_1\bar{y}_1 - A_0\bar{y}_0 + \frac{Q^2}{g(A_1 - A_0)} =$$

$$\text{III} \qquad A_1\bar{y}_1 - A_0\bar{y}_0 + \frac{Q^2}{gA_3}$$

If this equation is plotted as a function of water depth, three curves evolve. With the usual plot of increasing depth as the ordinate and increasing P+M as the abscissa, these curves are concave to the right with minimums at critical depth and represent the total momentum at each section for various depths.

From these three curves and the known elements

(e.g. the upstream depth for Class C bridges) the remaining unknowns are readily available. A typical plot of the curves is shown on Page 23.

(d) Pier Extension and Debris Allowances

Streamlined debris walls reduce the debris collection of pier walls of culverts and bridges. The inclined, curved slope of the pier extension is designed to allow the debris to ride to the top of the water surface. Hence, effective debris area is a six-foot distance below the water surface times a variable width to allow for debris production capabilities of the channel system plus the projected area of the pier extension below the water surface. Four categories of debris wall criteria are defined as follows:

Category 1: No debris wall and no debris factor.

Category 2: Construct debris wall but apply no debris factor

Category 3: Construct debris wall and apply one-foot wide debris factor.

Category 4: Construct debris wall and apply three-foot wide debris factor.

The debris criteria category to be used for each channel are listed on Pages 22b through 22d.

Two exceptions are noted. (1) If an existing facility is without debris walls, the full depth of water shall be used; (2) The engineer shall give special consideration to each instance where the projected pier of a skewed pier or piling row is larger than the debris area.

The streamlined extensions shall conform to the dimensions shown on the sketches on Pages 24 and 24a.

In the selection of categories, consideration has been given to drainage area, location, type of channel, anticipated watershed development, and level of protection. The designer may consider special circumstances which may become evident in detailed design studies, and may deviate from the

tabulated categories with written justification approved by the Design Division Engineer. Additionally, as some of the watersheds develop, their debris production capability may diminish, thereby justifying a lower number category.

Clear spans (or reduced number of spans) in lieu of piers with extensions are recommended for Category 4 facilities whenever the cost of the clear span does not increase cost significantly. The determination of significance should include consideration of the consequences of a larger amount of debris (than Category 4) catching in the pier.

Channel Debris Categories

Number	Channel Name	Category
A01	Coyote Creek Channel	3
A02	Brea Creek Channel	3
A03	Fullerton Creek Channel	3
A03S01	Buena Park Storm Channel	2
A03S02	Houston Storm Channel	2
A03S03	Ash Storm Channel	2
A03S05	Kimberly Storm Channel	2
A04	Brea Canyon Channel	4
A04S01	Memory Gardens Storm Channel	2
A05	East Brea Channel	2
A06	Loftus Diversion Channel	3
A07	Imperial Channel	2
A08	La Mirada Channel	3
A09	Tonner Canyon Channel	4
B01	Carbon Creek Channel	3
B01S01	Cypress Storm Channel	2
B01S03	Placentia Storm Channel	2
B02	Moody Creek Channel	2
B02S02	Dairyland Storm Channel	2
C01	Los Alamitos Channel d/s of Garden Grove Blvd.	1
C01	Los Alamitos Channel u/s of Garden Grove Blvd.	2
C01S01	Kempton Storm Channel	2
C01S02	Rossmoor Storm Channel	2
C01S03	Montecito Storm Channel	2
C01S04	Bixby Storm Channel	2
C01S05	Katella Storm Channel	2
C01S06	Federal Storm Channel	2
C02	Bolsa Chica Channel	2
C02S01	Stanton Storm Channel	2
C02S03	Jonathan Storm Channel	2
C03	Anaheim-Barber City Channel	2
C03S01	Milan Storm Channel	2
C03S02	Humboldt Storm Channel	2
C03S03	Bestel Storm Channel	2
C03S04	Rosalia Storm Channel	2
C03S05	Shannon Storm Channel	2
C04	Westminster Channel	2
C04S02	Bolsa Grande Storm Channel	2
C05	East Garden Grove-Wintersburg Channel	2
C05S01	Newland Storm Channel	2

Number	Channel Name	Category
C05S04	Slater Storm Channel	2
C05S05	Edinger Storm Channel	2
C05S10	Newhope Storm Channel	2
C05S11	Lewis Storm Channel	2
C06	Ocean View Channel	2
C07	Sunset Channel	2
D01	Huntington Beach Channel	1
D02	Talbert Channel	1
D03	Greenville-Banning Channel	2
D03S03	Gisler Storm Channel	2
D04	Fairview Channel	2
D05	Fountain Valley Channel	2
E01	Santa Ana River Channel	4
E01S01	East Richfield Channel	3
E02	Carbon Canyon Diversion Channel	3
E03	Carbon Canyon Channel d/s of dam	3
E03	Carbon Canyon Channel u/s of dam	4
E04	Atwood Channel	2
E05	Richfield Channel	2
E07	Collins Channel	2
E07S01	Marlboro Channel	2
E08	Santiago Creek Channel	4
E08S02	Alameda Storm Channel	2
E08S06	Handy Creek Storm Channel	3
E10	Fletcher Storm Channel	2
E11	Bitterbush Channel	2
E13	----------------------	4
E14	----------------------	4
E15	----------------------	4
E16	----------------------	4
E17	Silverado Canyon	4
E18	----------------------	4
E19	----------------------	4
E20	----------------------	4
E21	----------------------	4
E22	----------------------	4
E23	----------------------	4
F01	Santa Ana-Delhi Channel	2
F01S01	Airport Storm Channel	2
F02	Santa Ana Gardens Channel	2
F03	Paularino Channel	2
F04	Bonita Channel	3
F05	San Diego Creek Channel d/s of San Diego Freeway	4
F05	San Diego Creek Channel u/s of San Diego Freeway	3
F06	Peters Canyon Channel d/s of Santa Ana - Santa Fe Channel	4

Number	Channel Name	Category
F06	Peters Canyon Channel u/s of Santa Ana-Santa Fe Channel	3
F06S02	Valencia Storm Channel	2
F06S03	Como Storm Channel	2
F07	El Modena-Irvine Channel	2
F08	Lane Channel	2
F09	Barranca Storm Channel	2
F10	Santa Ana-Santa Fe Channel	2
F11	Southwest Tustin Channel	2
F12	North Tustin Channel	2
F13	Redhill Storm Channel	2
F14	San Joaquin Channel	2
F14S01	Culver Storm Channel	2
F15	Sand Canyon Channel	3
F16	Irvine Ranch Channel	3
F17	-----------------------	3
F18	-----------------------	3
F19	-----------------------	3
F20	-----------------------	3
F21	-----------------------	3
F23	-----------------------	3
F24	-----------------------	3
F25	Central Irvine Channel	3
F26	Rattlesnake Canyon Channel	3
F27	-----------------------	3
I02	Laguna Canyon Channel	3
J01	Aliso Creek Channel	4
J07	English Canyon Channel	4
L01	San Juan Creek Channel	4
L02	Trabuco Creek Channel	4
L03	Oso Creek Channel	3
L04	La Paz Channel	3
L05	Horno Creek Channel	4
L06	-----------------------	4
L07	-----------------------	4
L08	-----------------------	4
L09	-----------------------	4
L10	-----------------------	4
M01	Prima Deschecha Canada Channel	3
M02	Segunda Deschecha Canada Channel	3
M03	Christianito Canyon Channel	4
M04	-----------------------	4
M05	-----------------------	4
M06	-----------------------	4

PLAN

PIER

I   II   III

FLOW

BRIDGE PIER

CLASS A
CLASS B

CLASS A FLOW

$D_1 > D_c$

$D_1 > D_c$

$D_2 > D_{c2}$

$D_2 = D_{c2}$

$D_3 > D_c$

$D_3 > D_c$

CLASS B FLOW

$D_2 = D_{c2}$

CLASS C FLOW

CLASS C

$D_2 < D_{c2}$

$D_3 < D_c$

$D_3 < D_c$

$D_1 < D_c$

WATER DEPTH
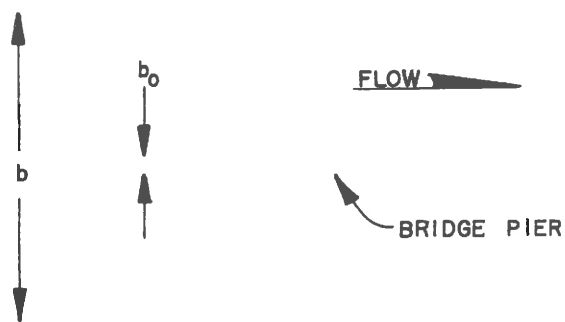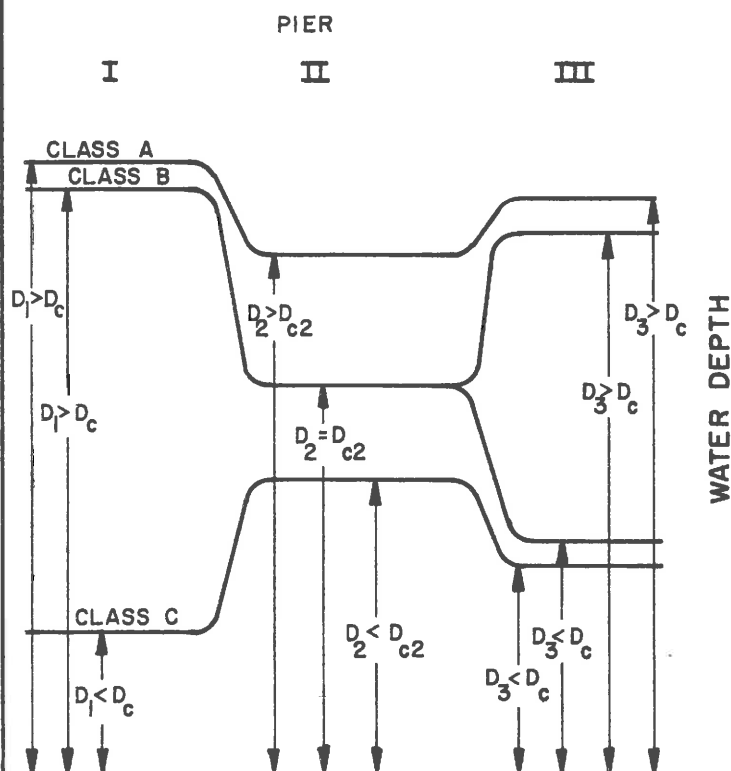
ELEVATION
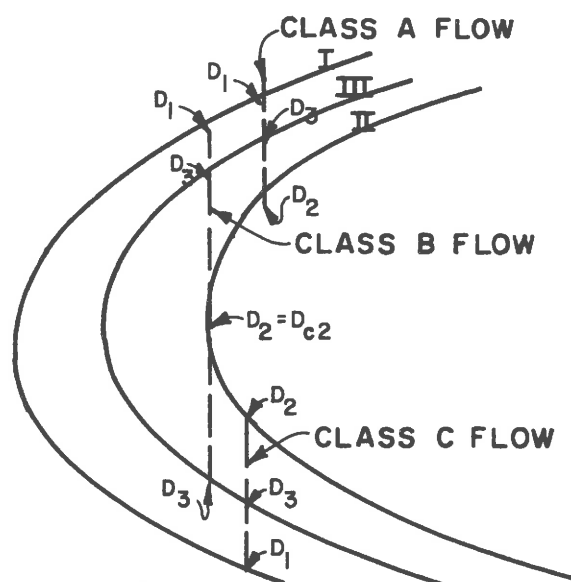
P + M
PRESSURE + MOMENTUM CURVES

NOTATIONS:

b_o = TOTAL PIER WIDTH

b = CHANNEL WIDTH

D_1 = UPSTREAM DEPTH

D_2 = DEPTH WITHIN PIER SECTION

D_3 = DOWNSTREAM DEPTH

D_c = CRITICAL DEPTH WITHIN UNOBSTRUCTED
CHANNEL SECTION

D_{c2} = CRITICAL DEPTH WITHIN PIER SECTION

ORANGE COUNTY FLOOD CONTROL DISTRICT

BRIDGE PIER
PRESSURE + MOMENTUM CURVES

SECTION A-A

P.I. of curve

3
1

1
1

2.5' min.

A

A

Radius = 3D

$\frac{D}{2}$

a = levee ht.

D = max. water depth

$\frac{D}{2}$

$3\frac{D}{2}$

6.0'

Pier Length

Radial

Radius

1.0'

1.0'

Pier width

For existing pier
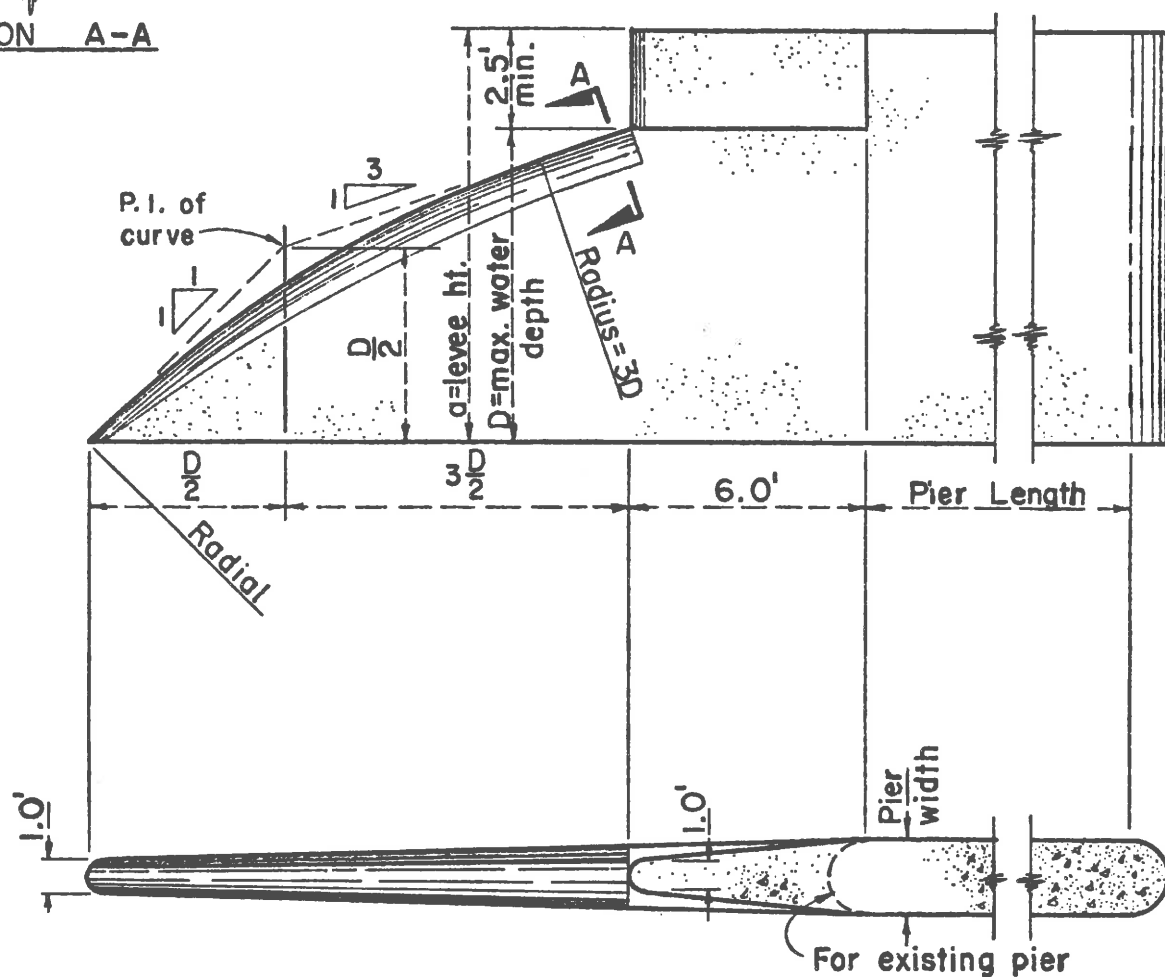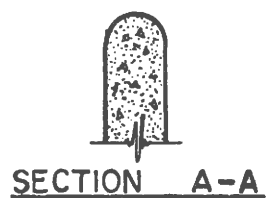
ORANGE COUNTY FLOOD CONTROL DISTRICT

TYPICAL PIER EXTENSION
BRIDGE

Rev. November 1974

SECTION A-A

Top of parapet

* If culvert is sealed D=depth of water.

Soffit

24" Max.

A
A

P.I. of curve

3
1

1
1

Radius = 3D

$\frac{D}{2}$

a=levee ht.
*D=max. water depth

$\frac{D}{2}$

3$\frac{D}{2}$

3.0'

Radial

W

T
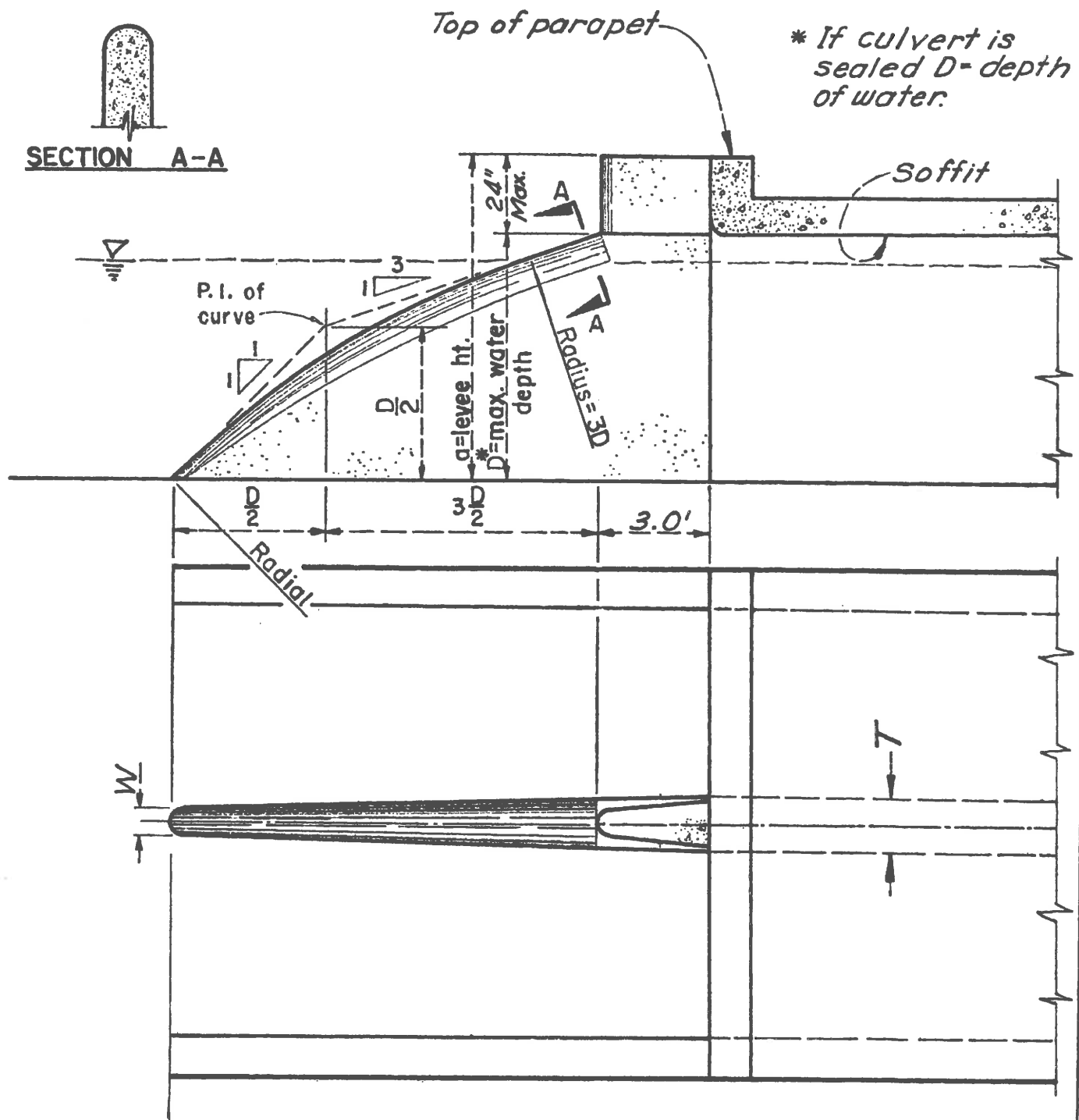
T= Wall width
W= Pier ext. width

If   T< 8", W=T
If 8"≤ T < 12", W=8"
If 12"≤ T   , W=12"

ORANGE COUNTY FLOOD CONTROL DISTRICT

TYPICAL PIER EXTENSION
CULVERT

Rev. November 1974

E.   SUPERELEVATION AND WAVE ACTION

(1)   General

The design of curved sections having free water
surfaces should include allowances for superele-
vation and, in the supercritical regime, for wave
action unless the waves are effectively controlled
by refinements in design.

The following equations should be used to determine
superelevation (including wave height) for single
isolated curves.

SECTION	SUBCRITICAL VELOCITY	SUPERCRITICAL VELOCITY
Rectangular	$e = \dfrac{V^2 b}{2gR}$	$e = \dfrac{V^2 b}{gR}$
Trapezoidal	$e = \dfrac{1.2V^2 (b + 2zD)}{2gR}$	$e = \dfrac{1.3V^2 (b + 2zD)}{gR}$

"e" is the rise in water surface above mean depth
in an equivalent straight reach.

Trapezoidal coefficients include 20 and 30 percent
safety factors.

The inclusion of a formula for superelevation in
supercritical trapezoidal channels should not be
construed to mean ready acceptance for this type
design. In fact, this method of conveyence would
be accepted only if accompanying unusually large
freeboard and a long straight reach of channel on
each end of curve.

The designer should remember that cross waves continue to oscillate in downstream tangents for relatively great distances and that wave amplitudes may be additive if not completely dampened between curves.[13]

(2)   Curve Wave Dampening[13]

The two most used methods of cross wave control involve the use of compound curves and spiral curves. However these two methods, while quite effective wave devices, have little effect on the rise in water surface normally associated with centrifugal force calculations. The third and probably most effective device, one which controls both superelevation and wave action, is invert banking coupled with spiral-transitioning. This method, unfortunately, usually introduces complexities in right-of-way descriptions and field construction.

The following criteria should be used for curved supercritical channels:

(a)   Where calculated superelevation is 0.5 feet or less an equivalent increase in wall heights (both walls) is satisfactory. The increased wall heights should extend 1200 feet (+) into the downstream tangent.

(b)   Where calculated superelevation is between 0.5 feet and 1.0 feet increase wall heights and dampen the cross waves with either compound curves or spiral curves.

Compound curves shall consist of three elements: a central curve and two exterior curves. The exterior curves shall have radii equal to double that of the central curve and shall have a length equal to one half the wave length. One half the wave length may be determined by the formula:

$$\frac{\pi\ bV}{\sqrt{12\ gD}} = \frac{0.908b}{\sin B} \qquad \text{where}$$

B is wavefront angle and Sin B $= \dfrac{\sqrt{gD}}{V} = \dfrac{1}{F_r}$

Spiral curves, if used, shall be placed both upstream and downstream of the central circular curve. The minimum length of spiral shall be determined by the formula:

$$L_s = \dfrac{1.82Vb}{\sqrt{gD}} = \dfrac{.32bV}{\sqrt{D}}$$

However, the selected spiral shall be of such length that field layout can be made on the basis of full 10-foot chords.

(c) Where calculated superelevation exceeds 1.0 feet or where downstream conditions require a smooth water surface (e.g. reversing curves), invert banking along with spiral curve transitions should be used.

The banked invert should have a channel cross slope equal to the calculated superelevation:

$$e = \dfrac{V^2 b}{Rg} \qquad \text{(Rectangular Section)}$$

The spiral transitions should have a length at least equal to 30 times the calculated superelevation.

Channel Hydraulics

All criteria outlined herein for curved
sections appear to be satisfactory where:

$$\frac{h_v}{R} \lessgtr 0.04$$

This maximum ratio of velocity head to
centerline radius should be maintained for
all supercritical channels.

(3) **Confluence Wave Dampening**

Cross waves will form in improperly designed super-
critical confluences and wave action will extend
into downstream sections relatively large distances.
In severe cases a hydraulic jump may occur voiding
P+M confluence calculations and creating high back-
water from the confluence.

To avoid the hydraulic jump and minimize downstream
wave effects confluences should be designed in accord-
ance with the following:

(a) The combining sections should be shaped to
provide approximately equal water surface eleva-
tions at the upstream end of the confluence.

(b) The main section should be enlarged through
the confluence to maintain approximately con-
stant flow depths throughout.

(c) The angle of flow intersection must be held to
(preferably zero but) less than 15 degrees.

At locations where inlet hydraulics permit a side-
channel spillway may be used. This method discharges
the side flow over a long weir at reduced velocity
such that the incremental increase in main flow is

extremely small.[9] The design requires a spillway crest slightly above and parallel to design water surface in the main facility of a sufficient length that crest depth (critical depth) is approximately one foot.

F. FREEBOARD AND HYDRAULIC GRADELINE LIMITATIONS SEE ADDENDUM #3

Freeboard for the purposes of this manual is the distance from the design hydraulic gradeline (plus wave height and superelevation) to the

(1) top of levee in ultimate unlined earth levee channels

(2) top of rock where riprap slope protection is utilized

(3) top of wall or structural section in concrete channels

(4) soffit where box conduits or culverts are to be designed as open channels.

Unless pumping stations for tributary drainage are an economical alternative, the design hydraulic gradeline in the main trunk must be sufficiently below surrounding ground to accomodate local drainage. A design hydraulic gradeline at least 2 feet below street gutter grade should be provided for local inlets. Design in undeveloped areas should consider the possibility that future street elevations may be one or more (usually 2) feet below average existing ground. Under these conditions freeboard may be determined by surrounding ground conditions or wall-height economics. The following freeboard allowances will usually be acceptable:

(1) Leveed channels - Two feet freeboard for small facilities with water surface not more than 4 feet above surrounding ground. Additional freeboard up to 4 feet should be considered for channels with higher levees and/or greater design frequency than 25 years.

In case of supercritical flow, freeboard should be added above the conjugate depth unless the probability of this greater depth is extremely small.

(2) Channels with design frequency <u>greater</u> than 25 years - Freeboard--1.5 feet.

(3) Channels with design frequency equal to 25 years - Freeboard--1.0 feet.

(4)   Channels with design frequency equal to 10 years -
      Freeboard--0.5 feet.

These criteria should not be accepted, however, without
consideration of the consequences of encroachment upon the
freeboard reservation.   In many instances the design hydro-
graph may indicate a peak of such short duration that the
addition of freeboard would be unreasonably conservative.
In other instances, e.g. concrete channels set below surround-
ing ground, flows even slightly above top of concrete may be
acceptable for short periods. Consideration should be given
to the hydraulic needs of tributary drainage. Generally,
when available friction slope for storm drains falls below
0.001($\pm$) the cost of the storm drains increases dispropor-
tionately.

REFERENCES USED

1. King, H. W., Handbook of Hydraulics, McGraw-Hill Book Co., Inc., New York (1963)

2. Chow, V. T., Open Channel Hydraulics, McGraw-Hill Book Co., Inc., New York (1959)

3. Report of Task Force, "Friction Factors in Open Channels", Proceedings, ASCE, Hydraulics Division, Journal, Vol. 89, HY2 (1963)

4. Tilp, Paul J., "Capacity Tests in Large Concrete Lined Canals", Transactions, ASCE, Vol. 131 (1963)

5. U. S. Bureau of Reclamation, Design of Small Dams, Government Printing Office, Washington (1960)

6. Barnes, Harry H. Jr., "Roughness Characteristics of Natural Channels", United States Geological Survey, Water Supply Paper No. 1849 (1967)

7. U. S. Army Engineer District, Los Angeles, "Hydraulic Model Study, Los Angeles River Channel Improvement, Stewart and Gray Road to Pacific Electric Railway", Los Angeles (1947)

8. Ippen, A. T., "Mechanics of Supercritical Flow", Transactions, ASCE, Vol. 116 (1951)

9. U. S. Army, Office, Chief of Engineer, Hydraulic Design of Flood Control Channels, Engineer Manual EM 1110-2-1601 (1970)

10. Ippen, A. T. and Dawson, J. H., "Design of Channel Contractions", Transactions, ASCE, Vol. 116 (1951)

11. Rouse, H., Bhoota, B. V., and Hsu, En-Yun, "Design of Channel Expansions", Transactions, ASCE Vol. 116 (1951)

12. Report of Task Force, "Factors Influencing Flow in Large Conduits", Proceedings, ASCE, Hydraulics Division, Journal, Vol. 91 HY6 (1965)

13. Knapp, R. T., "Design of Channel Curves for Supercritical Flow", Transactions, ASCE, Vol. 116 (1951)

PUBLIC WORKS MAINTENANCE REQUIREMENTS

A. Channel, Perimeter and Access Roadway

1. A 15' wide satisfactory all-weather roadway, aggregate base or equal, located adjacent to the channel within a 20' horizontal clear area shall be provided.

2. All roadways shall be continuous, except a 50' x 50' turnaround shall be provided at major obstacles such as freeways, railroads, etc. A cul-de-sac of R=35' may be used in lieu of a 50' x 50' turnaround.

3. The minimum radius horizontal curve shall be R=35' (for inside edge of track.)

4. The minimum 12' clear roadway width shall be provided adjacent to and around access ramps.

5. For leveed conditions, satisfactory back-slope stabilization shall be used to control erosion and sloughing. Such stabilization may include but not limited to walls, slope flattening, slope lining, drainage devices, landscaping, etc.

6. Roadways shall be provided on both sides for a channel with a top width greater than 30'. For a channel with a top width less than 30', a 5' walk path may be used on one side in lieu of one of the access roadways.

7. The roadway and walk shall slope to the channel at 2%. If the roadway slopes away from channel, a drainage collection system shall be provided to control storm flow entry into the channel.

B. Access Ramps

1. Ramps shall have a maximum 10% grade and should slope downstream. Ramps shall have a minimum of 2% cross-slope toward channel.

2. For a vertical-wall channel, the channel wall shall extend 1 foot above the ramp to form a 1-foot high curb. This curb shall transition to O-inch at the invert gradeline within a distance of 5 feet.

3. Access ramp width, excluding walls and curbs, shall be a minimum of 12 feet.

4. A 30-foot longitudinal landing pad shall be provided. The wall transition shall have an angle of 45° degrees, starting at the end of the 30' landing pad.

5. The area from the street entry to the beginning of the access ramp shall be paved with a road structural section (.35' AC/.50' AB or equivalent).

6. Ramps shall be installed as needed to provide continuous access but in no case shall ramp spacing be greater than 1 mile. Access is normally required for but not limited to the following:

   a. Retarding basins
   b. Between grade stabilizers
   c. Between drop structures
   d. Where access under bridges, culverts, etc. is restricted (less than 12')
   e. All channels with a bottom width greater than 12 feet.

7. Ramp surface shall have a transverse heavy broom finish for traction.

8. Storm drains shall not outlet onto the ramp surface.

9. Ramps shall be located as close as possible to cross streets, but at least 50 feet from the street right-of-way.

C. Entrances and Gates

1. Minimum 16-foot opening, double swing gates shall be provided.

2. Gates shall be set back a minimum of 20-feet from the right-of-way.

3. A minimum 16-foot wide concrete drive approach shall be provided.

4. Road structural section (.35' AC/.50' AB or equivalent) from drive approach to gate shall be provided and shall include provisions for adequate drainage.

5. Gates shall open inward (toward levee road).

6. Access onto channel maintenance roads shall be provided at all highway crossings.

7. A 12' wide median opening shall be required where access conditions warrant.

D. Access Under Bridges

1. Maintenance roadways shall be built under bridges only where there is a trail requirement.

2. The minimum clearance shall be 12' below soffit of bridge.

3. Roadways shall be a minimum of 2.0' above channel grade line.

4. Portland Cement Concrete (PCC) with a transverse heavy broom finish shall be used for roadways under bridges.

5. Caltrans warning signs posting height of clearance under bridges shall be installed.

E. Access Shafts for Structural Box Culverts

1. Openings shall be 20' minimum in length and the entire width of structures.

2. Spacing between access shafts shall be between 0.5 mi. to 1.0 mi.

3. Openings shall be placed outside the roadway and pedestrian travelway or within the raised median area. A minimum of 3' clearance shall be provided from an above-ground shaft to the curb face if used in a raised median condition.

4. H-20 loading shall be provided for grate covers on shafts which project less than 30" above finished grade. For grate covers projecting 30" or greater, 10,000 lb. loaded pickup truck loading condition may be used in lieu of H-20 loading.

5. The maximum weight of any portion of the grate cover shall not exceed 6,000 lbs.

6. Covers shall have lifting hooks for use by a crane in maintenance operations.

F. Rock Riprap

1. Shall be placed a minimum of 25' upstream and downstream of structures related to main channel flow. Drop structures shall require an independent design. Additional riprap may be required depending on flow and channel characteristics.

2. Shall be placed where wave action, angle of attack or flow velocities may erode side slopes. Size, shape and location of riprap shall require independent design.

G. Storm Drain Depth

1. Pipes with cover greater than 20' shall be a minimum of 60" diameter and for larger sizes, pipe diameter shall be 12" larger than required for hydraulics. Design loads shall be increased by 25% for pipe cover greater than 25'.

NOTES-

1. MINIMUM HORIZONTAL RADIUS CURVE SHALL BE 35 FT. FOR INSIDE TRACK.

2. MAXIMUM DISTANCE BETWEEN RAMPS SHALL BE ONE MILE.

MAJOR OBSTACLES FREEWAY, RAILROAD ETC. WHEN AN UNDERCROSSING CAN NOT BE CONST- RUCTED.

R/W

50'

50'
TURNAROUND

FLOW

R/W

20'
CLEAR AREA

15' ALL WEATHER ROADWAY

A

20'
CLEAR

A

45°

12'
CLEAR

5' CURB TRANSI- TION TO 0' HEIGHT AT INVERT GRADELINE

2%
MIN.

30'
LANDING

30'
LANDING

12'
CLEAR

B

10%
MAX.

10%
MAX.

B

2%
MIN.

12' MIN.

CLEAR

2%
MIN.
12' MIN.
CLEAR

12'
CLEAR

AROUND RAMPS TYP.

12'
CLEAR

R/W

R/W

50' MIN. TYP.

2'
TYP.

R/W

R/W

16' WIDE DOUBLE SWING GATE

.35AC/.50AB FROM ROADWAY TO RAMP

STREET R/W

20' MIN. TO STREET R/W

15'

16' MIN. PCC DRIVE APROACH

DRIVE APROACH AREA

ORANGE COUNTY - EMA
PUBLIC WORKS MAINTENANCE REQUIREMENTS

1987	1 OF 2	

SECTION A-A

R/W
20'*
CLEAR AREA
15'
ROADWAY
2%
TOP WIDTH
℄
20'*
CLEAR AREA
15'
ROADWAY
2%
R/W

ALL WEATHER
SURFACE - TYP.

SATISFACTORY
BACK SLOPE
PROTECTION

GRADELINE

SECTION B-B

R/W
12'
ROADWAY
12'
CLEAR
RAMP
2%
℄
12'
CLEAR
RAMP
2%
12'
ROADWAY
1'

ALL WEATHER
SURFACE - TYP.

NOTES-

1. * FOR A CHANNEL WITH A TOP
   WIDTH OF LESS THAN 30 FT.
   A 5 FT. WALK PATH MAY BE
   SUBSTITUTED ON ONE SIDE
   OF THE CHANNEL.

2. STORM DRAINS SHALL NOT
   OUTLET ONTO RAMP SURFACE.

3. ACCESS RAMPS AND UNDER
   CROSSINGS SHALL HAVE A
   TRANSVERSE HEAVY BROOM
   FINISH.

4. MAINTENANCE ROADWAYS SHALL
   BE BUILT UNDER BRIDGES ONLY
   WHERE THERE IS A TRAIL
   REQUIREMENT.

CALTRANS WARNING SIGN
POSTING HEIGHT OF
CLEARANCE

15'
2%
12' MIN.
2' MIN.

GRADE LINE
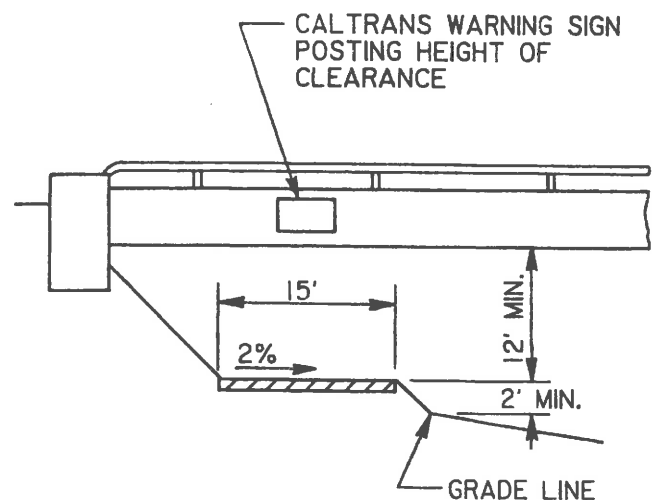
ACCESS UNDER BRIDGES

ORANGE COUNTY - EMA		
PUBLIC WORKS MAINTENANCE REQUIREMENTS		
1987	2 OF 2	

CHAPTER II

STRUCTURES

A. GENERAL

The structural design criteria have been selected from codes and specifications to satisfy the special needs of the district and the requirements of the various agencies from whom approval must be obtained for district projects. Where criterion is not specifically established the applicable provisions of the following references shall be used.

(1) Reinforced Concrete

Building Code Requirements for Reinforced Concrete (ACI 318-71) of the American Concrete Institute.

(2) Structural Steel

Manual of Steel Construction of the American Institute of Steel Construction.

(3) Railroad Structures

AREA Manual of the American Railway Engineering Association.

(4) Highway Structures

Standard Specifications for Highway Bridges adopted by the American Association of State Highway Officials, 1969.

The great majority of district projects are constructed of reinforced concrete; therefore, most criteria included herein are directed toward design of reinforced concrete members.

In recent years there has been a strong movement away from the traditional straight-line theory (working stress method) for design of reinforced concrete and toward ultimate strength design. This movement has not included the railroads, however, and the working stress method must be used for design of railway structures. On other structures either method properly applied, may be used for district projects although the ultimate strength method is preferred. (It is anticipated that

after a reasonable adjustment period the working stress method will be deleted as an option on other than railway structures.)

Until certain problems with locally produced concrete, primarily excessive cracking, have been resolved, the district will not adopt criteria which would have the effect of producing even thinner concrete members. However, upon resolution of the problems it may be anticipated that higher strength concrete (f'_c = 3500 psi) and steel (Grade 60) will become the district standard. It may further be anticipated that with expected improved construction control the low phi factors specified for use with ultimate strength design will be increased to more nearly coincide with the ACI code specification.

B. MATERIAL STRENGTHS

The design of reinforced concrete members, except pipe, should be based on:

Concrete having a 28-day compressive strength
  f'_c = 3000 psi

Reinforcing steel conforming to ASTM A-615
  Grade 40

n = 9.2

except that in cases where for architectural or construction reasons oversizing of members is necessary, design based on reduced strength materials may be acceptable. However, relaxing strength requirements may not result in cost savings since contractors often order additional cement for concrete (especially in formed members) to accelerate cylinder strength and permit early form stripping.

In the event reduced strength materials is indicated by cost analysis or other factors, the minimums shall be 2500 psi for concrete and Grade 40 for steel.

The design of reinforced concrete pipe (over 108 inches) should be based on:

Concrete having a 28-day compressive strength
$f_c' = 4500$ psi

Reinforcing steel conforming to ASTM A-615, Grade 60

n = 8

The following are allowable stresses for use with the Working Stress Method.

WORKING STRESS METHOD

Concrete	Any Strength	Stress for $f_c'-3000$psi	Maximum stress
Flexure, f_c			
Extreme fiber in Compression	$0.45f_c'$	1350 psi	---
Extreme Fiber in Tension (plain concrete)	$1.61 \sqrt{f_c'}$	88	---
Extreme fiber in Tension (Reinforced concrete)	none		---
Shear, v			
Beams without web reinforcing	$1.1 \sqrt{f_c'}$	60	---
Horizontal shear in shear keys	$0.10 f_c'$	300	---

WORKING STRESS METHOD

Concrete	Any Strength	Stress for f_c'-3000psi	Maximum stress
Bond, u			
Top bars (horizontal bars with more than 12 inches of concrete cast in the member below the bar)	$\dfrac{3.4 \sqrt{f_c'}}{D}$	185/D	350 psi
All others	$\dfrac{4.8 \sqrt{f_c'}}{D}$	260/D	500
Bearing, f_c			
On full area	$0.25\ f_c'$	750	---
On 1/3 area or less	$0.375\ f_c'$	1125	---

Reinforcing Steel		Allowed Stress
Tension		
Flexural members and web reinforcing	(Grade 60)	24,000 psi
	(Grade 40)	20,000 psi
Compression		
Combined flexure axial stress		nf_c
Compression, flexural members		n times the compression in the surrounding concrete

The following criteria shall be used with the ultimate strength method.

ULTIMATE STRENGTH METHOD

(1) Load Factors

In lieu of the factors shown in ACI 318-71 Load factors shall be in accordance with ACI 318-63. For structures where earthquake and wind loads may be neglected:

$$U = 1.5D + 1.8L$$

(2) Phi Factors

In lieu of the values shown in the ACI codes the following shall be used:

Flexure	$\emptyset = 0.75$
Diagonal tension, bond, and anchorage	$\emptyset = 0.75$
Spirally reinforced compression members	$\emptyset = 0.70$
Tied compression members	$\emptyset = 0.65$

C. LOADS ON STRUCTURES

(1) General

Structures shall be designed, in general, for the dead weight of the structure and other dead loads and live-loads in combinations which produce the greatest stress in the various parts of the structure.

The usual dead load may include existing or proposed structures such as buildings or abutments.

Live loads for structures in highways and district rights-of-way are normally as detailed in AASHO specifications except that the AASHO "truck train" is not recognized and wheel load spread has been modified for district projects to agree with that used by most California agencies.

Live loads for structures within railway rights-of-way must be designed in accordance with the requirements of the affected railroad.

The design of most structures, both highway and rail, require consideration and inclusion of live load impact.

(2) **Unit Weights**

Portland Cement Concrete	150 pcf
Asphalt Concrete	145 pcf

Railroad Items:

Ballast including ties	120 pcf
Rails and fastenings	200 plf
Earth (if AREA used)	120 pcf

Earth in accordance with instruction included herein.

Other items per AASHO or ACI Specifications.

(3) **Buried Conduits - Dead Loads**

(a) Vertical Earth

Unless soils analysis or judgement indicates an actual unit weight significantly different,

design weight of earth for both pipe and box
conduits may be assumed to be

110 pcf

which is the unit weight usually assumed in
design of D-load pipe but is a decided depar-
ture from the AASHO approach in design of box
culverts.

NOTE:
Earth pressures or loads on culverts
according to AASHO may be computed
ordinarily as the weight of earth dir-
ectly above the structure. This weight
is then reduced to 70 percent of its
actual weight which has the effect of
increasing dead load stresses by 40 per-
cent. The large AASHO increase in stresses
may be reasonable for agencies that liber-
ally size structural members but for
district projects where concrete, for
example, is dimensioned in thickness to
the nearest 1/4-inch after detailed
structural analysis, the large reduction
is considered excessive. Additionally
the AASHO load reduction seems somewhat
incongruent with design by the ultimate
strength method.

Vertical earth loads for district projects shall
be calculated by Marston's formula for loads
on buried conduits.

For Marston's formula the following symbols
are used:

Structures

F   = Height of fill measured from the top of conduit, in feet.

W   = Load per foot of length of conduit, in pounds per foot.

C_c = Load coefficient for positive projection condition, abstract number, based on a ratio of F over B_c

C_d = Load coefficient for the trench condition, abstract number, based on ratio of F over B_d.

C_n = Load coefficient for the negative projecting condition and imperfect trench, abstract number, based on ratio of F over B_c or F over B_d.

w   = The design unit weight of the fill material, in pounds per cubic foot.

B_c = Overall width of the conduit, in feet.

B_d = The width of the trench, measured at the top of the conduit, in feet.

Box conduits: Assume $B_d = B_c + 3.00'$
Pipes (except state
        highways) $B_d = B_c + 1.67'$
Pipes in state
highways $B_d = B_c + 2.00'$

B'_d = $B_d - 0.67$, for B_c - 33 inches or less.

    = $B_d - 1.00$, for B_c greater than 33 inches.

p   = The positive projection ratio. In the case of positive projecting conduits, the projection ratio is equal to the vertical distance between the top of the conduit and the natural ground surface adjacent thereto divided by the overall width of the conduit.

p'   = Projection ratio for negative projection conditions. It is the vertical distance between the top of the conduit and the natural ground surface adjacent thereto divided by the width of the trench.

Structures

r_{sd}= The settlement ratio. For ordinary soil foundations use:

+0.7 for positive projecting conduits

-0.5 for negative projecting conduits.

K = The ratio of active horizontal pressure at any point in the fill to the vertical pressure which causes the active horizontal pressure, abstract number.

u = The "Coefficient of Internal Friction", abstract number.

u' = The "Coefficient of Sliding Friction", abstract number.

Ku = 0.150 shall be used for ordinary conditions

= 0.110 for saturated clay

= 0.130 for clay

= 0.150 for saturated top soil

= 0.165 for sand and gravel

= 0.193 for granular materials without cohesion

Marston's formula for the various conditions are as follows.

1. Trench - - - - - - - - - - - $W = C_d \cdot w \cdot B_d^2$

A relatively narrow trench. (Usually less than 1 1/2 times conduit width).

2. Negative Projection - - - $W = C_n \cdot w \cdot B_d \cdot B'_d$

A relatively narrow trench of a depth greater than conduit height over which, after backfill, additional embankment will be placed.

3. Positive Projection - - - - $W = C_c \cdot w \cdot B_c^2$

A condition where the top of conduit projects above the surface of natural ground which is subsequently covered with an embankment.

Positive projection should also be assumed where top of conduit is at or even slightly below existing ground and a relatively high embankment will be placed over the area.

4. Wide Trench - - - - - - $W = C_d \cdot w \cdot B_d^2$ and

$$W = C_c \cdot w \cdot B_c^2$$

Where trench width is between 1 1/2 and 3 times conduit width both trench and positive projection conditions ($r_{sd} = 1.0$, $p = 1.0$) should be checked. Trench condition should be used until trench loads equal those computed by the projection formula. (Loads cannot exceed those indicated by the projection formula).

5. Jacked Conduit - - - - - $W = C_d \cdot w \cdot B_c^2$ and

$$W = C_d \cdot w \cdot B_d^2 - 2C_d \cdot B_d \cdot C$$

For cover less than 15 feet use the trench condition except assume trench width equal to conduit width. For cover more than 15 feet the reduced formula should be used where C is the cohesion of the overburden soil. Use C per following:

Materials	C
Sand, loose dry	0
Clay, very soft	40
Sand, silty	100
Top soil, saturated	100
Clay, medium	250
Sand, dense	300
Clay, hard	1000

6. Imperfect Ditch

This method of load reduction has become
unacceptable to most progressive local
agencies and is therefore not outlined
herein.

The "C" coefficients for use with Marston's
formula may be selected from the curves
shown on pages ST-1, ST-2, and ST-3.

(b) Horizontal Earth

Active horizontal earth pressures for buried
conduits except reinforced concrete pipe
ordinarily shall be assumed to be 36 psf
equivalent fluid pressure. However, in cases
where substantially higher lateral pressures
may occur (such as in expansive soil areas)
the higher pressures should be used and some
structures, box conduits for example, must be
designed against the eventuality of both the
36 pound and higher loading.

Horizontal earth loads as such are not applied
in the design of reinforced concrete pipe ex-
cept for "Design Pipe" under projection condi-
tion.

(c) Water

By reason of infrequency of occurrence, internal
water pressure loading usually need not be in-
cluded in the design of buried conduits. In
the rare event of quite highly pressurized faci-
lities or extremely high-walled box conduits,
inclusion of internal water pressure loading
may be required. In this situation, however,
an encroachment of 25 percent upon structural
safety factors is acceptable.

In areas known to have a high groundwater table

Structures

or areas known to be poorly draining, structural
loading conditions should include one case where
external water is applied horizontally along
with horizontal earth.

(4) <u>Buried Conduits - Live Loads</u>

(a) Vertical Highway Loads

Buried conduits with earth cover of 10 feet
or less shall be designed for one HS20-44
truck per lane. Buried conduits having cover
greater than 10 feet usually need not include
any live load in the design.

Wheel loads on pipes are assumed to act as
uniform loads spread in accordance with the
following equations:

Transverse (with reference to the truck)
spread of wheel load = 1.2 + 1.6 F

Longitudinal (with reference to the truck)
spread of wheel load = 1.5 + 1.5 F

Where F is depth of fill over the
pipe in feet.

Unit pressures for wheel loads on pipes (including
impact where applicable) are shown in the table
on page 45.

While wheel loads on pipes, regardless of cover,
are assumed to act uniformly, under shallow
cover these loads on box conduits are assumed
to be <u>c</u>oncentrated on the top slab and distri-
bution must be determined in each case.

For box conduits with earth cover 2'-11" or
less, wheel loads shall be distributed on the
top slab in accordance with AASHO specifica-
tions 1.3.2 which assumes a distribution in
accordance with:

Structures

$$E = 4 + 0.06S$$

where   E = Width of slab in feet
over which a wheel is
distributed (beam width)

and   S = center to center span.

Impact shall be added in accordance with
AASHO specification 1.2.12 (c) which lists
the following percentages.

Cover 0' to 1'-0"	30%
1'-1" to 2'-0"	20%
2'-1" to 2'-11"	10%

A single 16 kip wheel load plus impact is
considered sufficient for span of 12 feet
or less having cover of 3 feet or less.

Where the cover is over 2'-11" but not
greater than 10 feet the wheel loads shall be
distributed through the fill to the top slab
of box conduits (as for pipe) in accordance
with the following equations.

Transverse (with reference to the truck)
spread of wheel load = 1.2 + 1.6 F

Longitudinal (with reference to the truck)
spread of wheel load = 1.5 + 1.5 F

Where F = depth of fill over box
in feet.

The following tabulated live load pressures apply to pipes and to the top slab of box conduits where listed.

Conduit Type	Cover "F" (Feet)	Wheel Load (Kips)	Pressure (psf)
Pipe	1	20.8	2480
only	2	19.2	970
- - - - -	- - - - -	- - - - -	- - - - -
Pipe	3	16.0	444
	4	16.0	314
	5	16.0	234
and	6	16.0	182
	7	16.0	145
	8	16.0	119
Box	9	16.0	102
	10	16.0	90

These values include the effect of overlapping wheel loads.

Wheel loads shall be distributed to the bottom slab of box conduits as follows when fill over top of conduit is 10 feet or less:

Transverse (with reference to the truck) spread of wheel load = 1.2 + 1.6 F + H for traffic parrellel to main reinforcing
= 1.2 + 1.6 F for traffic perpendicular to main reinforcing

Structures

Longitudinal (with reference to the truck)
spread of wheel load $= 1.5 + 1.5\, F + H$ for
traffic perpendicular
to main reinforcing
$= 1.5 + 5\, F$ for traffic
parallel to main
reinforcing

where F = depth of fill over box
in feet and

H = height of box from invert
at base of wall to soffit.

The effect of overlapping wheel loads shall
be taken into account.

Graphs showing H20 truck loads on invert slabs
are shown on pages ST-4 to ST-9. For cover
0 to 2'-11" the graphs include both conditions
of traffic parallel and perpendicular to main
reinforcing in the conduit. The more severe
of the two should be used except where traffic
flow is only possible in one direction, in
which case the applicable condition shall be
used.

(b) Horizontal Highway Loads

Horizontal loads due to trucks should be in-
cluded in design of buried conduits, except
reinforced concrete pipe, for all earth covers
of 10 feet or less. Graphs of lateral HS20-44
truck loads are shown on page ST-10.

(c) Vertical Railroad Loads

Conduits to be placed under railroads shall be
designed in accordance with the requirements
of the particular railroad.

The Cooper loadings for the three railroads within district boundaries are:

        Atchison, Topeka and Santa Fe - E72
        Southern Pacific - - - - - - - E72
        Union Pacific - - - - - - - - - E75

Railroad loads on pipe and box conduits are assumed to be uniform. Tables showing railroad loads on pipe and for the top slab of box conduits are shown on page ST-11. Curves for loads on the invert of box conduits are shown on pages ST-12 and ST-13. These curves include impact in accordance with AREA specifications. For the A.T. & S.F. railroad the so-called "diesel impact", which is a maximum of 40 percent at minimum cover and reduces 5 percent for each additional foot of cover, may be used.

(d) Horizontal Railroad Loads

Side railroad loads on box conduits are assumed to be uniform. Curves for these loads are shown on pages ST-14 and ST-15.

(5) Open Channels - Dead and Live Loads

(a) Vertical Loads

Vertical loads for design of channels are normally limited to the loads resulting from the weight of structure plus heel loads and the pressures induced by the application of lateral loads. Unless otherwise indicated (by the soils report for example) no additional loads need be applied.

Vertical soil pressures on rigid frame "U" channels shall be computed assuming the invert slab as a beam on an elastic foundation in

accordance with theories outlined in:

Hetenyi, M., Beams on Elastic Foundations,
University of Michigan Press, Ann Arbor, 1946.

On the basis of an assumed external load of
62.5 psf equivalent fluid pressure (Case I) and
a differential hydrostatic pressure of 40 psf
(Case II) moments in invert slabs using Hetenyi's
theories have been graphed for various wall
heights, and channel widths. Copies of these
graphs are included on pages ST-15 through ST-24.

In the event design lateral load is different
from the listed Case I and Case II loads, the
moments shown on the mentioned pages do not apply.
In most cases, however, an acceptable approxi-
mation of actual invert moments can be determined
by increasing (or decreasing) the moments shown
on the graphs by a direct ratio of the actual
versus graphed moments at the base of the wall.

Vertical soil pressure on "L" wall channels
shall be fixed in accordance with standard
retaining wall design and the soils report.

(b) Horizontal Loads

Channel walls 12 feet or less in height with
reasonable level backfill should be designed
for the following loads.

(1) Where no possibility exists for truck
surcharge and where the soils report
indicates no greater lateral pressure,
a load of 62.5 psf equivalent fluid
pressure (E.F.P.) should be applied to
the earth face with the channel empty.

Curves giving moments and shears at the
base of walls for this loading are shown
on page ST-32.

Structures

(2) Where walls support maintenance roads or public thoroughfares and the soils report indicates no unusual conditions a load of 36 psf E.F.P. plus a H20 truck 2 feet from the wall should be applied to the earth face with the channel empty.

Curves giving moments and shears at the base of walls for this loading are shown on page ST-31.

The horizontal truck load is based on criteria outlined in the building code of the City of Los Angeles for surcharge on retaining walls modified to assume that for a concentrated load a wall length equal to wall height acts as a unit.

Wall heights greater than 12 feet should be designed only after careful study of condition and of the economics of the various alternatives. Ordinarily a loading of truck plus provisions for insuring no greater earth load than the 36 psf E.F.P. will prove to be the most economical.

In the event walls are sloped outward to form a trapezoidal shape the horizontal component of pressure and resulting moments are reduced. Curves showing the ratio of moment for sloping walls to the moment for vertical walls are shown on page ST-30.

Where walls are subject to a sloping surcharge the unit equivalent fluid pressure applicable to level backfill should be increased in accordance with the formula:

$$w' = w \left[2 - \left(\frac{H-H'}{H} \right)^2 - \frac{3}{2} \frac{X}{H} \right]$$

where

w' = design E.F.P. (not less than w or more than 2w)
w = unit E.F.P. for level backfill
H = wall height
H' = vertical height of slope
x = horizontal distance back of wall to beginning of slope

In the design of vertical wall open channels an internal pressure of 40 psf E.F.P. should be applied to the full height of walls with no supporting backfill on the outside. This loading guards against failure should the channel be loading from the inside before the backfill is placed and protects the wall against some contractor's methods of form removal.

D. REINFORCED CONCRETE PIPE

(1) D-Load Pipe

(a) Design Method

Pipe 108-inches or less in diameter shall be designed for D-load designation where

$$D\text{-load} = \frac{(\text{Total Vert. Load per Lin.Ft.})(\text{Safety Factor})}{(\text{Internal Diameter (Ft.)})(\text{Load Factor})}$$

Total vertical load should be determined in accordance with Section C - LOADS ON STRUCTURES. For covers of 10 feet or less earth load should be based on positive projection condition assuming projection ratio equal to one. For covers greater than 10 feet applicable condition should be used.

The safety factor is 1.25 for district projects.

The load factor for ordinary bedding is 1.80. Load factors for various bedding conditions are shown on page ST-33. These may also be used for projection condition except that in specialized cases it may be necessary to determine load factor in accordance with the method outlined in:

Spangler, M. G., Soil Engineering, International Textbook Company, Scranton, 1951.

(b) Rounding Off

Calculated D-load values should be rounded off to the next higher even value as follows:

36-inch and under	Nearest 250
39 to 60-inch	" 100
63 to 108-inch	" 50

(c) Minimum D-Loads

Minimum strength of pipe under the various jurisdictions shall be:

District right-of-way and local streets	800-D
State highways	1350-D
Railroads	2000-D
Except A.T. & S.F.	3000-D

Structures

(d) Maximum Economical Strength

Maximum D-loads for pipe, except for very
specialized conditions, shall be:

Diameter	D-load	Diameter	D-load
12	6000	42-48	3500
15	5000	51-57	3250
18	4750	60-63	3000
21	4500	66-72	2750
24	4250	75-78	2500
27-30	4000	81-87	2250
33-39	3750	90-108	2000

In the event calculated D-loads based on
ordinary bedding exceeds the listed strength
an improved bedding normally should be used.

(e) Jacked Pipe

Pipe to be jacked should be designed with a
load factor of 1.80 and for superimposed
loads only. The loads that may be placed
on the pipe as a result of jacking operations
shall be the responsibility of the contractor
(project specifications must so state).

(f) Steel Clearances

Steel clearances for D-load pipe are specified
in the district's basic specification (ASTM-C76)
and call out is not ordinarily required, however,
for severe conditions additional protection is
needed.

The following increases in cover and/or minimum
clear cover should be specified.

Exposure to salt water	1/2-inch -- 1-1/2 inches clear -- both faces.
Harmful ground water	1/2-inch -- 1-1/2 inches clear -- both faces.
Velocity above 20 fps	1/2-inch -- 1-1/2 inches clear -- inside face at flowline.
Rock laden debris	1-inch -- 2 inches clear -- inside face at flow line.

These increases are cumulative and the increments should be used to establish total clearance where more than one condition exists.

(g) Minimum Earth Cover

Earth cover, including pavement, must be at least one foot unless the pipe is adequately protected by concrete backfill.

(h) D-load Table

D-loads for ordinary bedding, 110 pcf earth plus highway live loading are tabled on page ST-29.

(2) Specially Designed Pipe

(a) Design Method

Pipe over 108 inches in diameter shall be designed for combined bending and axial forces.

Structures

The two critical points for design are the
invert and the side wall at spring line. The
loads applied for design in trench condition
are the total vertical loads including dead
load of structure and earth and live loads.
In projection conditions a horizontal earth
load of 36 psf equivalent fluid pressure is
also included. No internal water pressure is
assumed except under high pressure conditions.

(b) Conditions of Support

For ordinary bedding, load factor 1.80, vertical
loads are assumed over 180° of top and 90° of
bottom.

For concrete bedding, the bottom support may
be assumed equal to the degrees of encasement
but not more than 120°.

(c) Moments and Thrusts

For the assumed loading conditions moments
and thrusts may be determined from coefficients
calculated from information presented in
Engineering News-Record, page 768, November 10,
1921. These coefficients are shown on page ST-34.
Using these coefficients:

Design Moment (M) = $C_m.W.R$ and
Design Axial Load (N) = $C_n.W$ where

C = Applicable coefficient (M or N in table)
W = Total vertical live and dead load in pounds
R = Mean radius (not diameter) of ring in feet

(d) Concrete thickness

Concrete thickness shall be based on a minimum
steel cover of 1-1/2 inches clear (with upward

increases if necessary as specified for D-load
pipe under adverse conditions) and the required
beam depth corresponding with steel area used.
However, pipe manufacturers should be consulted
for determination of standard, or at least
acceptable, thicknesses and where possible
these should be used.

(e) Steel Patterns

Three alternate methods of reinforcement should
normally be included. These are:

1) An inner circular cage plus an outer
    circular cage.

2) An inner circular cage plus an elliptical
    cage.

3) A single elliptical cage.

By reason of the maximum possible reinforce-
ment being approximately 3 square inches per
foot per face, elliptical cages must sometimes
be omitted. For pipe to be jacked two circular
cages must be specified.

E. CAST-IN-PLACE PIPE

(1) General

A firm tight ground capable of standing unsupported
from the bottom of the trench to the top of the pipe
without sloughing furnishes the best natural foundation
for cast-in-place pipe. While the pipe may be used in
other soils, economics are not nearly as favorable.
Prior to specifying cast-in-place pipe the designer
should have the suitability of the foundation verified
by an engineer specializing in soils and foundation
engineering.

By reason of its proprietory nature, cast-in-place
pipe should not be specified without inclusion of
an alternate section.

(2) Method of Design

The concrete shall be dimensioned on the basis of
analysis of beams subject to combined bending and
axial forces. Stresses in the concrete shall be
calculated by means of the coefficients for elastic
rings (chart ST-34) from Engineering News-Record,
Page 768, November 10, 1921. Vertical earth loads
shall be determined in accordance with Marston's
theory and may be assumed spread over the top 180
degrees of the pipe. Bottom support may be assumed
uniform over 180 degrees. Live loads will be identi-
cal with that for reinforced concrete pipe.

Horizontal support should be neglected in any area,
such as streets, where it is impossible to control
future excavations.

In right-of-way fully controlled by the district
lateral support of 36 psf equivalent fluid pressure
may be assumed.

After determination of loads, moments and thrusts and
shears can be conveniently computed for the applicable
combinations of loading using the coefficients shown
on mentioned Chart ST-34.

Neither the ultimate strength nor working stress
methods of design are directly applicable to design
of non-reinforced concrete beams, especially beams
with some degree of fixity. However, the working
stress assumption that stress varies directly with
the distance from the neutral axis (center of gravity)
is the more logical approach and should be used for
design.

(3) Allowed Stresses

The allowable unit compressive stress in the extreme

Structures

fiber in flexure shall be 1350 psi. The allowable
unit tensile stress in the extreme fiber in flexure
shall be 225 psi. The allowed tensile stress is based
on an assumed modulus of rupture of at least 500 psi,
therefore, project specification must include this
requirement.

F. REINFORCED CONCRETE BOX CONDUITS

(1) General

Reinforced concrete box conduits, except for small
sections symmetrically loaded and having maximum
dimensions of 7 feet or less, shall be designed as
rigid frames. The most economical height to width
ratio should be selected. This ratio is generally
assumed to be considerable greater than one although
when all factors are analyzed, including utility
relocations, considerable variation may be expected.
In cases where a box conduit is an alternate to pipe,
however, the height of the box conduit should, if
possible, be fixed equal to the internal diameter of
the pipe.

For hydraulic and maintenance reasons single cell
conduits are much preferred over multiple barrel
conduits and single conduits with spans up to 14
feet are usually less expensive to construct, except
that for railroad loadings (which are usually short
sections) the break point occurs at shorter spans.
Therefore single barrel conduits should be used up
to 14 feet in width and with supercritical velocity
or debris problems an even larger single section may
be acceptable.

(2) Loading

Combinations of dead and live loads producing maxi-
mum shears and moments are shown on pages ST-35 to
ST-37.

(3)   Method of Design

Rigid frame shear determinations and moment distri-
bution shall be based on centerline spans.

Design moment at supports shall be at the face of the
support. The reduction in moment from support center-
line to face shall be based on an assumed linear shear
variation between these same points.

Design maximum shear shall be a distance "d" from the
face of support where "d" is the effective depth of
member and the small fillet or haunch is ignored.

Walls shall be designed for combined bending and axial
thrust but axial thrust shall not be included in design
of top and bottom slabs.

The effect of large haunches or tapered members should
be included in the design. Small fillets of the type
normally included in box conduits should be ignored in
all calculations of stiffness, unit shear, bond and
steel area.

Where moderate sized channels cross streets or other
transportation media on a skew and a box culvert is
used as the bridge, the ends of the culvert should
be squared off normal to the channel. The parapets,
for appearance reasons, should be placed parallel to
traffic. This leaves a triangle of conduit beyond
the parapet but greatly simplifies the very prevalent
later extension of covered section.

Where very large channels cross streets with large
skew angles it may be advantageous to design by skew
analysis and place main reinforcing and the ends of
the culvert parallel to traffic. The method of
analysis shall be as presented in Paper 2474, ASCE
Transactions, Vol. 116, 1951, titled "Practical Design
of Solid Reinforced Concrete Skew Structures" by
Bernard L. Weiner.

Under the method of skew analysis a sample section
is taken as for a perpendicular section and basic
moments, thrusts and shears are determined for this
right angle section. Design moments, thrusts and

shears are obtained by multiplying these basic elements by the secant squared of the modified skew angle. The modified skew angle is equal to the skew angle for slab analysis but is equal to zero for vertical wall analysis.

(4) <u>Minimum Thicknesses</u>

Minimum thicknesses of small box sections designed as simple beams shall be:

       Top Slab . . . . . 6.0 inches
    Bottom Slab . . . . . 7.0 inches
          Walls . . . . . 6.0 inches

Minimum thicknesses for rigid frame box sections shall be:

       Top Slab . . . . . 7.0 inches
    Bottom Slab . . . . . 7.5 inches
          Walls . . . . . 8.0 inches

(5) <u>Steel Clearances</u>

Steel clearance call outs for district projects is <u>clear cover</u>. Basic clearances with bars through #8 is 1.5 inches for formed members and 2.0 inches for concrete to be cast upon earth. (AREA specifications required 3.0 inches clear cover for railroad culverts). By reason of erosion protection, clearance for top steel in invert slabs (railroads excepted), shall be as follows:

Design velocity less than 12 fps . . . . . 1.5 inches
Design velocity 12 to 20 fps . . . . 2.0 inches
Design velocity 20 to 35 fps . . . . 2.5 inches
Design velocity above 35 fps . . . . 3.0 inches

All listed steel clearances, except railroads, should be increased 0.5 inch if subject to the action of sea water, harmful groundwater or other adverse condition.

(6) Longitudinal Reinforcement

Longitudinal reinforcement shall consist of #4 bars
at 18-inch centers in each reinforced face except
for exposed slabs or short lengths where appreciable
temperature variations may be expected. Temperature
reinforcement, if needed, should be equal to 0.002
of the concrete area.

(7) Distribution Steel

When design cover is less than 3 feet distribution
steel shall be included. The amount including normal
longitudinal reinforcement shall be the following
percentage of the transverse reinforcement required
for positive moment in the top slab.

$$\text{Percentage} = \frac{100}{\sqrt{S}} \qquad \text{maximum } 50\%$$

where S equals centerline span in feet

(8) Fillets and Vees

Fillets shall be included at the top corners of all
rigid frame box conduits. They should be either
4" x 4" or 6" x 6" at the contractor's option.

Vees to concentrate low flows should be placed in
the invert slab. Vee depth should be fixed on the
basis of a cross-slope of 3/4-inch per foot without
regard to the fractions of inches that may result.
In multi-barrel sections, the given cross-slope
should be projected to the low-flow concentration
point nearest the structure centerline. The practice
of placing vees in each barrel of multi-barrelled
structures should be discontinued

The use of large vees in box conduits having one, three, or other odd numbers of cells introduces the possibility for a reversal of stresses or other adverse effects. With the 3/4-inch per foot cross-slope however, additional structural analysis is not needed for district projects.

(9) Steel Patterns

By reason of the tolerance capabilities of ordinary fabricating shops steel patterns should generally avoid more than one bend per bar. This is especially true where dimensions are critical such as in "U" bars and more than one bend will not be accepted in these cases.

Typical steel patterns and other details for single and double box conduits are shown on sheets ST-38 and ST-39, respectively.

G. RECTANGULAR OPEN CHANNELS

(1) Method of Design

Channels should be designed as "U" rigid frames except that extremely wide channels may be designed as "L" walls with a connecting floater slab.

Rigid frame "U" sections with sizable differential lateral loadings shall be checked for stability, soil reaction and sliding. The "L" wall sections shall be checked for stability, soil reaction and sliding. Footings for "L" sections shall be sufficiently long that the resultant of all forces falls within the middle third. The center invert slab shall also be checked for buckling forces transmitted by the adjoining retaining walls. The factor of safety against sliding shall be at least 1.5.

(2)   Concrete Thicknesses

Side walls for rectangular channels shall have the
following minimum thicknesses:

1.   Low walls with one curtain of steel -- 6 inches

2.   Low walls with two curtains of steel
     or if fencing is to be placed on top - 8 inches

3.   Walls 8 feet to 10 feet in height - - 8 inches

4.   Walls more than 10 feet to 13 feet - -10 inches

5.   Walls greater than 13 feet in height -12 inches

The earth face of walls shall be battered from the
required thickness at the base, if greater than the
minimum, to the specified minimum thickness at the
top.

Floor slabs shall have a thickness at the inside
face of the wall equal to (or greater than if necessary)
the wall thickness at the base. Generally floor slabs
shall have a minimum thickness of 8 inches and shall
include a 6-inch heel.

(3)   Steel Clearances

Steel clearances for rectangular channels shall be
identical with that specified for box conduits.

(4)   Longitudinal Reinforcement

Longitudinal steel shall provide a ratio of reinforce-
ment area to gross concrete area of 0.0020, but in no
case shall such reinforcing bars be placed more than
18 inches on centers.

Longitudinal reinforcement shall not be continuous through the transverse construction joints.

(5) Transverse Floor Slope

Vees to concentrate low flows should be placed in the invert slab as follows:

(a) Channels having a base width of 34 feet or less shall have a cross slope of 3/4-inch per foot.

(b) Channels with base width more than 34 feet should have a trapezoidal low flow channel with a base width of 6-feet and a depth of one foot and 4 to 1 side slopes. That portion outside the low flow channel should slope at 2 percent toward the channel.

H. BRIDGES

Except for railroad bridges which must be designed in accordance with AREA specifications, and the requirements of the individual railway company, bridges for district projects shall be designed with the HS20-44 truck loading using the Bureau of Public Road modification. The design of bridges shall be in accordance with the

Manual of Bridge Design Practice
Third Edition, 1971

published by the Division of Highways of the State of California. This is a very complete treatise on bridges, even furnishing assistance in the design of railroad structures, therefore, no special criteria are included herein.

Horizontal axis (top): 1   1.5   2   3   4   5

Vertical axis (left): 40 30 25 20 15 10 9 8 7 6 5 4 3 2 1

VAL. OF $\frac{F}{B_d}$

Curve labels: A   E   (upper), MATCH, A   E   (right), 2

Vertical axis (right): 1.5 1 .9 .8 .7 .6 .5 .4 .3 .25 .2 .15 .1

VAL. OF $\frac{F}{B_d}$

Horizontal axis (bottom): .1 .15 .2 .25 .3 .4 .5 .6 .7 .8 .9 1 1.5

VALUES OF COEFFICIENT — C_d
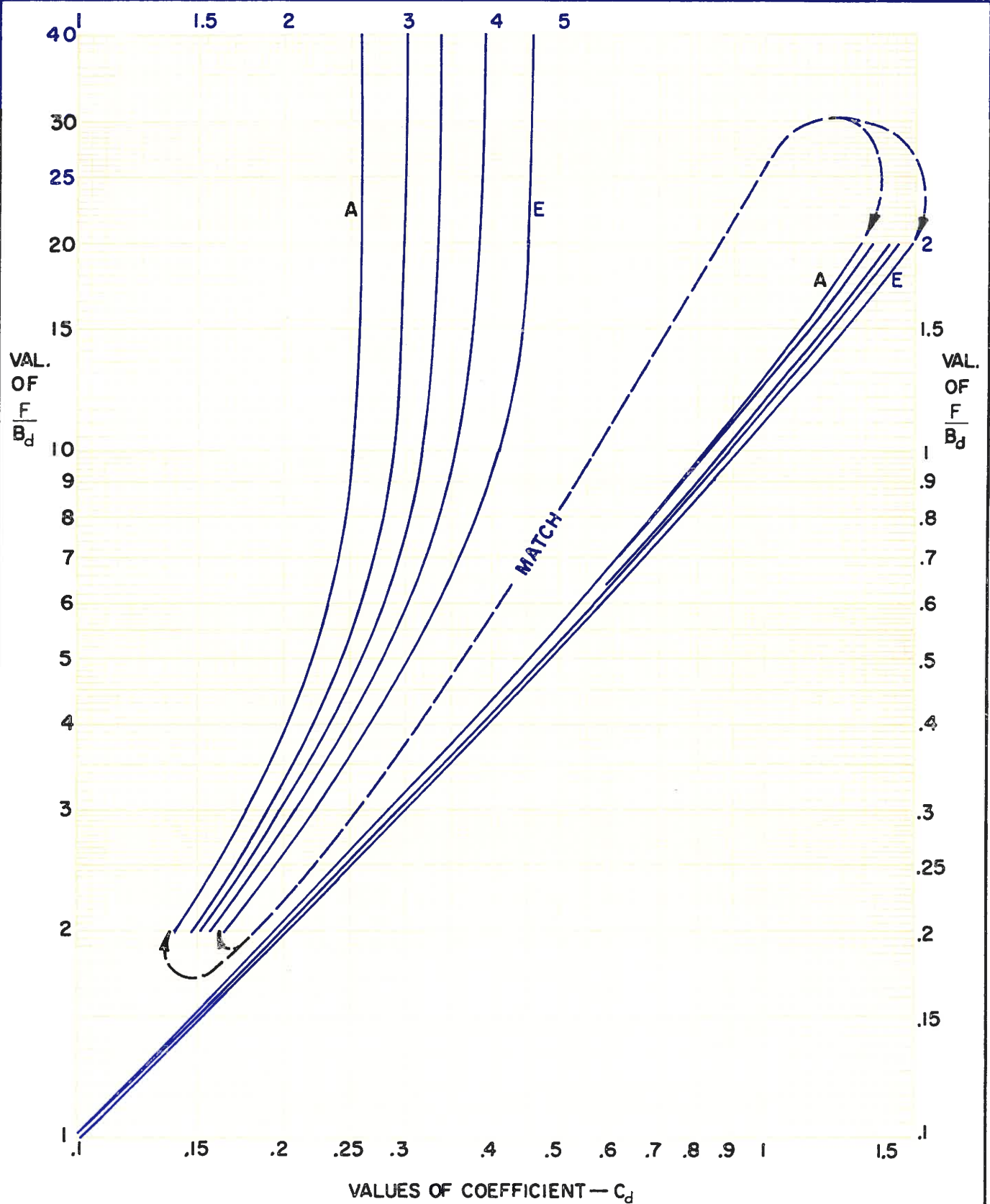
A = C_d for K_μ and $K_{\mu'}$ =.1924 for Granular Materials Without Cohesion
B = C_d for K_μ and $K_{\mu'}$ =.165 Maximum for Sand and Gravel
C = C_d for K_μ and $K_{\mu'}$ =.150 Maximum for Saturated Top Soil
D = C_d for K_μ and $K_{\mu'}$ =.130 Ordinary Maximum for Clay
E = C_d for K_μ and $K_{\mu'}$ =.110 Maximum for Saturated Clay

ORANGE COUNTY FLOOD CONTROL DISTRICT		
LOAD COEFFICIENTS FOR TRENCH CONDUITS		
1972	1 OF 1	ST-1

Graphs showing Load Coefficients for Negative Projecting Conduits. Four plots of $\frac{F}{B_d}$ or $\frac{F}{B_c}$ versus Load Coefficient C_n, each showing COMPLETE TRENCH CONDITION and INCOMPLETE TRENCH CONDITION curves for $r_{sd} = -2.0, -1.0, -0.5, -0.3, -0.1, 0$.

$p' = 0.5, K_\mu = 0.13$

$p' = 1.0, K_\mu = 0.13$

$p' = 1.5, K_\mu = 0.13$

$p' = 2.0, K_\mu = 0.13$
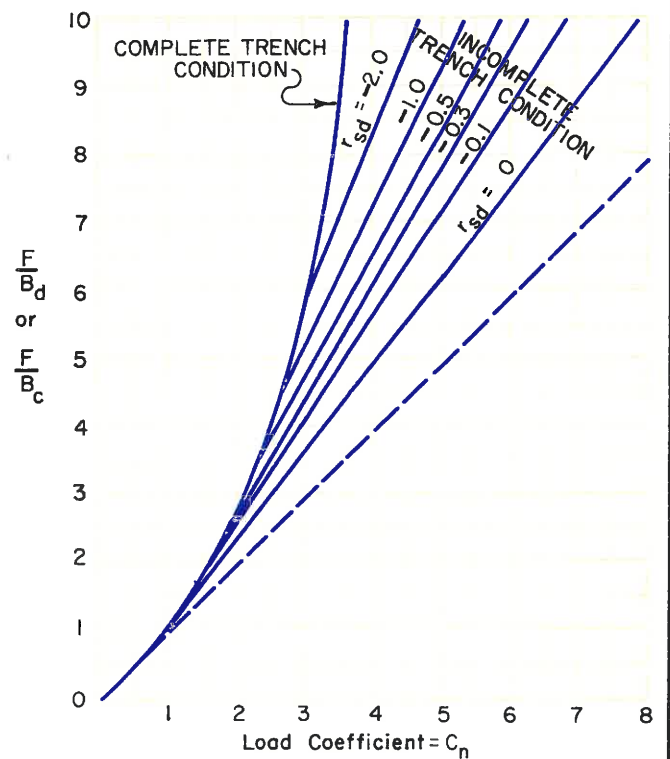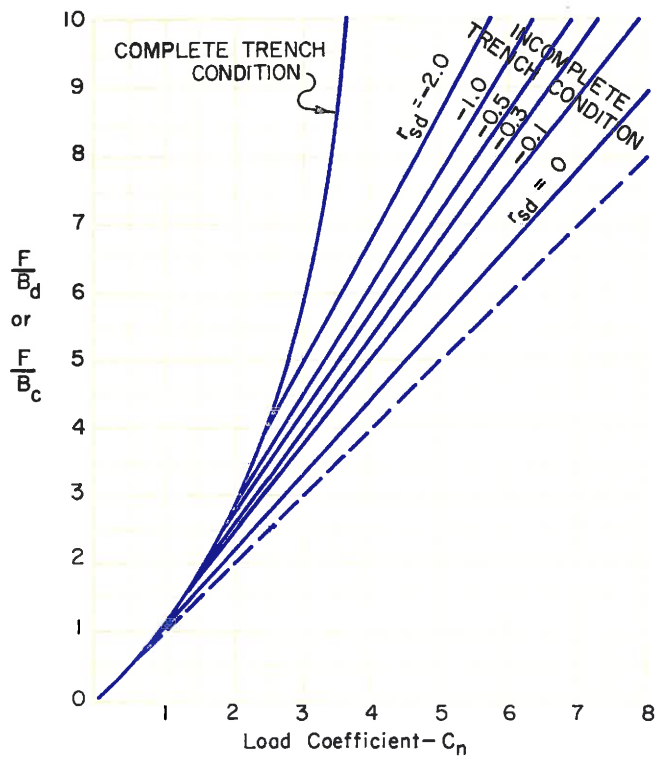
ORANGE COUNTY FLOOD CONTROL DISTRICT

LOAD COEFFICIENTS
NEGATIVE PROJECTING
CONDUITS

| 1972 | I OF I | ST-2 |

COEFFICIENT — C_c

RATIO $\dfrac{F}{B_c}$

$r_{sd}p = -2.0$

-1.0

-0.7

-0.5

-0.3

-0.1

$r_{sd}p = 0$

$+0.1$

$+0.3$

$+0.5$

$+0.7$

$+1.0$

$r_{sd}p = +2.0$

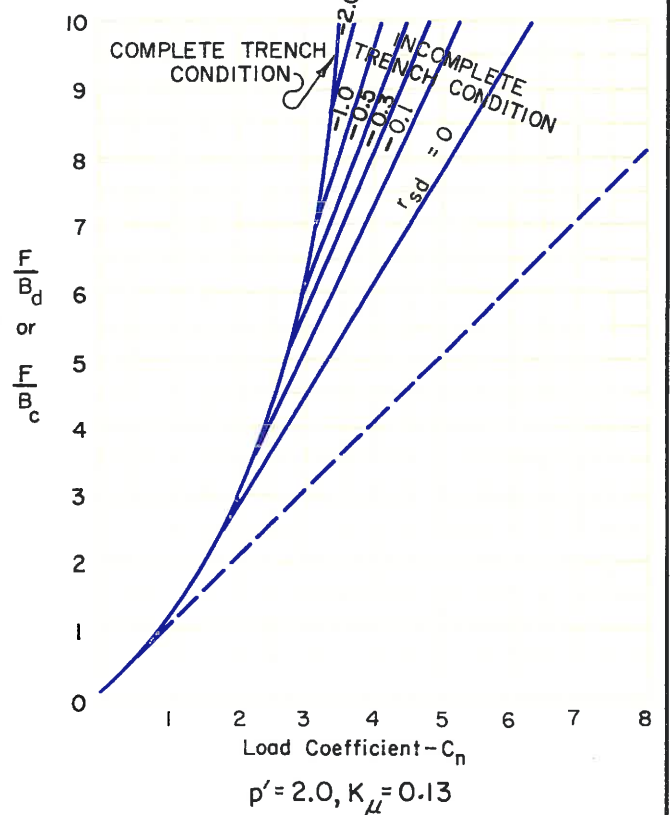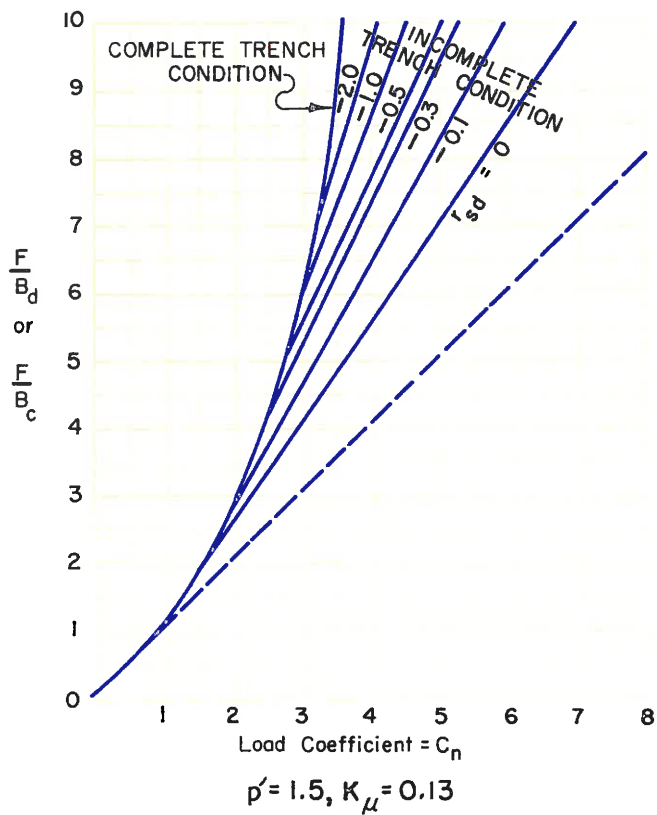$K\mu = .19$

RATIO $\dfrac{F}{B_c}$

COEFFICIENT C_c

ORANGE COUNTY FLOOD CONTROL DISTRICT

LOAD COEFFICIENTS
FOR
PROJECTING CONDUITS

| 1972 | 1 OF 1 | ST-3 |

H, HEIGHT OF BOX SECTION, (Ft.) — vertical axis, values 1 through 14

(Lbs. per Sq. Ft.) — horizontal axis, values 0 to 1100

Curve labels: F = 2'-11"   F = 2'   F = 1'   F = 0

Top diagram labels: 14'-0"   32K/LANE   H/2   1.5 + 1.5F   1.5 + 1.5F + H

Middle diagram labels: 6' TYP.   16K TYP.   F   H   W

EFFECT OF WHEEL
OVERLAP INCLUDED

MAIN REINFORCEMENT IS PERPEN-
DICULAR TO TRAFFIC.
DEPTH OF COVER IS 0' TO 2'-11".

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	1 OF 6	ST-4

H, HEIGHT OF BOX SECTION IN FEET

POUNDS PER SQUARE FOOT

F=10 F=9 F=8 F=7 F=6 F=5 F=4 F=3

14' 32 K/LANE

1/2 H 1.5 + 1.5F
1.5 + 1.5F + H

6' TYP. 16 K TYP.

F
H
W

EFFECT OF WHEEL
OVERLAP INCLUDED
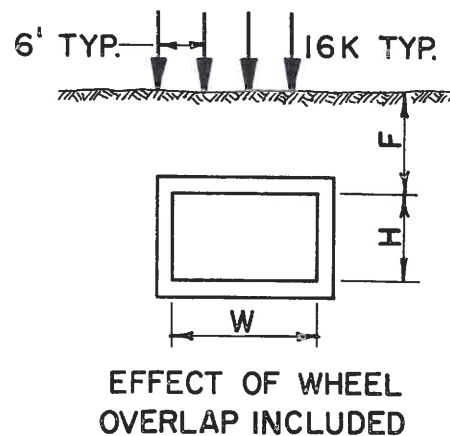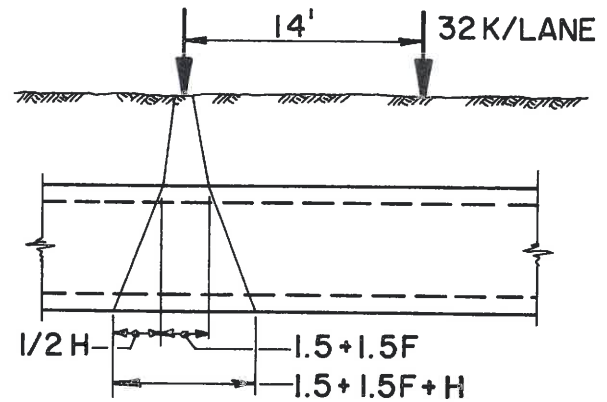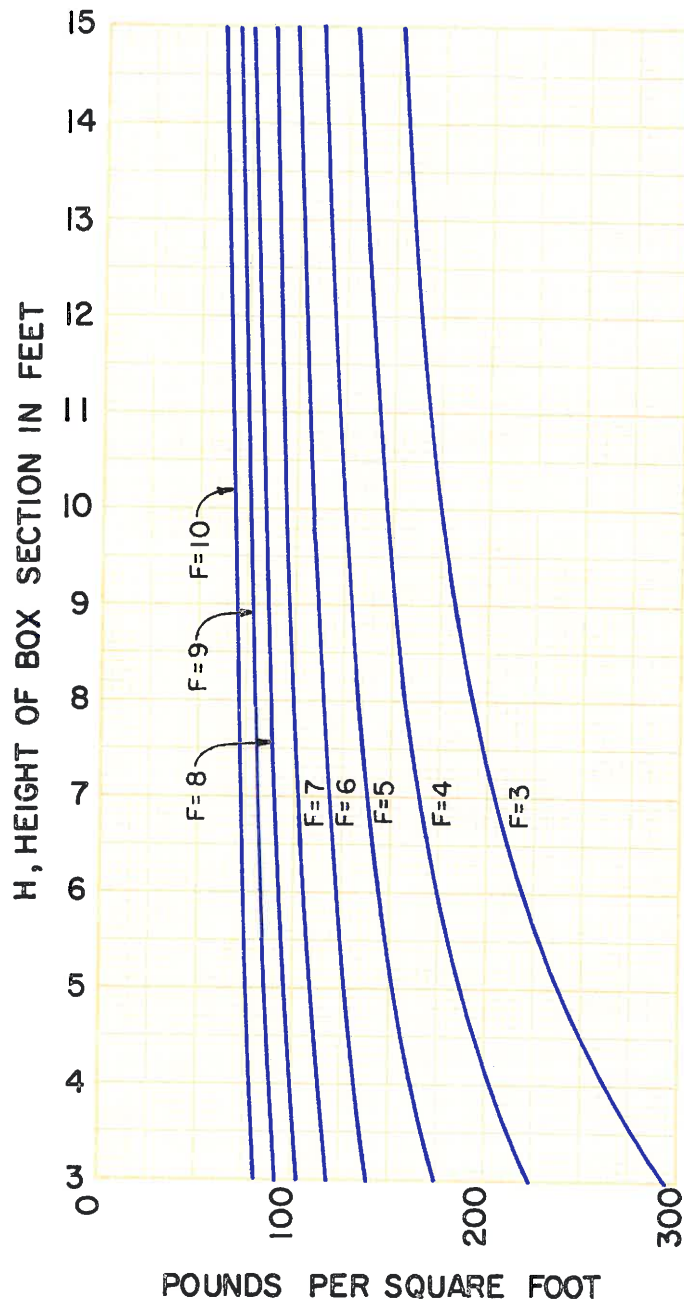
MAIN REINFORCEMENT IS PERPENDICULAR
AND/OR PARALLEL TO TRAFFIC.

DEPTH OF COVER IS 3' TO 10'.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
H-20 TRUCK LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	2 OF 6	ST-5

The chart contains the following labels and annotations:

- Y-axis: H, HEIGHT OF BOX SECTION IN FEET (values 3 to 15)
- X-axis: POUNDS PER SQUARE FOOT (values 100 to 1000)

Curve labels (left to right): W=30', W=28', W=30', W=26', W=22', W=18', W=14', W=12', W=10', W=8', W=6', W=5', W=4'

Top right diagram:
- 6' TYP.
- 16K TYP.
- 1/2 H
- 1.2
- 1.2+H

Second diagram:
- 14'-0" 32K
- H
- W
- EFFECT OF WHEEL OVERLAP INCLUDED
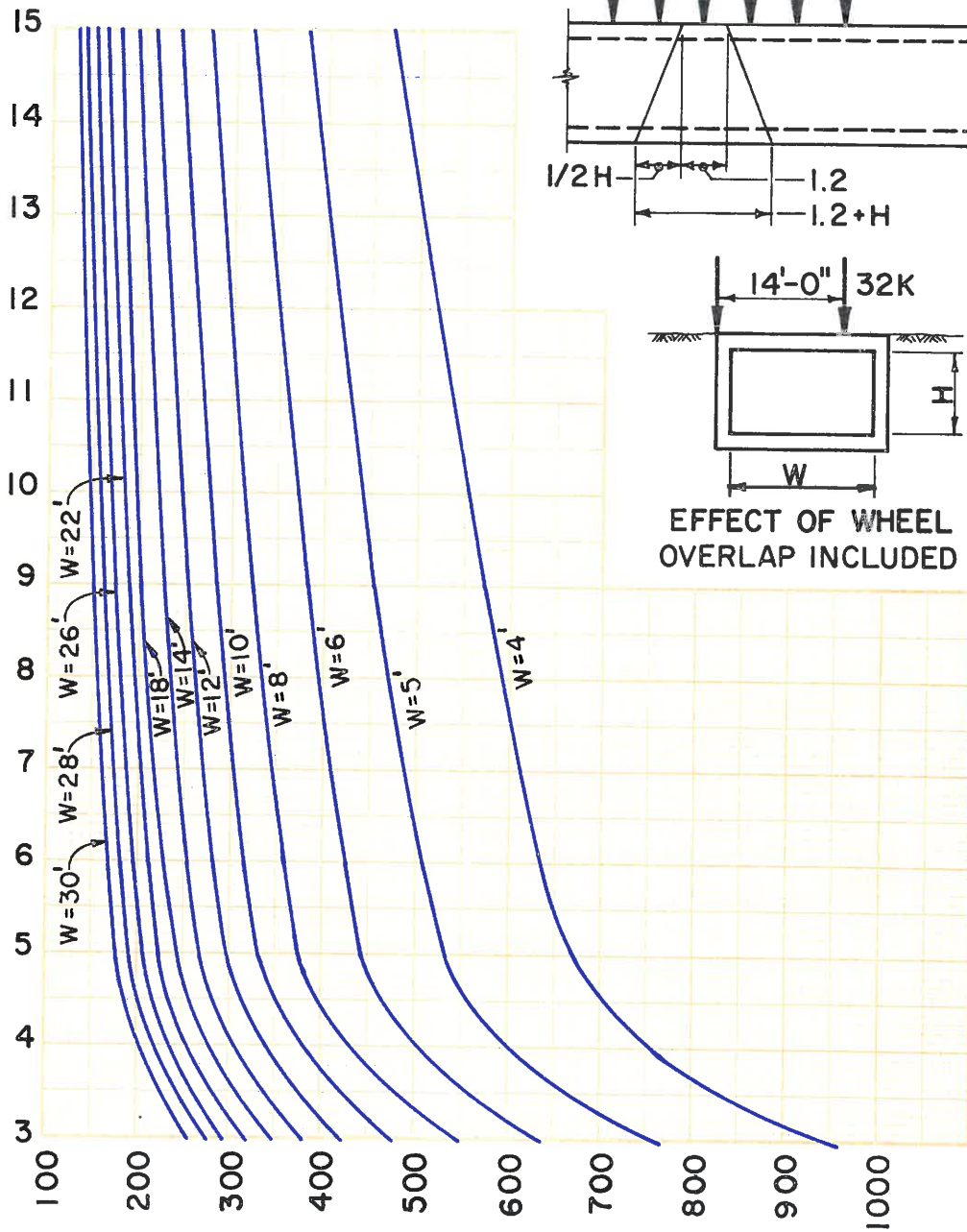
MAIN REINFORCEMENT IS PARALLEL TO TRAFFIC.

DEPTH OF COVER IS 0'.

ORANGE COUNTY FLOOD CONTROL DISTRICT

H-20 TRUCK LOADS
ON INVERT SLABS
OF BOX CONDUITS

| 1972 | 3 OF 6 | ST-6 |
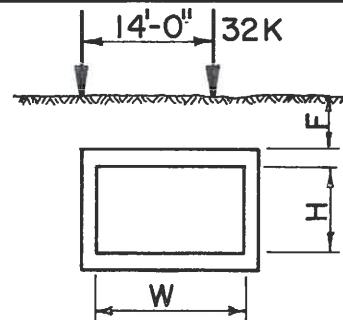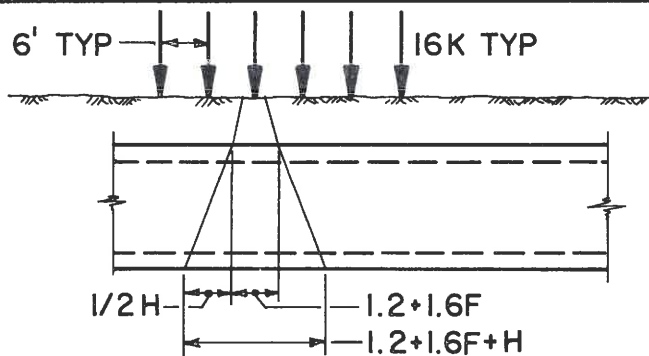
6' TYP — 16K TYP

14'-0" 32K

F

H

W

EFFECT OF WHEEL
OVERLAP INCLUDED

1/2H — 1.2+1.6F

1.2+1.6F+H

H, HEIGHT OF BOX SECTION IN FEET

W=30
W=28
W=26
W=22
W=18
W=14
W=12
W=10
W=8
W=6
W=5
W=4

POUNDS PER SQUARE FOOT

MAIN REINFORCEMENT IS PARALLEL
TO TRAFFIC.

DEPTH OF COVER IS 1'-0"

ORANGE COUNTY FLOOD CONTROL DISTRICT

H-20 TRUCK LOADS
ON INVERT SLABS
OF BOX CONDUITS

| 1972 | 4 OF 6 | ST-7 |
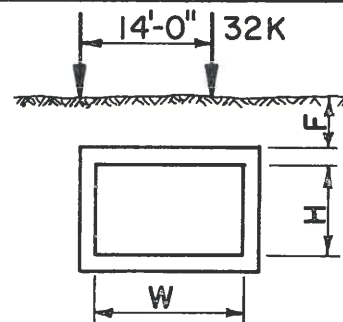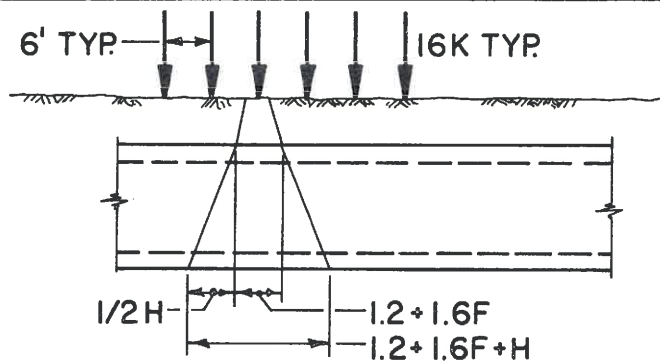
6' TYP. — 16K TYP.

14'-0" 32K

1/2 H — 1.2 ÷ 1.6F
1.2 ÷ 1.6F + H

EFFECT OF WHEEL
OVERLAP INCLUDED

H, HEIGHT OF BOX SECTION IN FEET

W=30 W=28 W=26 W=22 W=18 W=14 W=12 W=10 W=8 W=6 W=5 W=4

POUNDS PER SQUARE FOOT
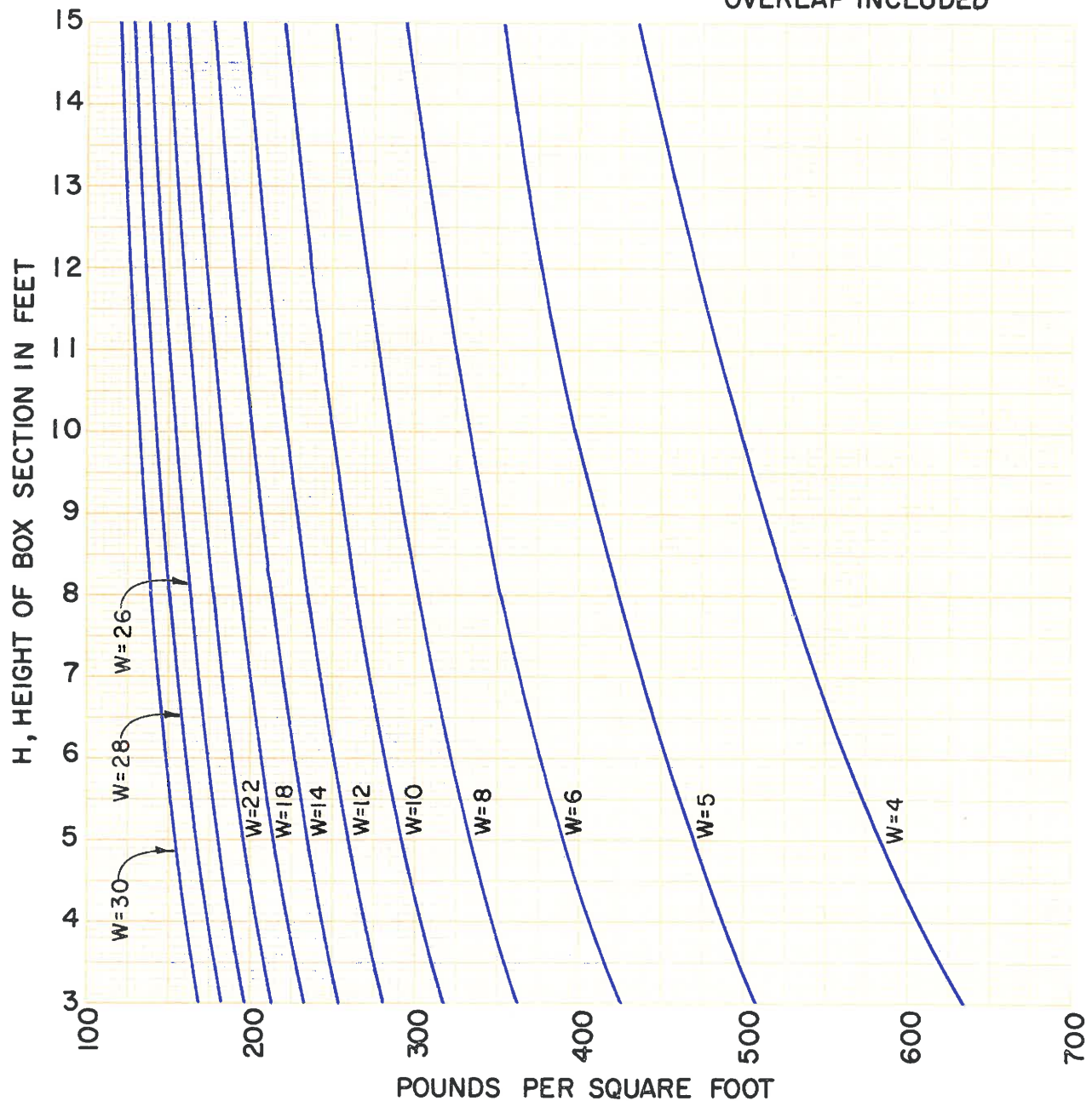
MAIN REINFORCEMENT IS PARALLEL
TO TRAFFIC.

DEPTH OF COVER IS 2'-0"

ORANGE COUNTY FLOOD CONTROL DISTRICT

H-20 TRUCK LOADS
ON INVERT SLABS
OF BOX CONDUITS

| 1972 | 5 OF 6 | ST-8 |

6' TYP — 16K TYP.

1/2 H — 1.2+1.6F
— 1.2+1.6F+H

14'-0" 32K

EFFECT OF WHEEL
OVERLAP INCLUDED

H, HEIGHT OF BOX SECTION IN FEET

W=30   W=28   W=26   W=22

W=18   W=14   W=12   W=10   W=8   W=6   W=5   W=4

POUNDS PER SQUARE FOOT
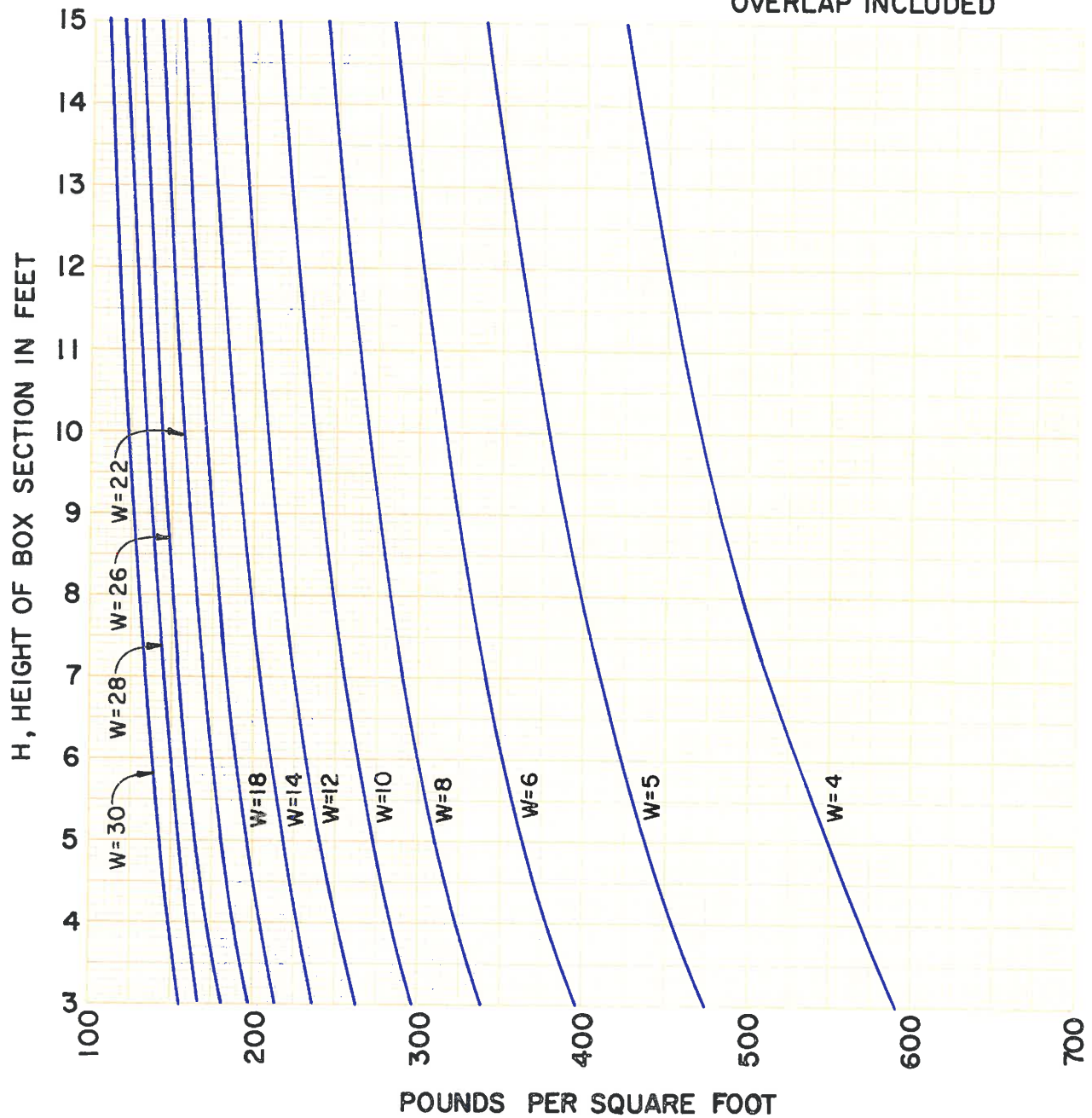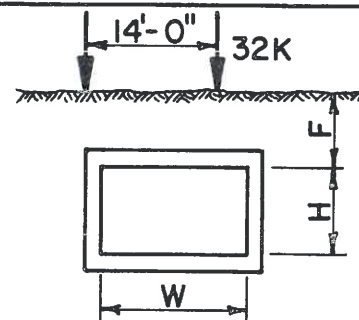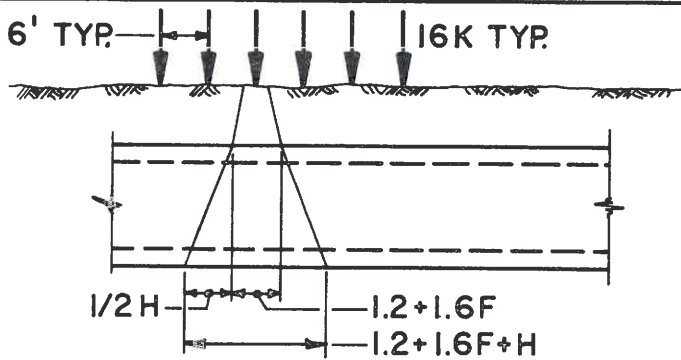
MAIN REINFORCEMENT IS PARALLEL
TO TRAFFIC.

DEPTH OF COVER IS 2'-11"

ORANGE COUNTY FLOOD CONTROL DISTRICT

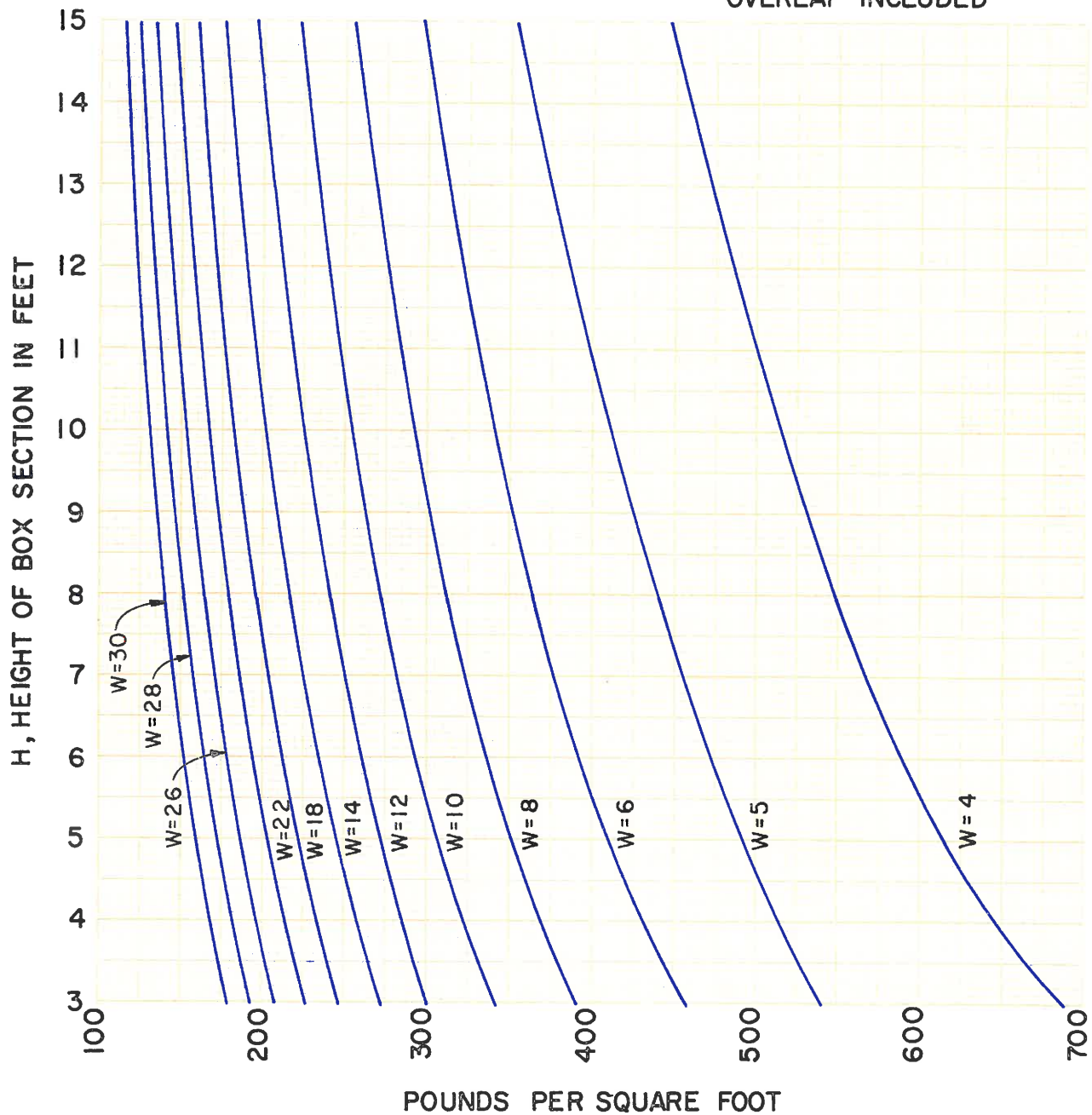H-20 TRUCK LOADS
ON INVERT SLABS
OF BOX CONDUITS

| 1972 | 6 OF 6 | ST-9 |

Average Side H-20 Truck Loads on Box Conduits

Chart axes:
- Vertical: H, HEIGHT OF BOX SECTION, (FT.), ranging from 2 to 14
- Horizontal: W_H, AVERAGE UNIFORM HORIZONTAL LOAD, (Lbs. per Sq. Ft.), ranging from 0 to 550

Curves labeled: 7'-10' COVER, 6' COVER, 5' COVER, 4' COVER, 3' COVER, 2' COVER, 1' COVER, 0' COVER

Inset diagram labels: TRUCK LOADS, GROUND, COVER, H, W_H
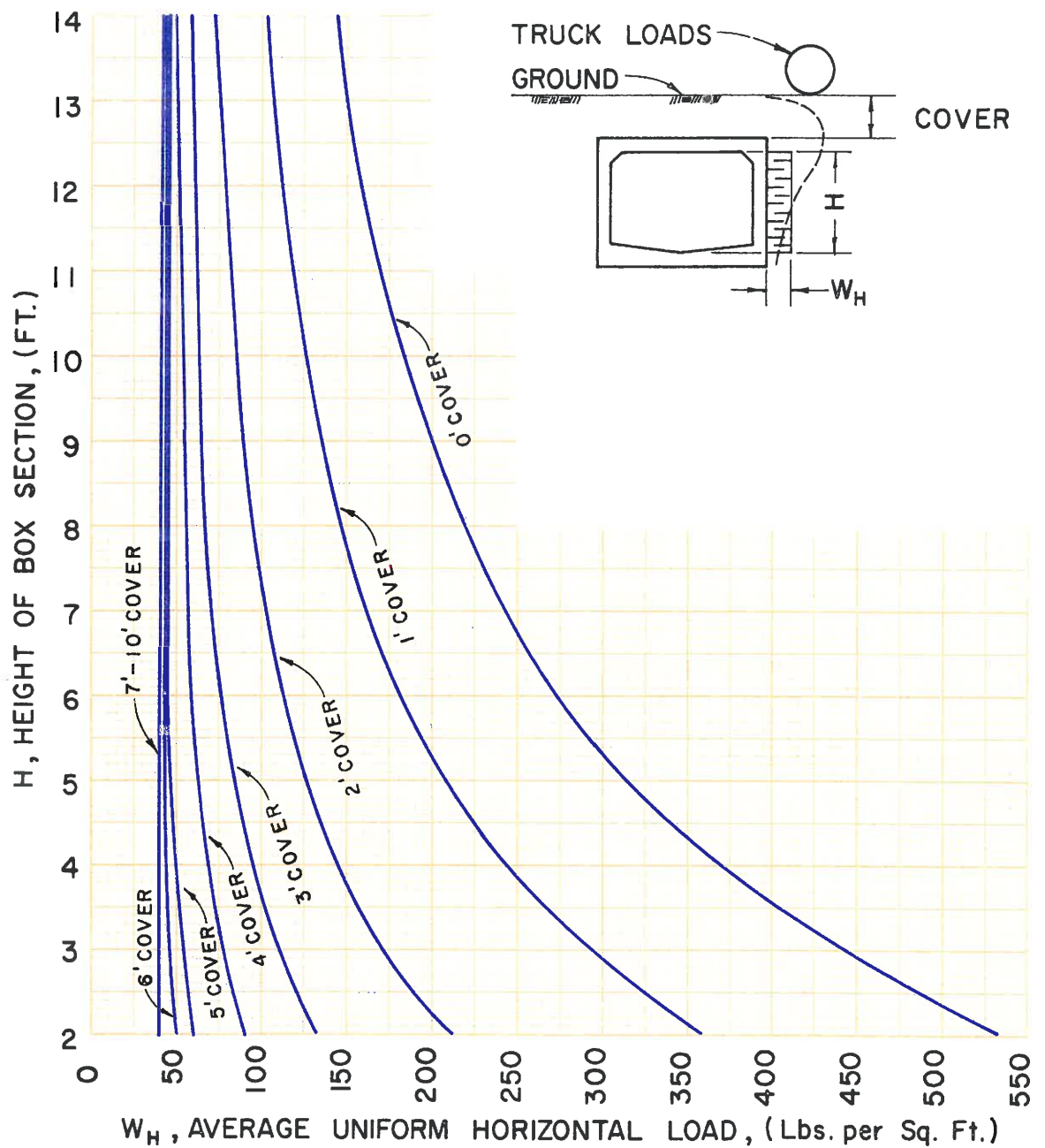
IMPACT COEFFICIENTS:

Cover	Impact %	
0' TO 1' COVER	30 %	
1'-1" TO 2' COVER	20 %	INCLUDED IN CURVES
2'-1" TO 2'-11" COVER	10 %	
3' COVER AND OVER	0 %	

ORANGE COUNTY FLOOD CONTROL DISTRICT

AVERAGE SIDE
H-20 TRUCK LOADS ON
BOX CONDUITS

| 1972 | 1 OF 1 | ST-10 |

Single track impact equals LL/(DL*+LL)
DL not included

*In calculation of DL the following
assumptions were made:
   Rails, ballast, ties etc. = 115 lbs/sq.ft.
   Top slab = 150 lbs/sq.ft.
   Earth = 120 lbs/cu.ft. x cover in feet
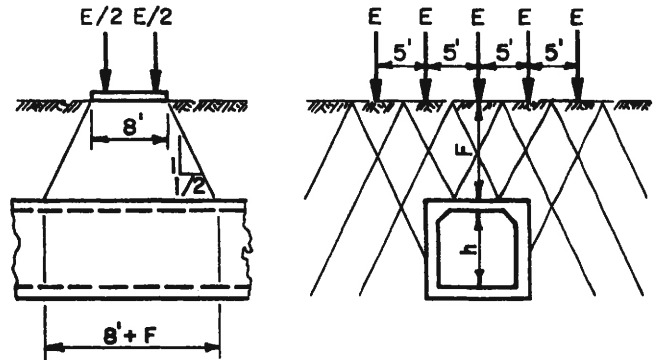
COVER IN FEET	VERTICAL LOAD POUNDS PER SQUARE FOOT	
	E – 72	E – 75
0	3400	3500
1	2900	3000
2	2500	2600
3	2200	2300
4	1950	2050
5	1750	1800
6	1550	1600
7	1400	1450
8	1300	1350
9	1200	1250
10	1100	1150
11	1000	1050
12	950	1000
13	900	950
14	850	900
15	800	850

COOPER'S E – 72 AND E – 75
RAILROAD LOADS

ORANGE COUNTY FLOOD CONTROL DISTRICT		
VERTICAL RAILROAD LOADS ON TOP SLABS OF BOX CONDUITS		
1972	1 OF 1	ST–11

h, HEIGHT OF BOX SECTION IN FEET (y-axis: 2 to 15)

Curve labels: F=16', F=12', F=10', F=8', F=6', F=4', F=3', F=2', F=1', F=0'

VERTICAL LOAD – KIPS PER SQUARE FOOT (x-axis: 0.4 to 2.8)

Diagram labels: E/2, E/2, 8', 1/2, h/2, 8'+F, h/2, 8'+F+h

E, E, E, E, E, 5', 5', 5', 5', F, h

DATA:

Single track impact equals $LL/(DL^* + LL)$
DL not included

*In calculation of DL the following assumptions were made:
  Rails, ballast, ties etc. = 115 lbs/sq.ft.
  Top slab = 150 lbs/sq.ft.
  Earth = 120 lbs/cu. ft. x cover in feet
  Side walls = 175 lbs/sq.ft.

COOPER'S E-72
RAILROAD LOADS

NOTE:
  E-70 curves adjusted linearly
  for E-72

ORANGE COUNTY FLOOD CONTROL DISTRICT		
RAILROAD LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	1 OF 2	ST-12
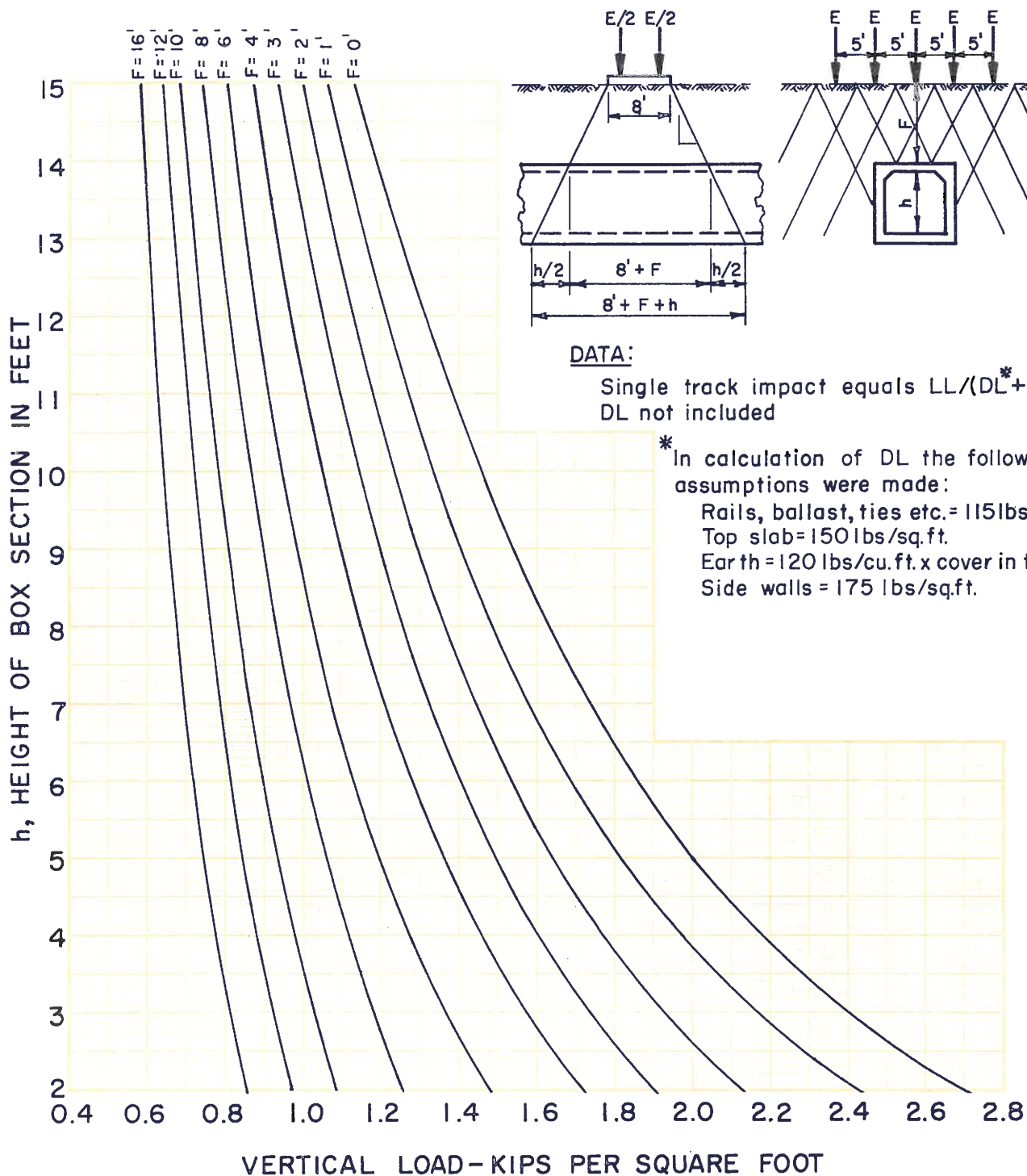
VERTICAL LOAD—KIPS PER SQUARE FOOT

y-axis: h, HEIGHT OF BOX SECTION IN FEET

Curves labeled: F=16', F=12', F=10', F=8', F=6', F=4', F=3', F=2', F=1', F=0'

DATA:

Single track impact equals LL/(DL*+LL)
DL not included

*In calculation of DL the following
assumptions were made:
Rails, ballast, ties etc.= 115 lbs/sq.ft.
Top slab = 150 lbs/sq.ft.
Earth = 120 lbs/cu.ft. x cover in feet
Side walls = 175 lbs/sq.ft.

COOPER S E-75
RAILROAD LOADS

NOTE:
E-70 curves adjusted linearly
for E-75

ORANGE COUNTY FLOOD CONTROL DISTRICT		
RAILROAD LOADS ON INVERT SLABS OF BOX CONDUITS		
1972	2 OF 2	ST-13

h, HEIGHT OF BOX SECTION IN FEET (vertical axis, 2 to 15)
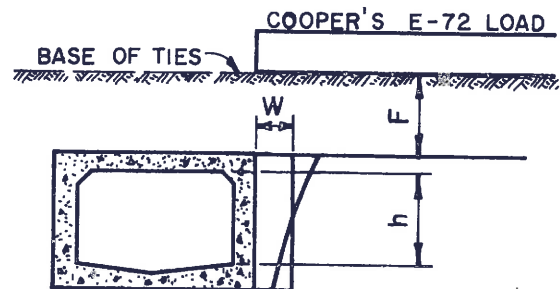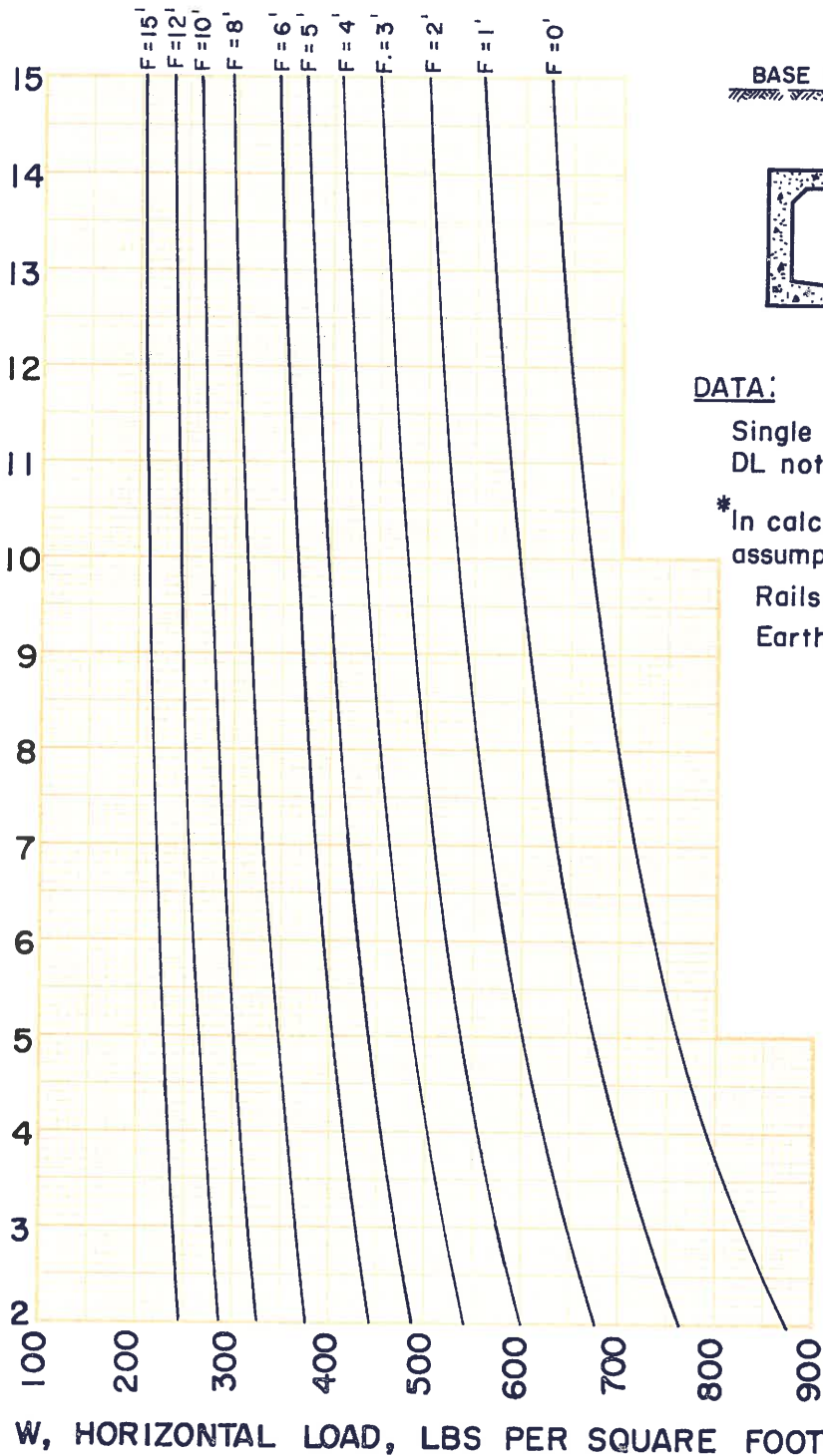
Curve labels: F=15', F=12', F=10', F=8', F=6', F=5', F=4', F=3', F=2', F=1', F=0'

W, HORIZONTAL LOAD, LBS PER SQUARE FOOT (horizontal axis, 100 to 900)

COOPER'S E-72 LOAD

BASE OF TIES

W

F

h

DATA:

Single track impact equals LL/(DL* +LL)
DL not included

*In calculation of DL the following assumptions were made:

Rails, ballast, ties etc = 115 lbs/sq.ft.

Earth = 120 lbs/cu. ft. x cover in feet

COOPER S   E-72
RAILROAD   LOADS

ORANGE COUNTY FLOOD CONTROL DISTRICT

HORIZONTAL
RAILROAD   LOADS
ON   BOX   CONDUITS

| 1972 | 1 OF 2 | ST-14 |

NOTE:
E-70 curves adjusted linearly for E-72

h, HEIGHT OF BOX SECTION IN FEET

F=15 F=12 F=10 F=8 F=6 F=5 F=4 F=3 F=2 F=1 F=0

W, HORIZONTAL LOAD, LBS PER SQUARE FOOT

COOPER'S E-75 LOAD

BASE OF TIES

DATA:

Single track impact equals LL/(DL*+LL)
DL not included

In calculation of DL the following
assumptions were made:

Rails, ballast, ties etc. = 115 lbs/sq. ft.
Earth = 120 lbs/cu. ft. x cover in feet

NOTE:

E-70 curves adjusted linearly for E-75

COOPER'S E-75
RAILROAD LOADS

ORANGE COUNTY FLOOD CONTROL DISTRICT

HORIZONTAL
RAILROAD LOADS
ON BOX CONDUITS

| 1972 | 2 OF 2 | ST-15 |

CASE I
Channel Empty

P = Resultant load due to weight of wall and earth
load (110 lbs./ft.^{3}) on heel.

M_1 = Moment at "A" due to external horizontal forces
acting on wall.
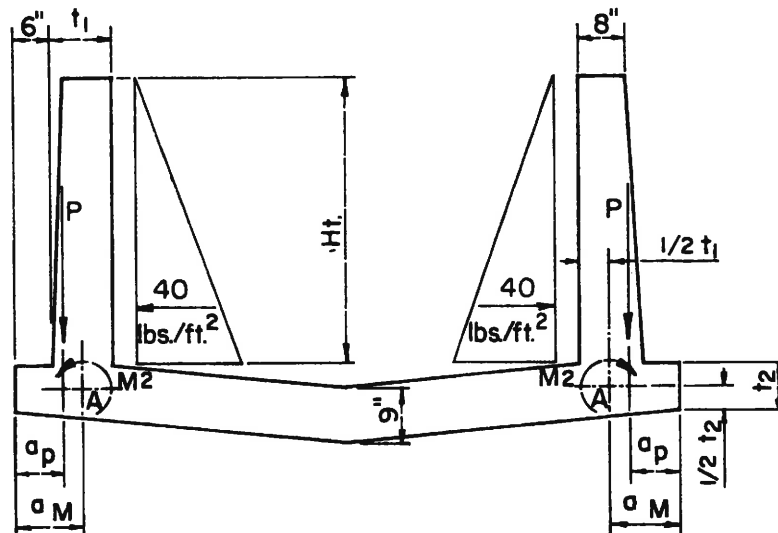
CASE II
Channel Full

P = Resultant load due to weight of wall and earth
load (110 lbs./ft.^{3}) on heel.

M_2 = Moment at "A" due to equivalent differential
hydrostatic pressure.

ORANGE COUNTY FLOOD CONTROL DISTRICT		
DESIGN OF INVERT SLABS IN RECTANGULAR CHANNELS		
1972	I OF 9	ST-16
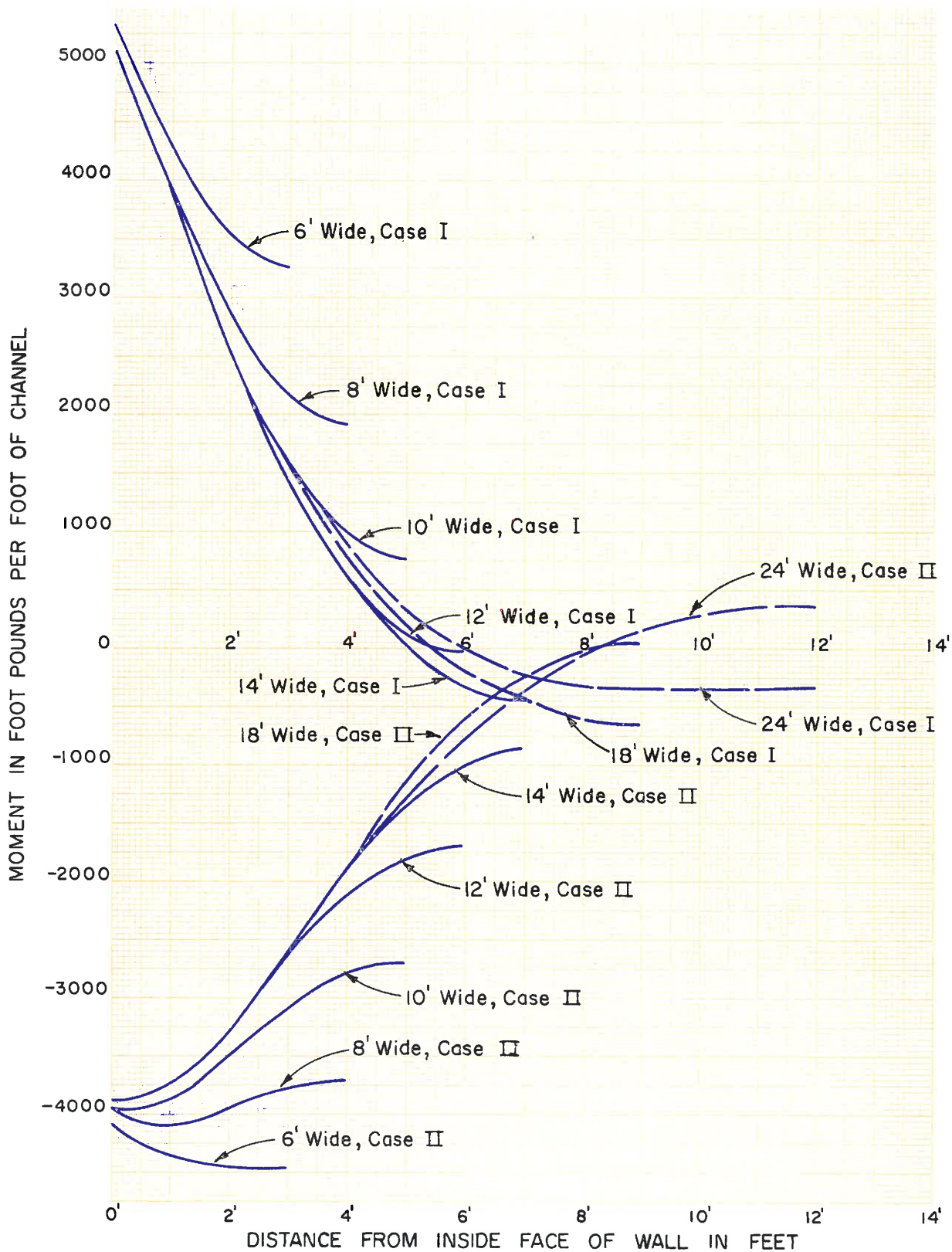
MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

DISTANCE FROM INSIDE FACE OF WALL IN FEET

6' Wide, Case I

8' Wide, Case I

10' Wide, Case I

12' Wide, Case I

24' Wide, Case II

14' Wide, Case I

18' Wide, Case II

24' Wide, Case I

18' Wide, Case I

14' Wide, Case II

12' Wide, Case II

10' Wide, Case II

8' Wide, Case II

6' Wide, Case II

CASE I & II

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS IN INVERT SLABS
RECTANGULAR CHANNELS
8 FT. HIGH WALLS

| 1972 | 2 OF 9 | ST-17 |

MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

DISTANCE FROM INSIDE — FACE OF WALL

8' Wide, Case I
10' Wide, Case I
12' Wide, Case I
16' Wide, Case I
20' Wide, Case II
26' Wide, Case II
26' Wide, Case I
20' Wide, Case I
16' Wide, Case II
12' Wide, Case II
10' Wide, Case II
8' Wide, Case II

CASE I & II

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS IN INVERT SLABS
RECTANGULAR CHANNELS
10 FT. HIGH WALLS

| 1972 | 3 OF 9 | ST-18 |

CASE I

MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

DISTANCE FROM INSIDE FACE OF WALL IN FEET

8' Wide, Case I
10' Wide, Case I
12' Wide, Case I
14' Wide, Case I
16' Wide, Case I
18' Wide, Case I
Over 18' Wide, Case I

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS IN INVERT SLABS
RECTANGULAR CHANNELS
12 FT. HIGH WALLS

| 1972 | 4 OF 9 | ST-19 |

CASE II

MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

DISTANCE FROM INSIDE FACE OF WALL IN FEET

Over 18' Wide, Case II

18' Wide, Case II

16' Wide, Case II

14' Wide, Case II

12' Wide, Case II

10' Wide, Case II

8' Wide, Case II

ORANGE COUNTY FLOOD CONTROL DISTRICT		
MOMENTS IN INVERT SLABS RECTANGULAR CHANNELS 12 FT. HIGH WALLS		
1972	5 OF 9	ST-20

MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

26000
24000
22000
20000
18000
16000
14000
12000
10000
8000
6000
4000
2000
0
-2000

10' Wide, Case I

12' Wide, Case I

14' Wide, Case I

16' Wide, Case I

18' Wide, Case I

28' Wide, Case I

20' Wide, Case I

22' Wide, Case I

DISTANCE FROM INSIDE FACE OF WALL IN FEET

0   2   4   6   8   10   12   14

CASE I

MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

DISTANCE FROM INSIDE FACE OF WALL IN FEET

28' Wide, Case II
26' Wide, Case II
22' Wide, Case II
20' Wide, Case II
18' Wide, Case II
16' Wide, Case II
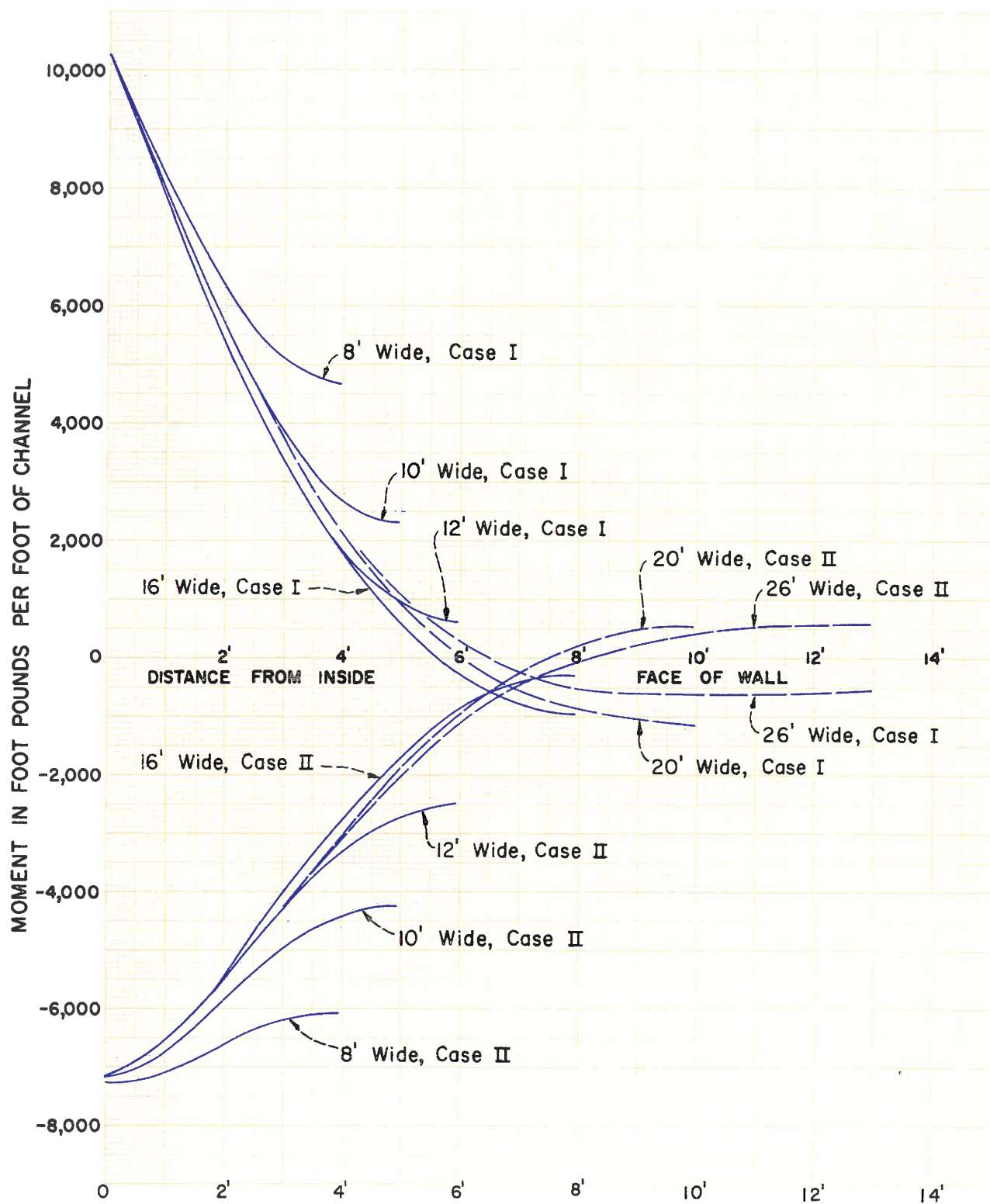14' Wide, Case II
12' Wide, Case II
10' Wide, Case II

CASE II

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS IN INVERT SLABS
RECTANGULAR CHANNELS
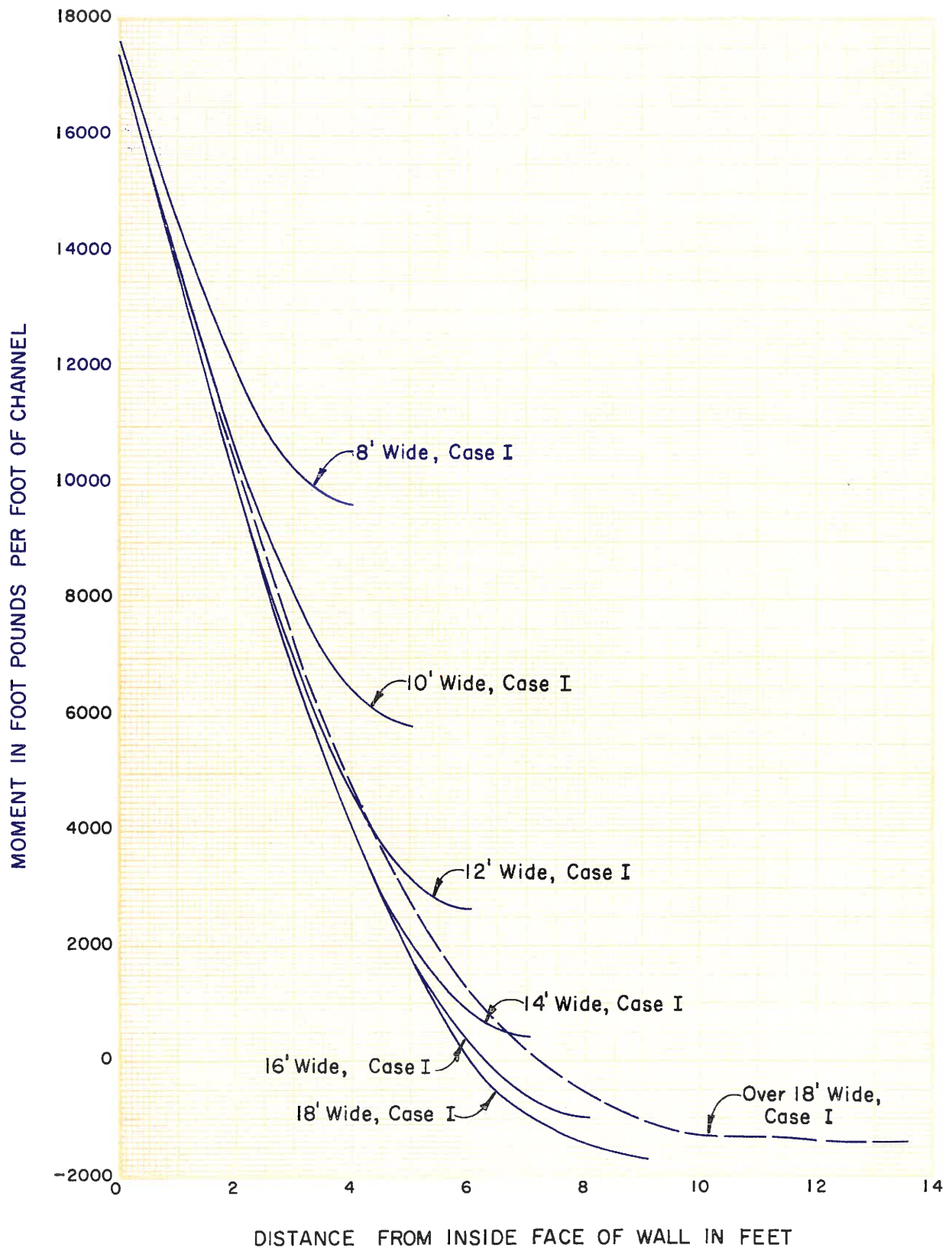14 FT. HIGH WALLS

| 1972 | 7 OF 9 | ST-22 |

CASE I

Moment in foot pounds per foot of channel (y-axis)
Distance from inside face of wall in feet (x-axis)

- 10' Wide, Case I
- 12' Wide, Case I
- 14' Wide, Case I
- 16' Wide, Case I
- 18' Wide, Case I
- 20' Wide, Case I
- 22' Wide, Case I
- 28' Wide, Case I

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS IN INVERT SLABS
RECTANGULAR CHANNELS
16 FT. HIGH WALLS

| 1972 | 8 OF 9 | ST-23 |
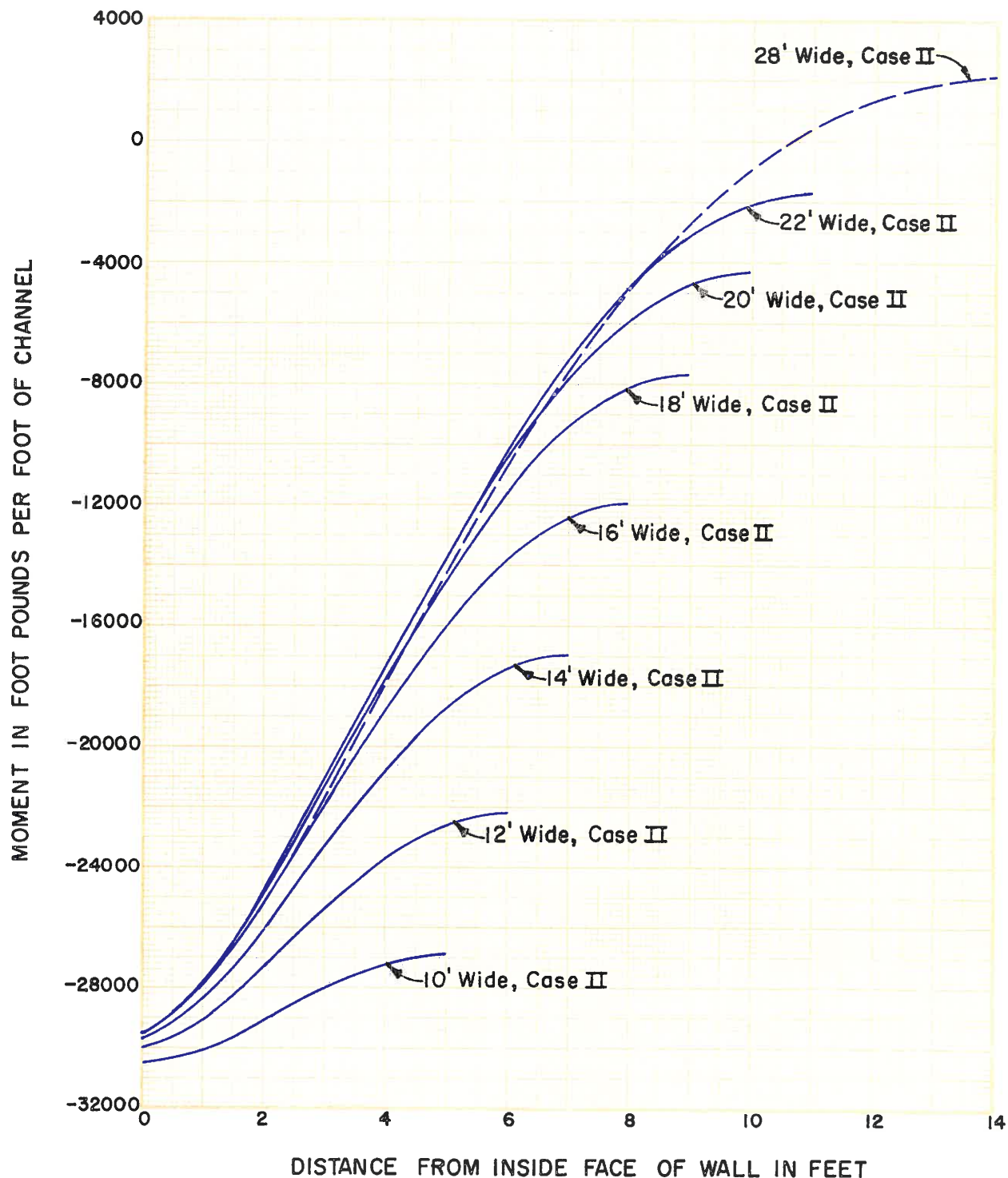
MOMENT IN FOOT POUNDS PER FOOT OF CHANNEL

DISTANCE FROM INSIDE FACE OF WALL IN FEET

28' Wide, Case II
22' Wide, Case II
20' Wide, Case II
18' Wide, Case II
16' Wide, Case II
14' Wide, Case II
12' Wide, Case II
10' Wide, Case II

CASE II

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS IN INVERT SLABS
RECTANGULAR CHANNELS
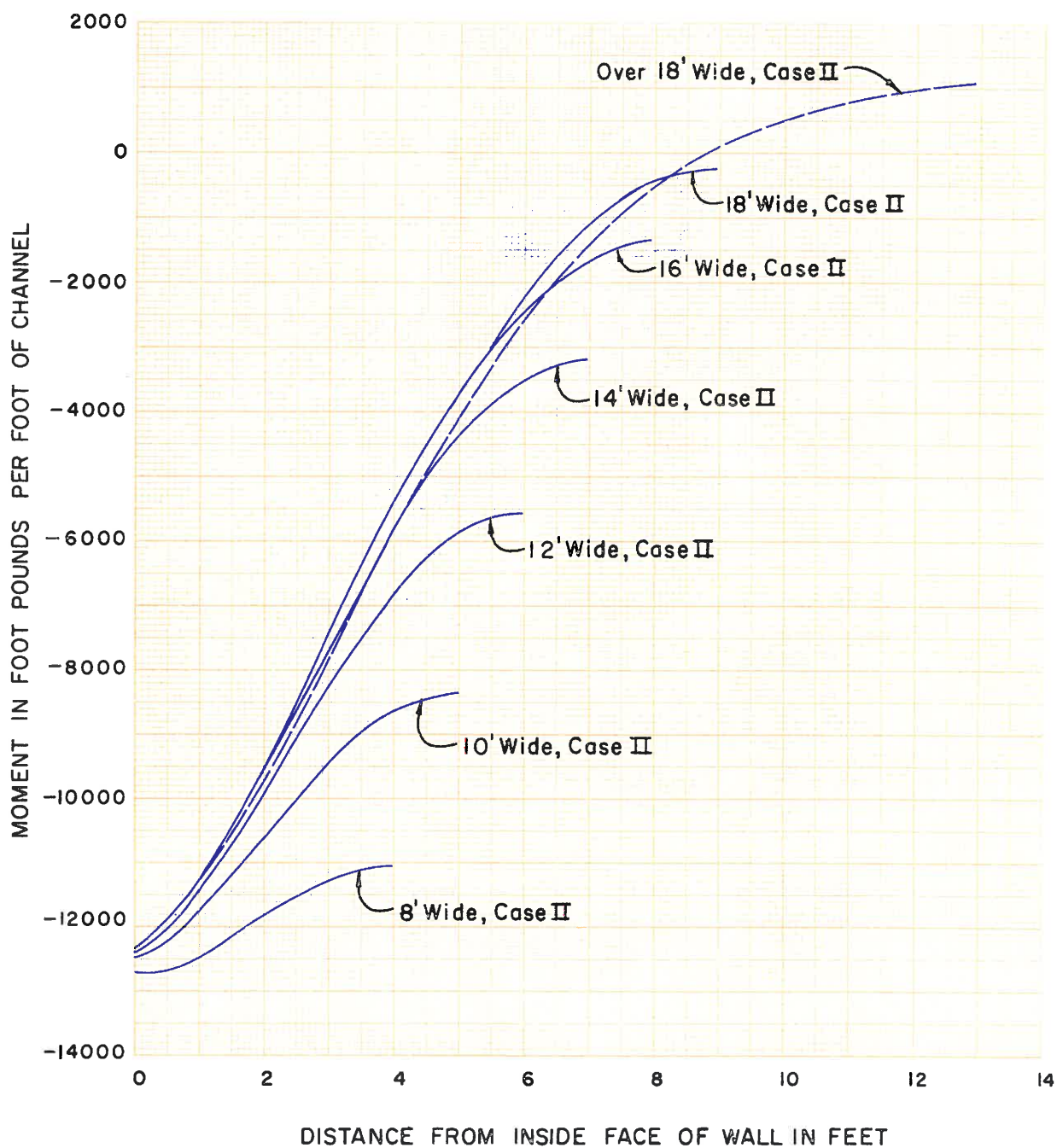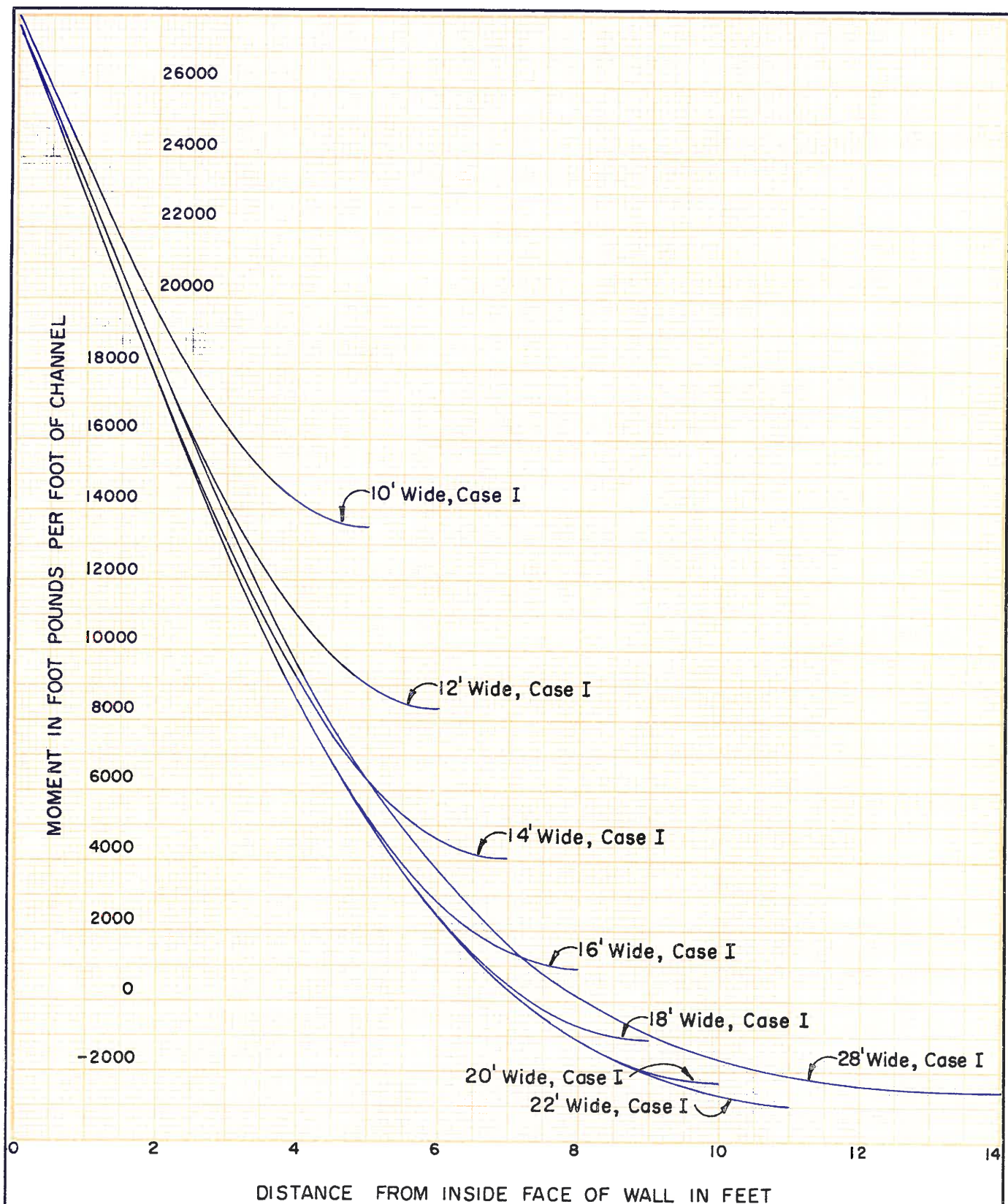16 FT. HIGH WALLS

| 1972 | 9 OF 9 | ST-24 |
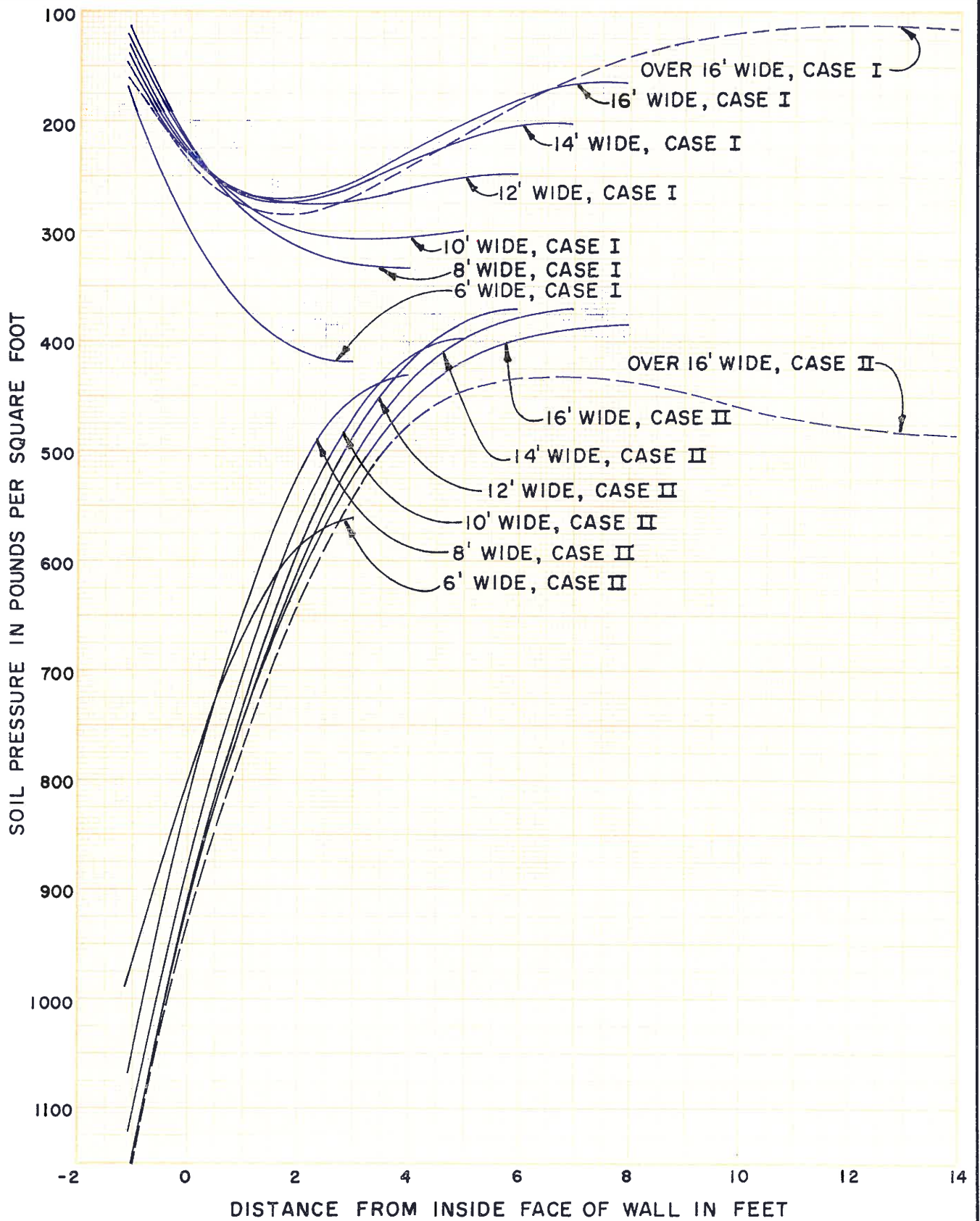
CASE I & II
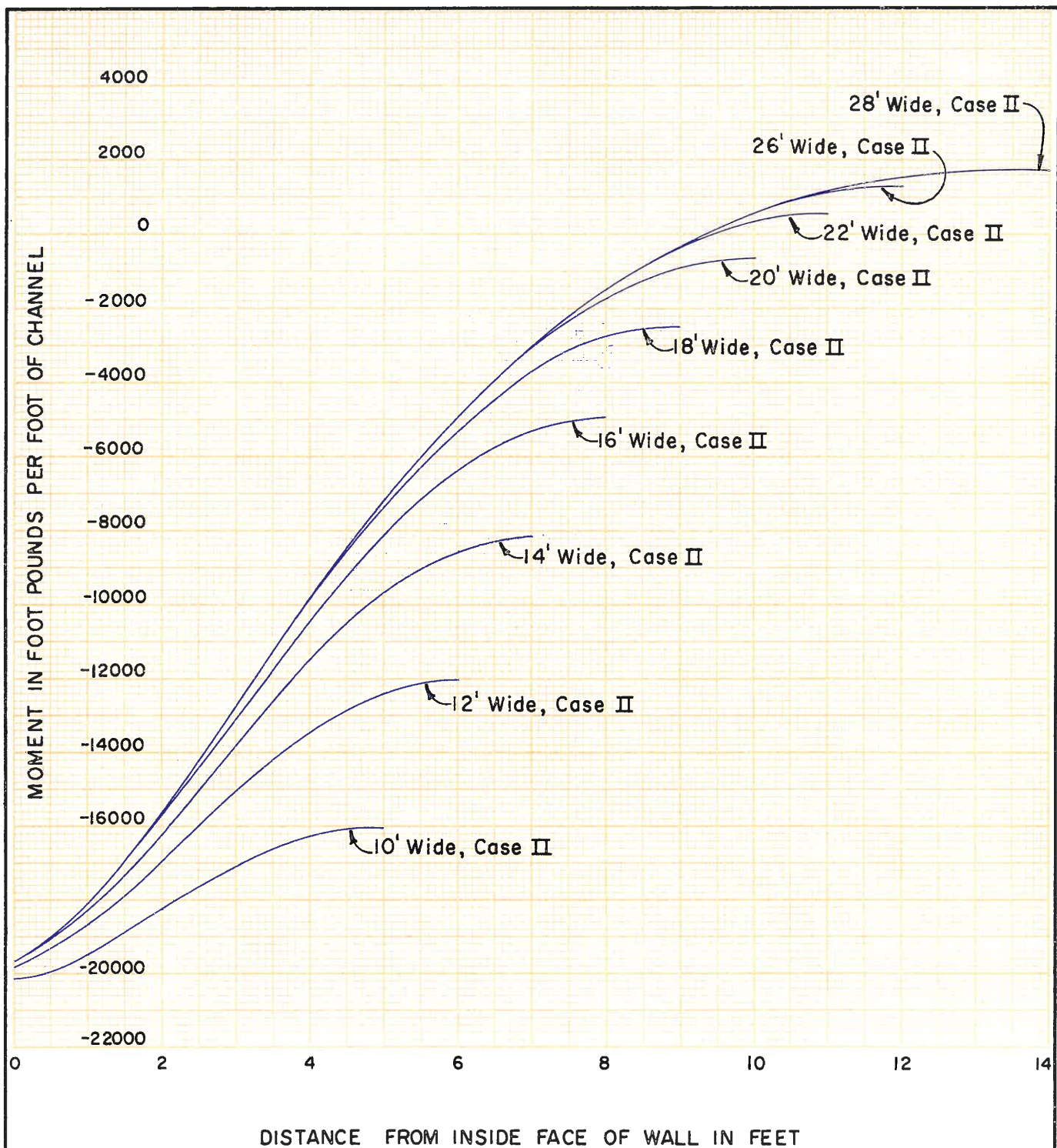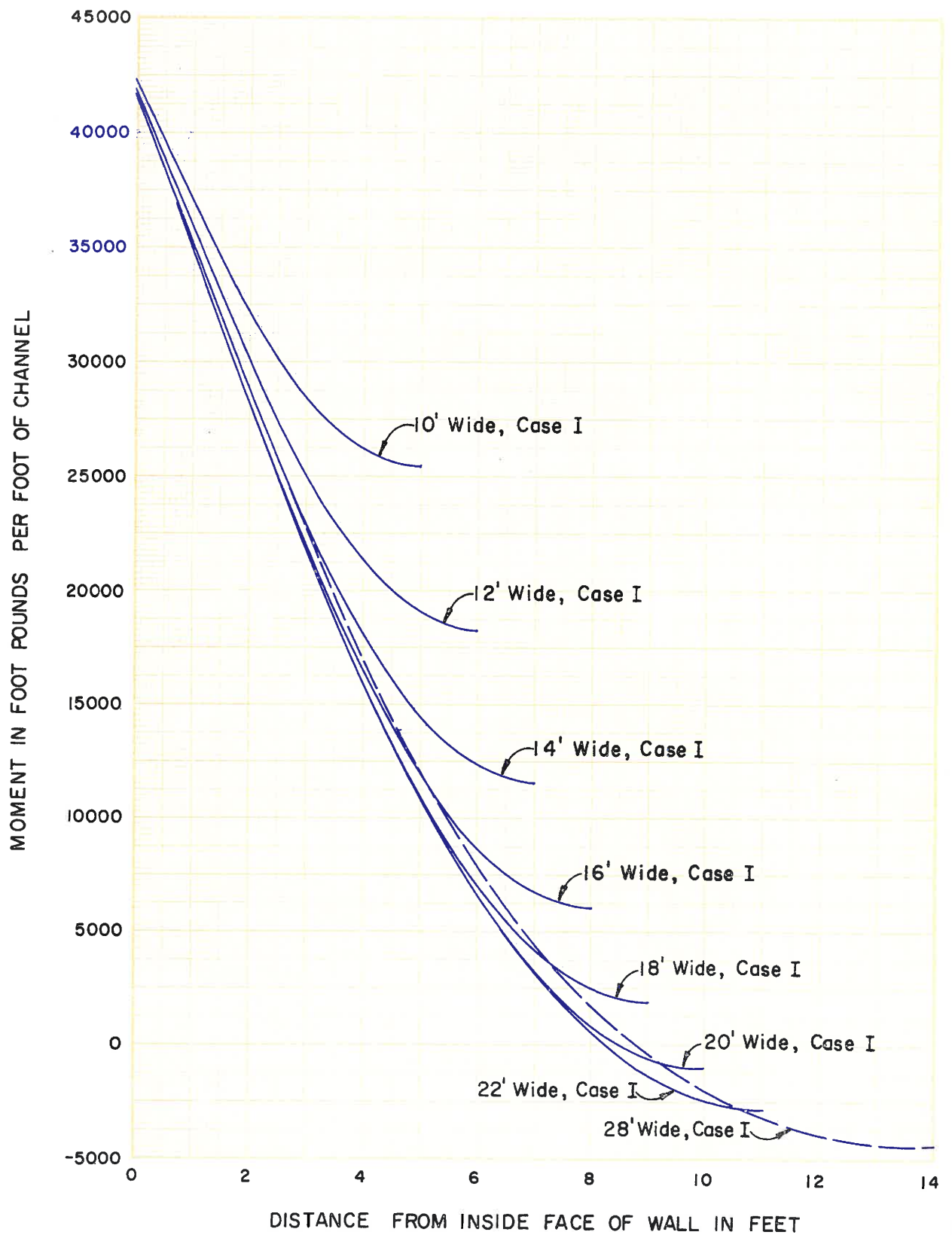
ORANGE COUNTY FLOOD CONTROL DISTRICT		
SOIL PRESSURES ON INVERTS RECTANGULAR CHANNELS 6 FT. HIGH WALLS		
1972	I OF 4	ST-25

Y-axis: SOIL PRESSURE IN POUNDS PER SQUARE FOOT (400, 200, 0, 200, 400, 600, 800, 1000, 1200, 1400, 1600)

X-axis: DISTANCE FROM INSIDE FACE OF WALL IN FEET (-2, 0, 2, 4, 6, 8, 10, 12, 14)

Curve labels:
- 8' WIDE, CASE I
- 10' WIDE, CASE I
- 12' WIDE, CASE I
- 14' WIDE, CASE I
- 16' WIDE, CASE I
- OVER 16' WIDE, CASE I
- OVER 16' WIDE, CASE II
- 16' WIDE, CASE II
- 14' WIDE, CASE II
- 12' WIDE, CASE II
- 10' WIDE, CASE II
- 8' WIDE, CASE II

CASE I & II

ORANGE COUNTY FLOOD CONTROL DISTRICT

SOIL PRESSURES ON INVERTS
RECTANGULAR CHANNELS
8 FT. HIGH WALLS

| 1972 | 1 OF 4 | ST-26 |

CASE I & II

SOIL PRESSURE IN POUNDS PER SQUARE FOOT

DISTANCE FROM INSIDE FACE OF WALL IN FEET

8' WIDE, CASE II
10' WIDE, CASE II
12' WIDE, CASE II
14' WIDE, CASE II
16' WIDE, CASE II
18' WIDE, CASE II
OVER 18' WIDE, CASE II

OVER 18' WIDE, CASE I
18' WIDE, CASE I
16' WIDE, CASE I
14' WIDE, CASE I
12' WIDE, CASE I
10' WIDE, CASE I
8' WIDE, CASE I

ORANGE COUNTY FLOOD CONTROL DISTRICT

SOIL PRESSURES ON INVERTS
RECTANGULAR CHANNELS
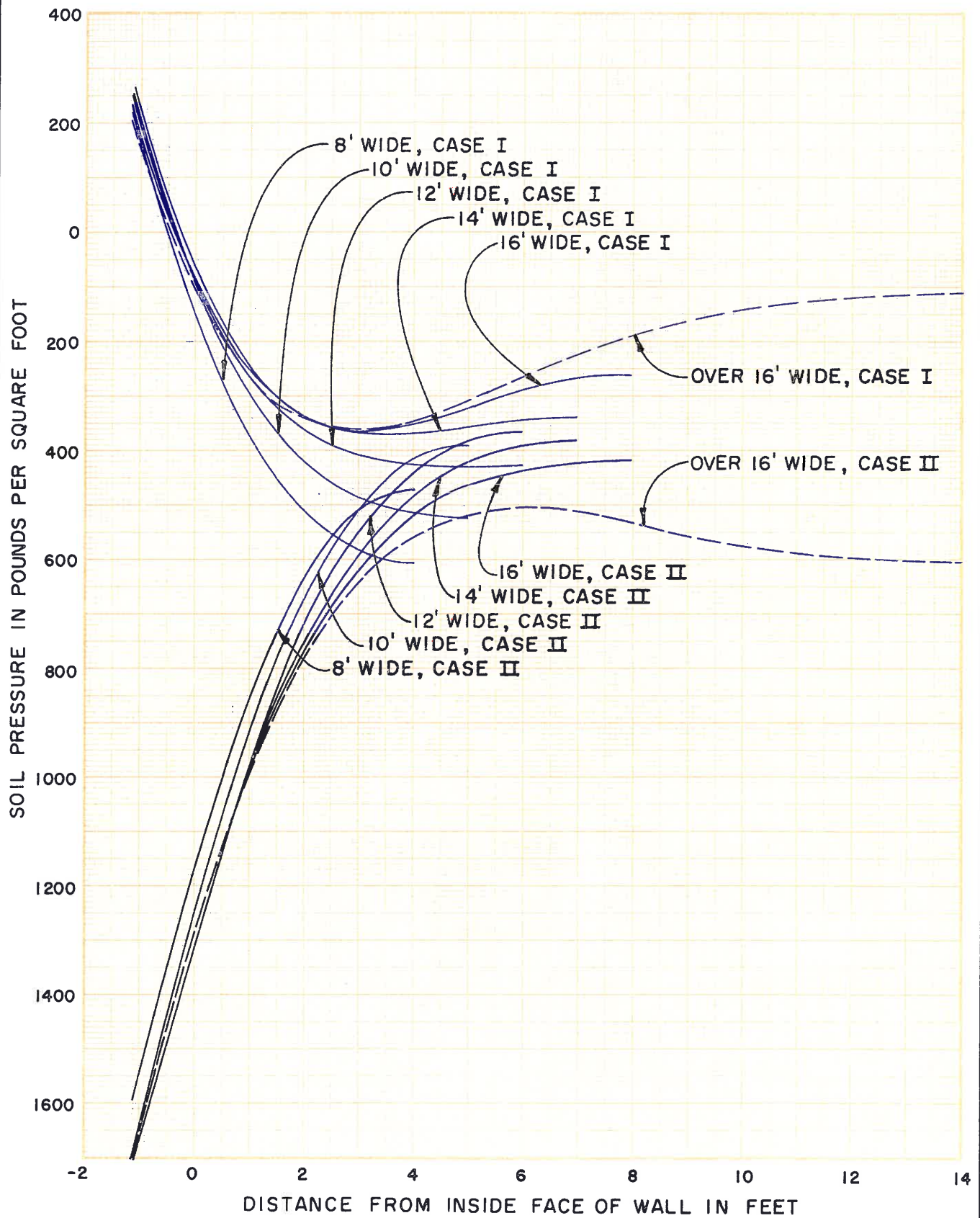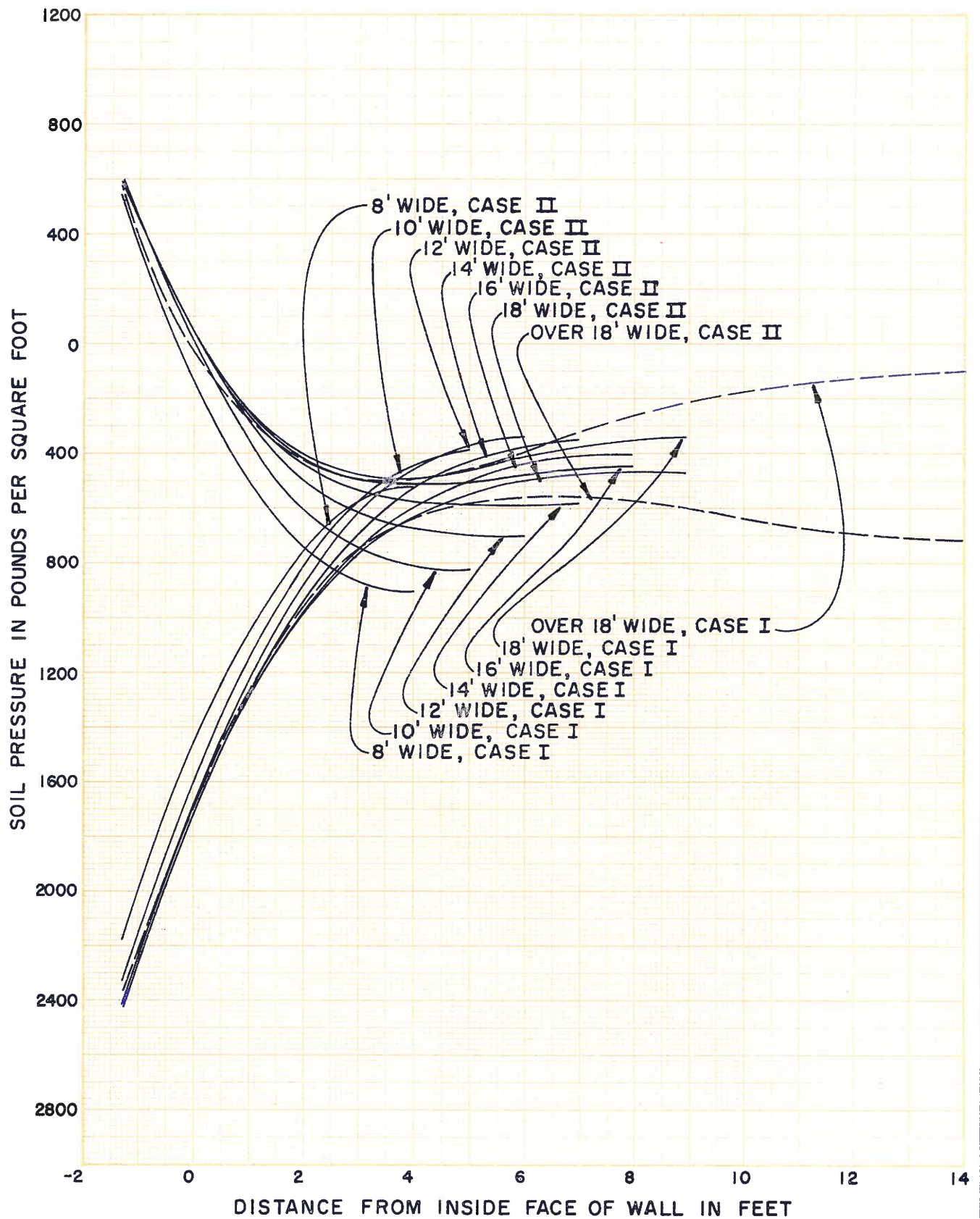10 FT. HIGH WALLS

| 1972 | 3 OF 4 | ST-27 |

CLASS I & II

ORANGE COUNTY FLOOD CONTROL DISTRICT		
SOIL PRESSURES ON INVERTS RECTANGULAR CHANNELS 12 FT. HIGH WALLS		
1972	4 OF 4	ST-28

Chart title and axes:

SOIL PRESSURE IN POUNDS PER SQUARE FOOT (y-axis: 1200, 800, 400, 0, 400, 800, 1200, 1600, 2000, 2400, 2800)

DISTANCE FROM INSIDE FACE OF WALL IN FEET (x-axis: -2, 0, 2, 4, 6, 8, 10, 12, 14)

Curve labels:
- 10' WIDE, CASE II
- 12' WIDE, CASE II
- 14' WIDE, CASE II
- 16' WIDE, CASE II
- 18' WIDE, CASE II
- OVER 18' WIDE, CASE I
- OVER 18' WIDE, CASE II
- 18' WIDE, CASE I
- 16' WIDE, CASE I
- 14' WIDE, CASE I
- 12' WIDE, CASE I
- 10' WIDE, CASE I
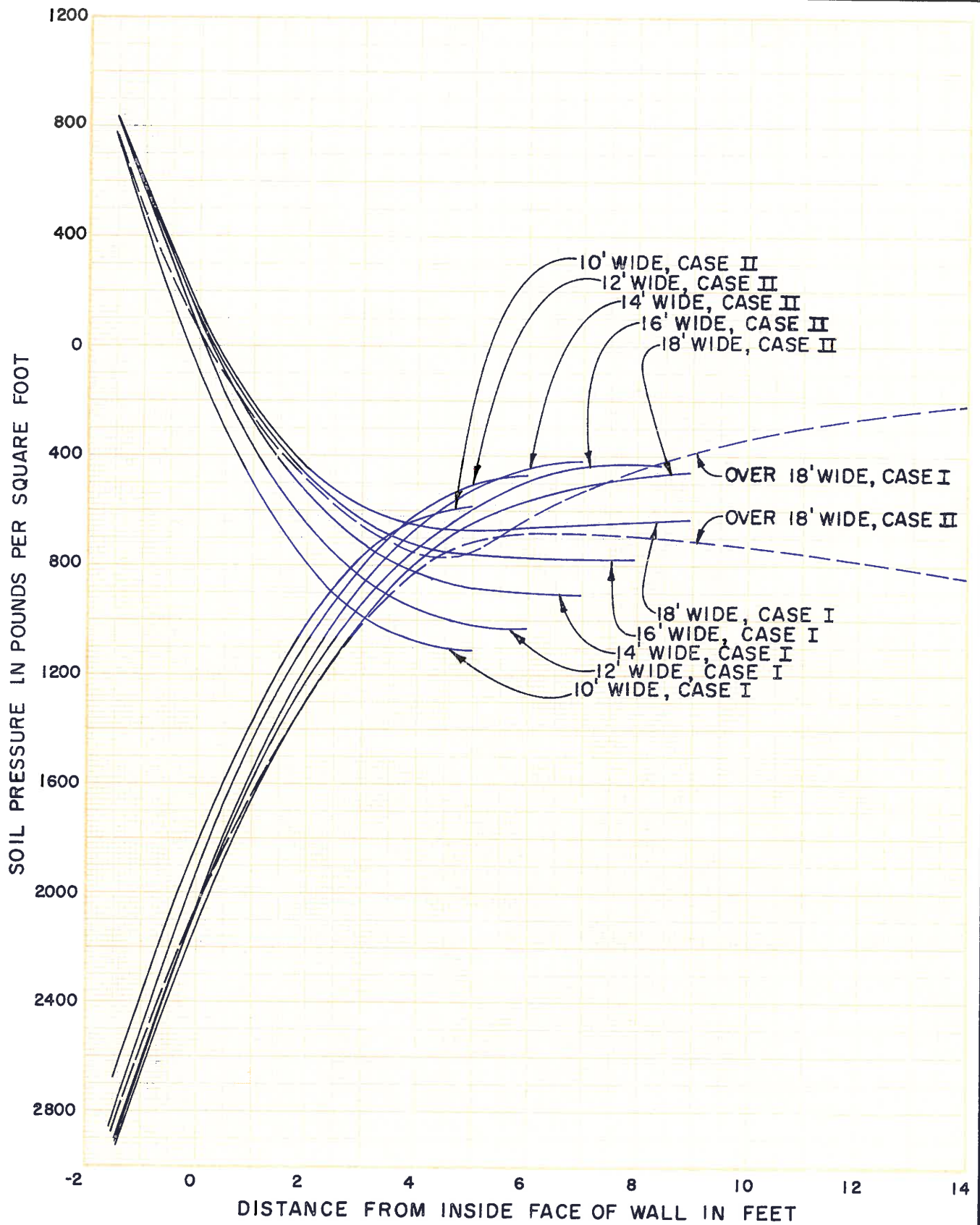
PIPE SIZE	1	1.25	1.5	1.75	2	3	4	5	6	7	8	9	10	PIPE SIZE
12	2000	2000	1500	1500	1500	1250	1250	1250	1500	1750	2000	2250	2500	12
15	CONCRETE		CONCRETE			1000				1500	1750	2000	2250	15
18	BACKFILL		BACKFILL	1250			1000		1250				2000	18
21		1500										1750		21
24	1500			1500				1000			1500			24
27		2000	1750							1250				27
30													1750	30
33		1750	1500									1500		33
36				1250					1000					36
39	1600	1500	1400	1300	1100					1100	1300		1600	39
42	1500	1400	1300	1200								1400		42
45	1400	1300	1200	1100	1000						1200			45
48	1300												1500	48
51		1200	1100	1000								1300		51
54	1200	1100							1000					54
57			1000											57
60				900	800	800	800	800	900		1100		1400	60
63				950	850				850			1250		63
66		1150	1050							950				66
69	1250											1200	1350	69
72	1300	1200	1100								1050			72
75	1350			1000										75
78		1250											1300	78
81	1400											1150		81
84			1150						800	900				84
87	1450	1300									1000			87
90													1250	90
93			1200	1050	900									93
96	1500	1350										1100		96
102	1550													102
108		1400											1200	108

Projection Condition
Unrestricted Trench Width

PIPE SIZE	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	PIPE SIZE
12	1750	2000	2000	2000	2250	2250	2250	2250	2250	2250	2250	2250	2500	2500	2500	12
15		1750	1750		2000	2000	2000	2000					2250	2250	2250	15
18	1500			1750	1750				2000	2000						18
21		1500				1750	1750				2000	2000	2000			21
24			1500					1750						2000		24
27				1500					1750						2000	27
30	1250				1500											30
33										1750						33
36		1250														36
39	1200	1300	1300	1400		1500	1600	1700	1700	1800	1800	1900	1900	1900		39
42								1600				1800				42
45		1200								1700						45
48				1400												48
51	1100															51
54																54
57																57
60				1300			1500									60
63		1150	1250			1450			1650		1750		1850		1950	63
66																66
69																69
72								1550								72
75	1050				1350											75
78			1200													78
81															2000	81
84														1950		84
87																87
90																90
93													1900			93
96																96
102																102
108		1100										1850				108

Trench Condition
Trench Width = O.D. + 20 inches
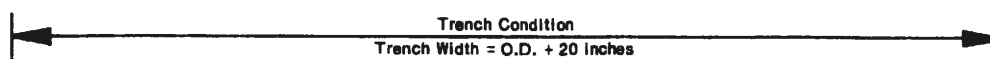
DATA:

DESIGN DENSITY = 110 pcf

LOAD FACTOR = 1.8

LIVE LOAD - H20 - S16 - 44 TRUCK

ORANGE COUNTY FLOOD CONTROL DISTRICT

D - LOAD TABLE FOR REINFORCED CONCRETE PIPE

| 1972 | I OF I | ST-29 |

SLOPE OF WALL (Z)

φ = 2:1

φ = 1¾:1

φ = 1½:1

φ = 1:1

RATIO OF RESULTING PRESSURES TO
PRESSURE ON A VERTICAL WALL
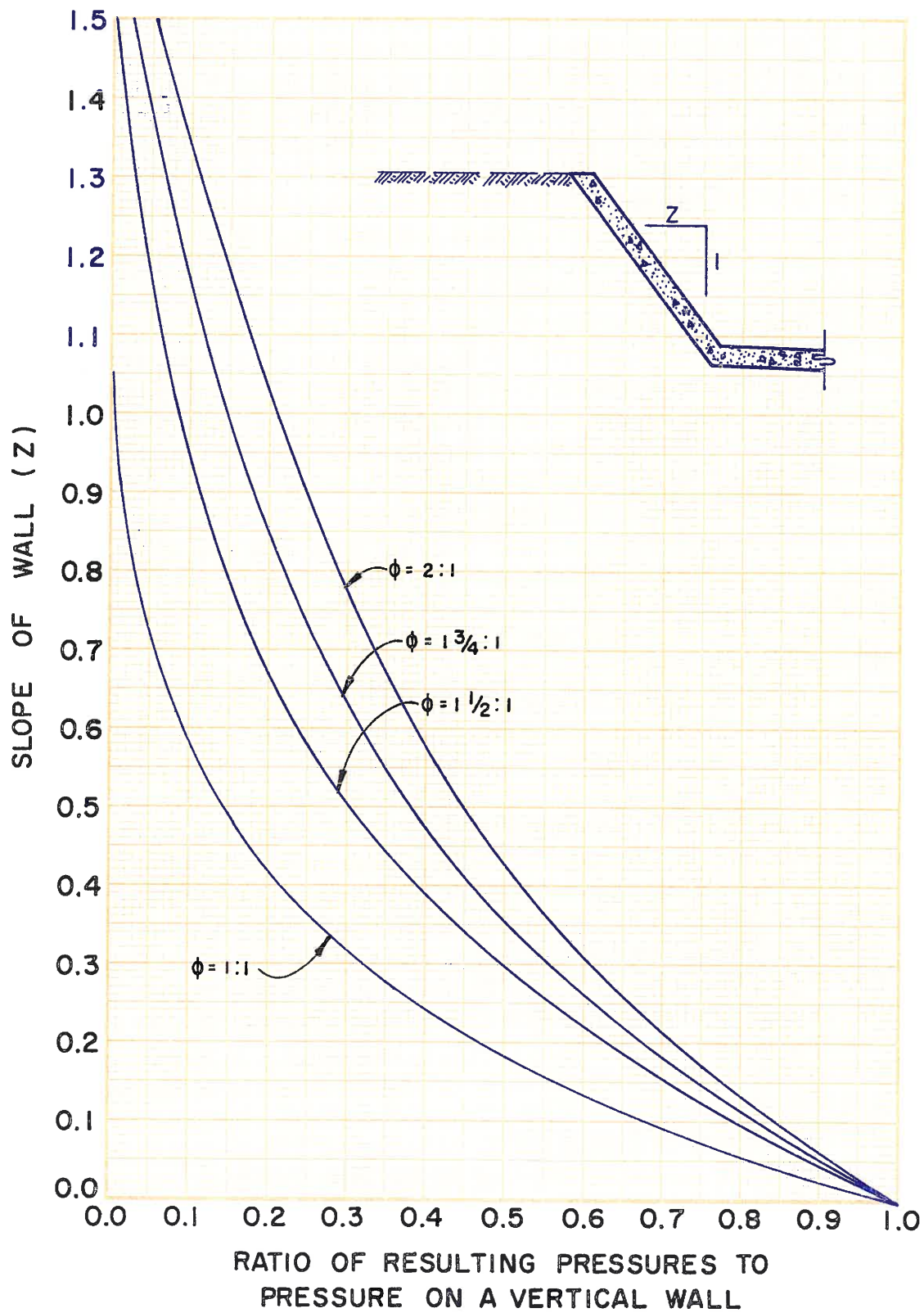
φ = Angle of repose

Source: Bureau of Reclamation, "Canals
and Related Structures"

ORANGE COUNTY FLOOD CONTROL DISTRICT

PRESSURES   ON
SLOPING   WALLS

| 1972 | 1 OF 1 | ST-30 |

SHEAR IN KIPS/FOOT

HEIGHT OF WALL IN FEET

MOMENT

SHEAR

X
16K 6' 16K

HEIGHT

36 PSF
E F P

X=2'
X=4'
X=6'
X=8'

X=2'
X=4'
X=6'
X=8'

MOMENT IN FOOT KIPS/FOOT

X=2'
X=4'
X=6'
X=8'
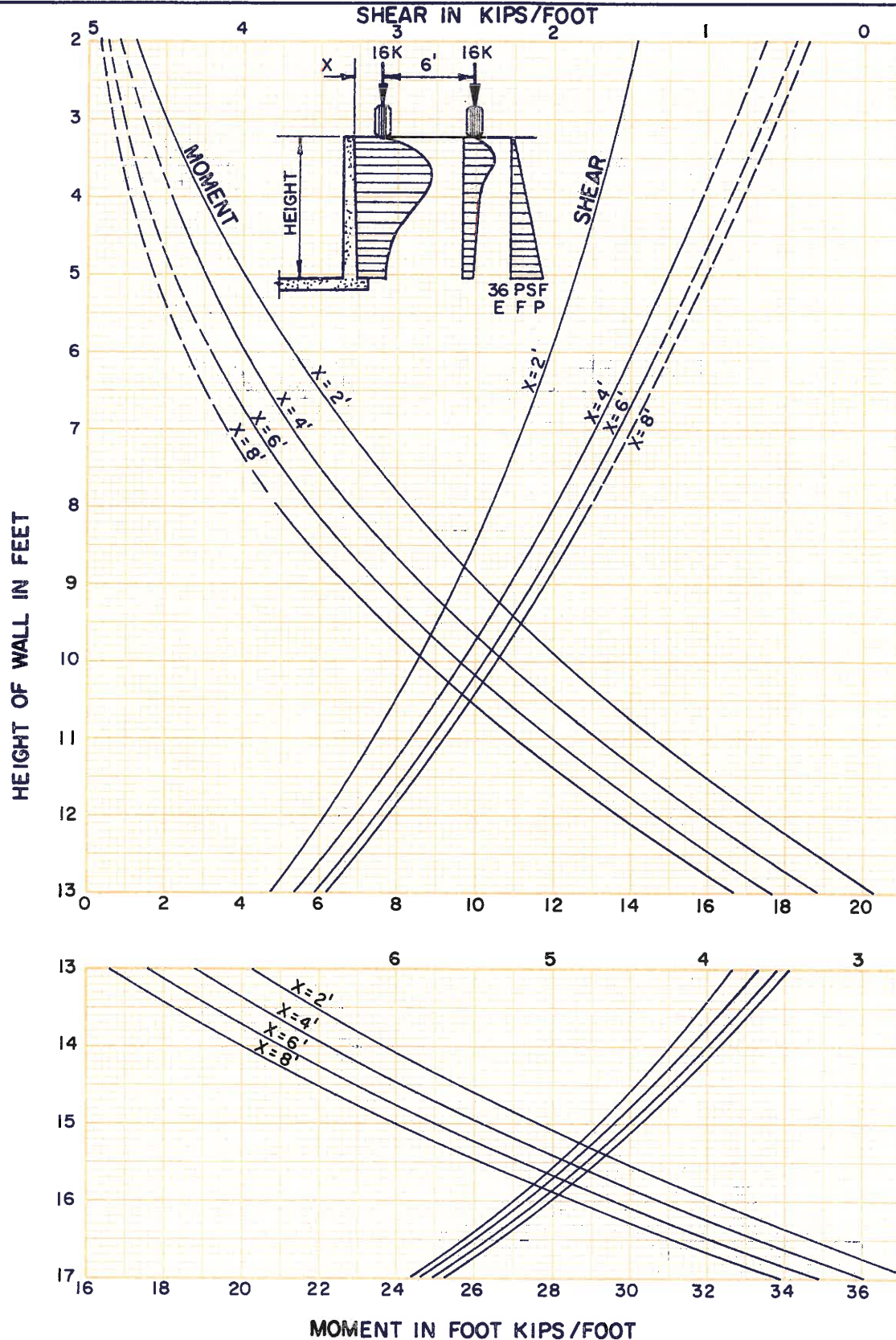
NOTE: The curves include truck surcharge
and impact and 36 PSF Equivalent
Fluid Backfill Pressure.

ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS AND SHEARS ON
CHANNEL WALLS FOR H-20
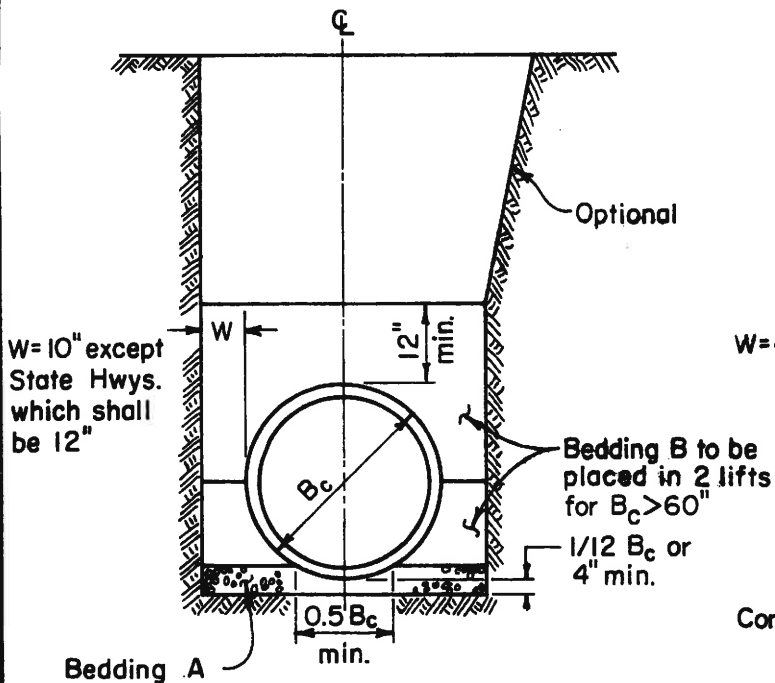TRUCKS PLUS EARTH

| 1972 | 1 OF 1 | ST-31 |

SHEAR - KIPS/FT.

6   5   4   3   2   1

HEIGHT OF WALL - FT.

MOMENT - 62.5 P.S.F. E.F.P.

MOMENT - 40 P.S.F. E.F.P.

SHEAR - 62.5 P.S.F. E.F.P.

SHEAR - 40 P.S.F. E.F.P.

24   26   28   30   32   34

MOMENT - FT. KIPS/FT.

0   2   4   6   8   10   12   14   16   18   20   22   24
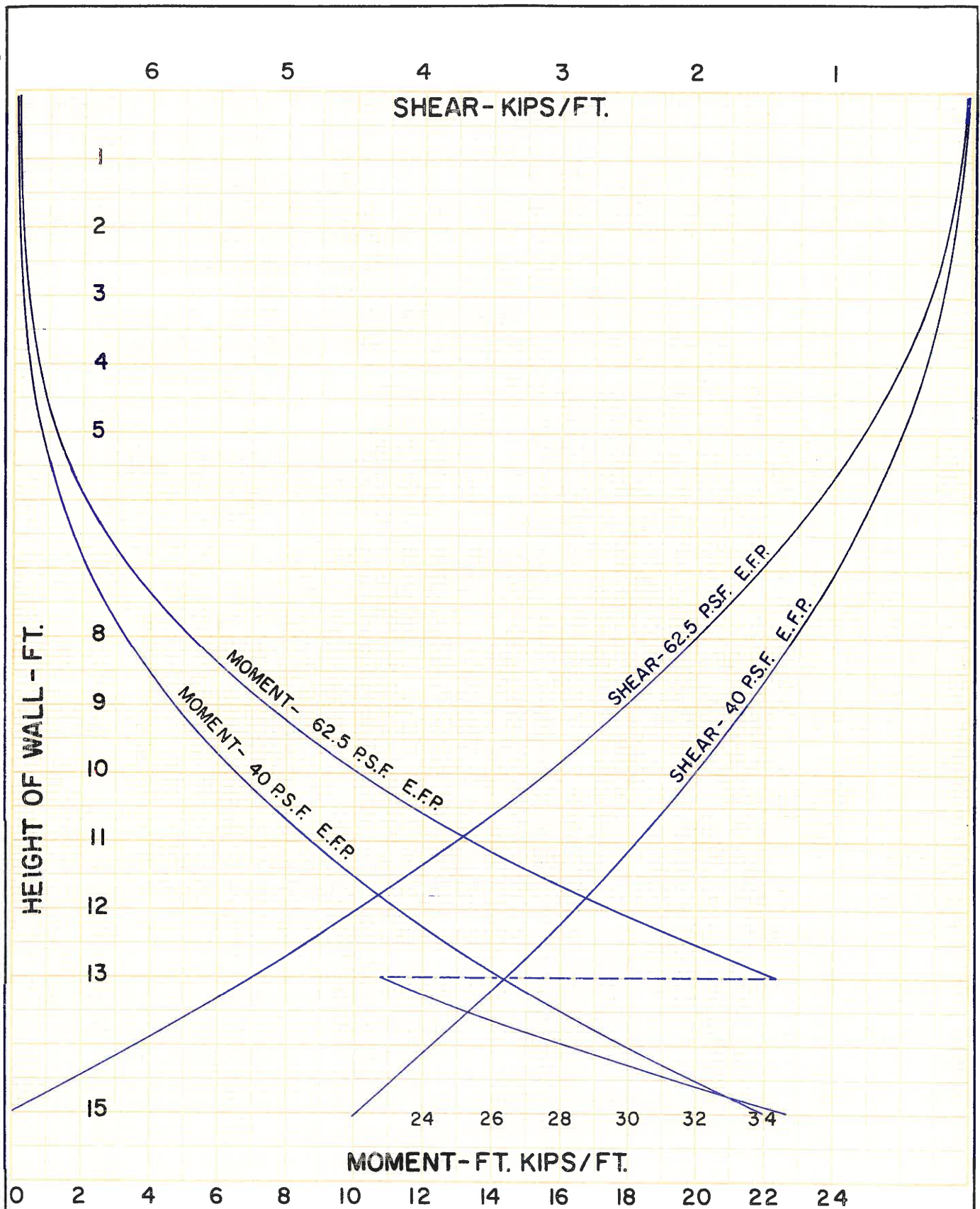
ORANGE COUNTY FLOOD CONTROL DISTRICT

MOMENTS AND SHEARS
FOR
CANTILEVER WALLS

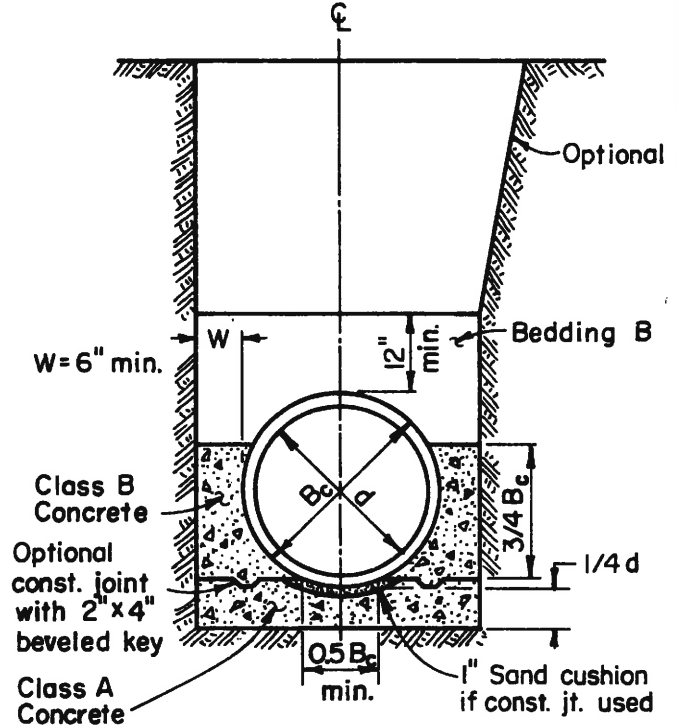| 1972 | 1 OF 1 | ST-32 |

GRANULAR BEDDING
LOAD FACTOR 1.8

W= 10" except State Hwys. which shall be 12"

W

12" min.

Optional

B_c

Bedding B to be placed in 2 lifts for $B_c > 60"$

1/12 B_c or 4" min.

0.5 B_c min.
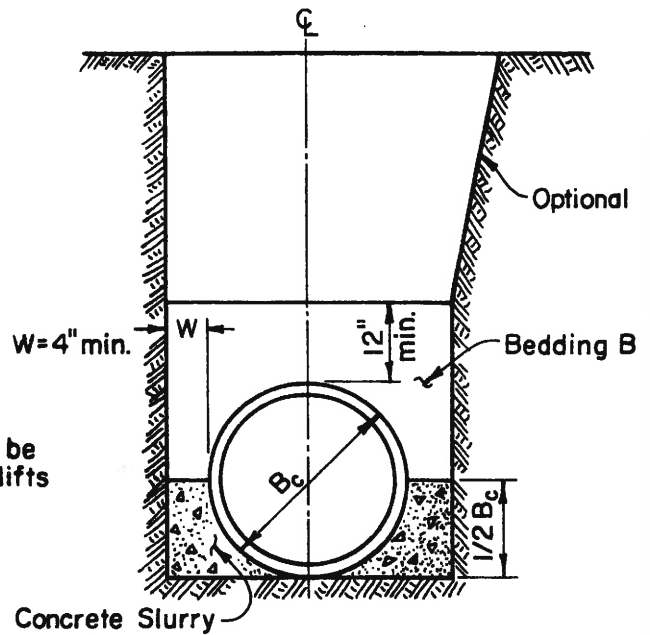
Bedding A

SLURRY BEDDING
LOAD FACTOR 2.4

W= 4" min.

W

12" min.

Optional

Bedding B

B_c

1/2 B_c

Concrete Slurry
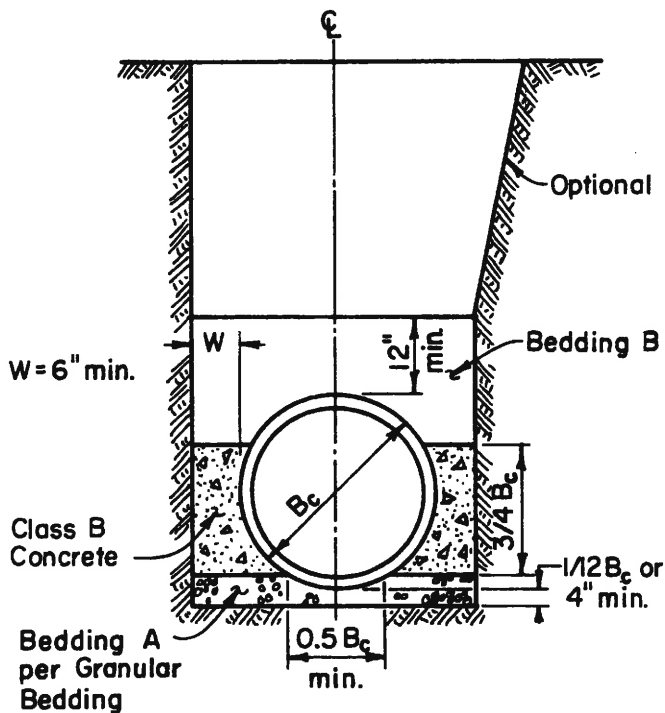
CONCRETE BEDDING (SIDES)
LOAD FACTOR 2.7

W= 6" min.

W

12" min.

Optional

Bedding B

B_c

3/4 B_c

Class B Concrete

1/12 B_c or 4" min.

0.5 B_c min.

Bedding A per Granular Bedding

CONCRETE BEDDING (CRADLE)
LOAD FACTOR 3.0

W= 6" min.

W

12" min.

Optional

Bedding B

Class B Concrete

B_c d

3/4 B_c

1/4 d

Optional const. joint with 2"x 4" beveled key

Class A Concrete

0.5 B_c min.

1" Sand cushion if const. jt. used

ORANGE COUNTY FLOOD CONTROL DISTRICT		
TYPES OF TRENCH BEDDING AND LOAD FACTORS		
1972	I OF I	ST-33

UNIFORM LOAD ON 180° TOP

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.1495	-.0530	0	+.1435	-.0400	0	+.1368	-.0268	0	+.1304	-.0132	0	+.1250	0	0
SIDE	-.1535	+.5000	+.0530	-.1465	+.5000	+.0400	-.1401	+.5000	+.0268	-.1327	+.5000	+.0132	-.1250	+.5000	0
INVERT	+.2935	+.0530	±.5000	+.1885	+.0400	0	+.1572	+.0268	0	+.1376	+.0132	0	+.1250	0	0

UNIFORM LOAD ON 90° TOP

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.1817	.0262	0	+.1757	-.0132	0	+.1690	0	0	+.1627	+.0136	0	+.1572	+.0269	0
SIDE	-.1683	+.5000	+.0262	-.1613	+.5000	+.0132	-.1549	+.5000	0	-.1475	+.5000	-.0136	-.1398	+.5000	-.0269
INVERT	+.3055	+.0262	±.5000	+.2005	+.0132	0	+.1690	0	0	+.1496	-.0136	0	+.1370	-.0269	0
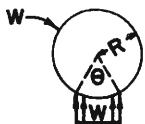
LOADING DUE TO WEIGHT OF RING

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.0796	-.0796	0	+.0736	-.0666	0	+.0669	-.0534	0	+.0606	-.0389	0	+.0551	-.0266	0
SIDE	-.0909	+.2500	+.0796	-.0839	+.2500	+.0667	-.0775	+.2500	+.0536	-.0701	+.2500	+.0399	-.0624	+.2500	-.0267
INVERT	+.2389	+.0796	±.5000	+.1339	+.0666	0	+.1025	+.0534	0	+.0829	+.0389	0	+.0704	+.0266	0

LOADING DUE TO WATER; PIPE FULL, ZERO PRESSURE HEAD ON SOFFIT

	Conc. Support at Invert			θ = 60°			θ = 90°			θ = 120°			θ = 180°		
	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v	C_m	C_n	C_v
TOP	+.0796	-.2389	0	+.0736	-.2257	0	+.0669	-.2124	0	+.0606	-.1991	0	+.0551	-.1859	0
SIDE	-.0909	-.0680	+.0797	-.0836	-.0680	+.0667	-.0775	-.0680	+.0532	-.0701	-.0680	+.0399	-.0624	-.0680	+.0267
INVERT	+.2389	-.3981	±.5000	+.1337	-.4109	0	+.1025	-.4243	0	+.0829	-.4379	0	+.0704	-.4511	0

UNIFORM LOAD ON SIDES

	C_m	C_n	C_v
TOP	-.1250	+.5000	0
SIDE	+.1250	0	0
INVERT	-.1250	±.5000	0
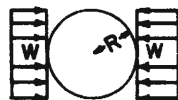
TRIANGULAR LOAD ON SIDES

	C_m	C_n	C_v
TOP	-.1042	+.3125	0
SIDE	+.1250	0	-.0625
INVERT	-.1458	+.6875	0

NOTES:
MOMENT = C · W · R
THRUST = C · W
SHEAR = C · W
W = TOTAL LOAD IN EACH CASE
R = MEAN RADIUS OF RING

SIGN CONVENTION:
+ M = TENSION ON INSIDE FACE
+ N = COMPRESSION
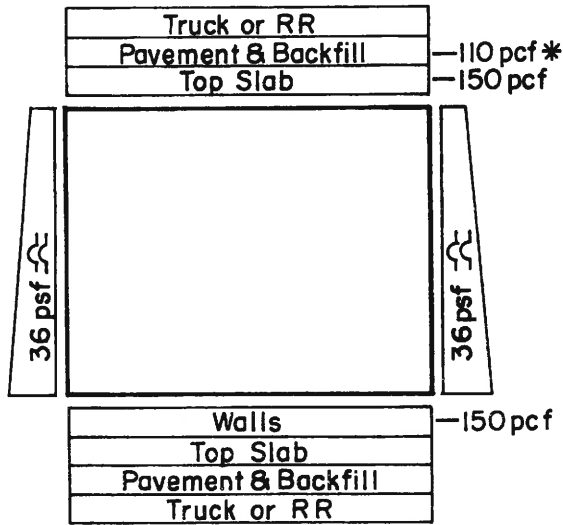+ V = SHEAR POSITIVE FOR LEFT SIDE

ORANGE COUNTY FLOOD CONTROL DISTRICT

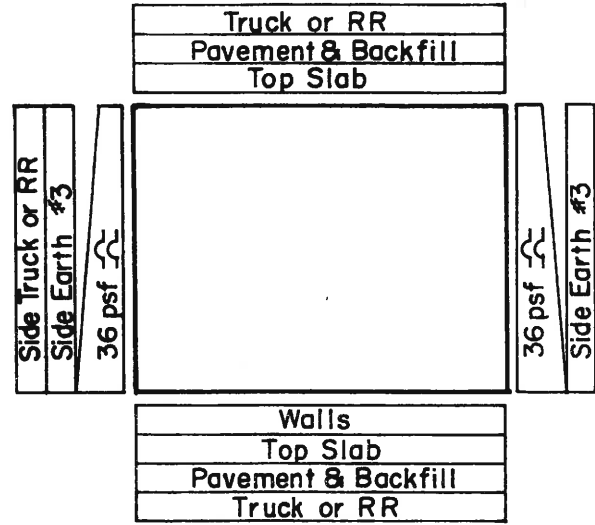## MOMENT, THRUST AND SHEAR COEFFICIENTS FOR ELASTIC RINGS
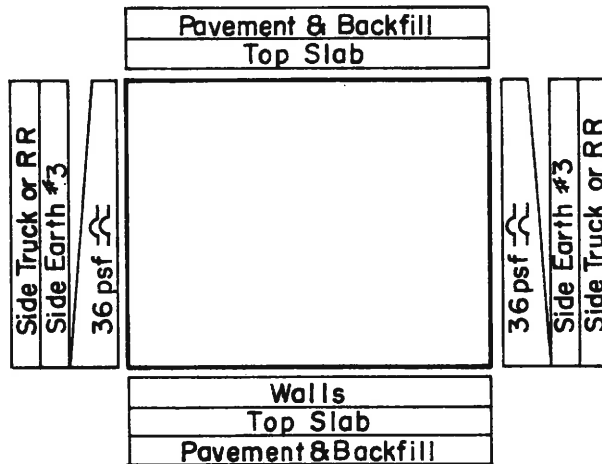
1972	I OF I	ST-34

CASE I

Truck or RR
Pavement & Backfill — 110 pcf *
Top Slab — 150 pcf

36 psf ≈ ≈ 36 psf

Walls — 150 pcf
Top Slab
Pavement & Backfill
Truck or RR

CASE I

MAX. (+) MOMENT, TOP AND INVERT SLABS
MAX. (−) MOMENT, SIDE WALLS

CASE II

Truck or RR
Pavement & Backfill
Top Slab

Side Truck or RR
Side Earth #3
36 psf ≈ ≈ 36 psf
Side Earth #3

Walls
Top Slab
Pavement & Backfill
Truck or RR

CASE II

MAX. (−) MOMENT CORNERS
MAX. SHEARS

CASE III

Pavement & Backfill
Top Slab

Side Truck or RR
Side Earth #3
36 psf ≈ ≈ 36 psf
Side Earth #3
Side Truck or RR

Walls
Top Slab
Pavement & Backfill

CASE III

MAX. (+) MOMENT, SIDE WALLS

CASE IV

Top Slab

Flooded Backfill ≈ ≈ Flooded Backfill
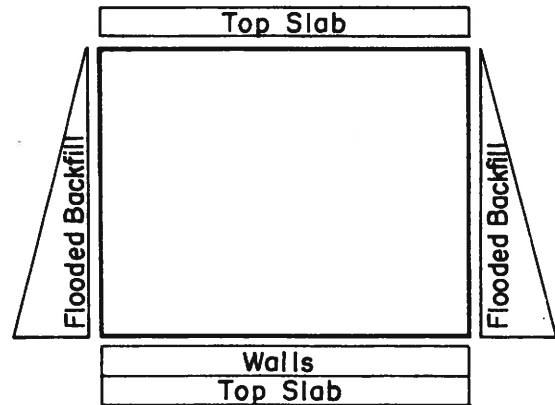
Walls
Top Slab

CASE IV

MAX. (+) MOMENT, SIDE WALLS

FLOODED BACKFILL
At 60 psf ≈ Normal Stresses

SPECIAL CONDITIONS – ALL BOXES

1. 0' − 2' Cover, treat as a bridge

2. Box under hydrostatic head.

3. Additional uniform side earth if indicated by soils report.

NOTE

(+) Indicates tension on inside of box.
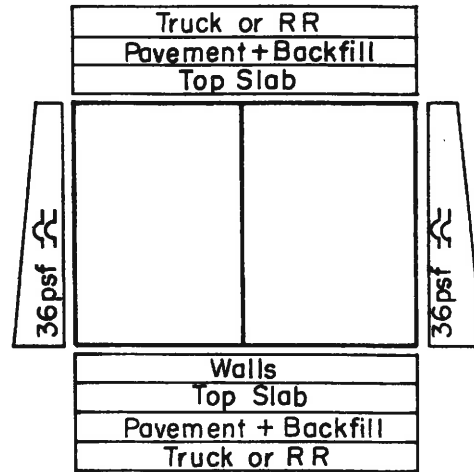(−) Indicates tension on inside of box.

* USING MARSTON FORMULA

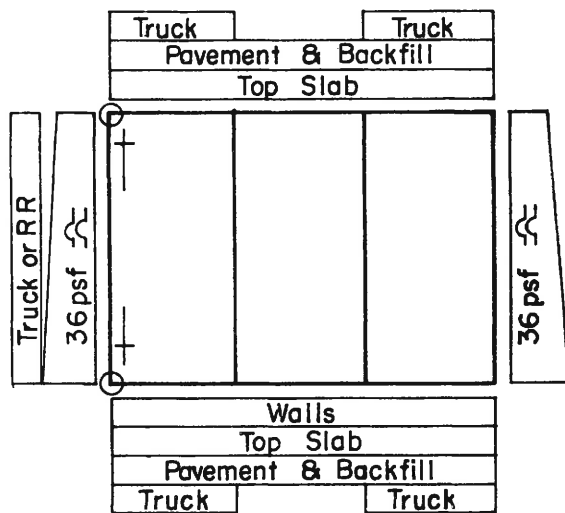| ORANGE COUNTY FLOOD CONTROL DISTRICT		
STANDARD LOADING CONDITIONS FOR DESIGN OF SINGLE BARREL BOX CONDUIT		
1972	I OF I	ST-35
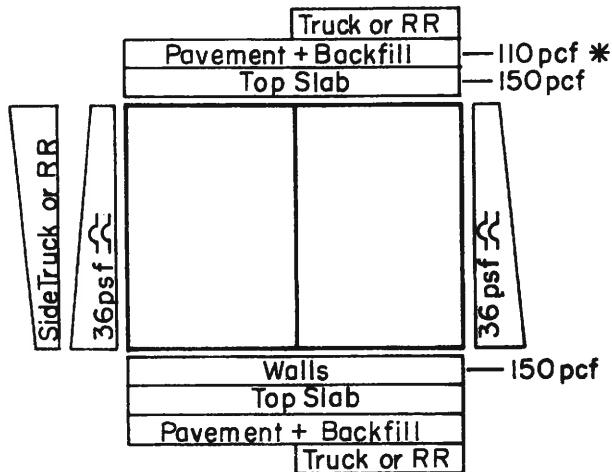
CASE I

Truck or RR
Pavement + Backfill — 110 pcf *
Top Slab — 150 pcf
Side Truck or RR
36 psf
36 psf
Walls — 150 pcf
Top Slab
Pavement + Backfill
Truck or RR

MAX.(+) MOMENT, TOP AND INVERT SLABS
MAX.(−) MOMENT, SIDE WALLS

CASE II

Truck or RR
Pavement + Backfill
Top Slab
36 psf
36 psf
Walls
Top Slab
Pavement + Backfill
Truck or RR

MAX.(−) MOMENT, TOP AND INVERT
SLABS AT CENTER WALLS

CASE III

Truck or RR
Pavement + Backfill
Top Slab
Side Truck or RR
Side Earth #3
36 psf
36 psf
Side Earth #3
Side Truck or RR
Walls
Top Slab
Pavement + Backfill
Truck or RR

MAX.(−)MOMENT, CORNERS
MAX. SHEARS

CASE IV

Pavement + Backfill
Top Slab
Side Truck or RR
Side Earth #3
36 psf
36 psf
Side Earth #3
Walls
Top Slab
Pavement + Backfill

MAX.(+) MOMENT, SIDE WALLS

CASE V

Top Slab
Flooded Backfill
Flooded Backfill
Walls
Top Slab

MAX.(+) MOMENT, SIDE WALLS

FLOODED BACKFILL

At 60 psf ~ Normal Stresses

SPECIAL CONDITIONS — ALL BOXES

1. 0'− 2' Cover, treat as a bridge.
2. Box under hydrostatic head.
3. Additional uniform side earth if indicated by soils report.

NOTE

(+) Indicates tension on inside of box.
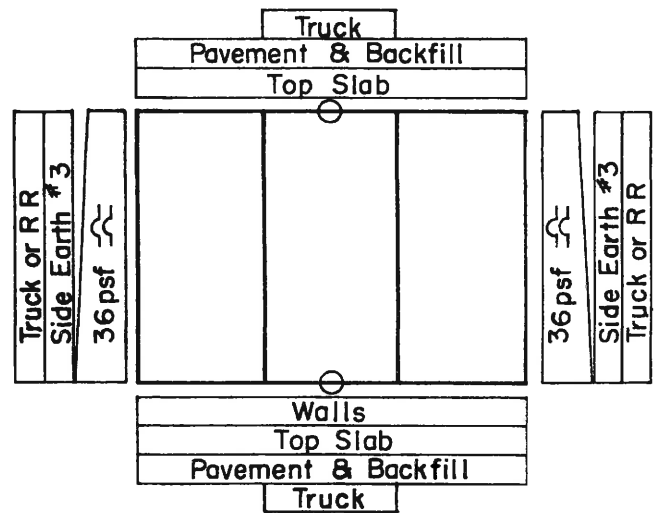
(−) Indicates tension on outside of box.

*USING MARSTON FORMULA

ORANGE COUNTY FLOOD CONTROL DISTRICT		
STANDARD LOADING CONDITIONS FOR DESIGN OF DOUBLE BARREL BOX CONDUIT		
1972	I OF I	ST-36

CASE I

Truck | Truck

Pavement & Backfill

Top Slab

Truck or RR | 36 psf | 36 psf

Walls

Top Slab

Pavement & Backfill

Truck | Truck

CASE I

MAX. (−) MOMENT, CORNERS
MAX. SHEAR, CORNERS

CASE II

Truck

Pavement & Backfill

Top Slab

Truck or RR | Side Earth #3 | 36 psf | 36 psf | Side Earth #3 | Truck or RR

Walls

Top Slab

Pavement & Backfill

Truck

CASE II

MAX. (+) MOMENT CENTER
BARREL, TOP AND INVERT SLABS

CASE III

Truck | Truck

Pavement & Backfill

Top Slab

36 psf | 36 psf

Walls

Top Slab

Pavement & Backfill

Truck | Truck
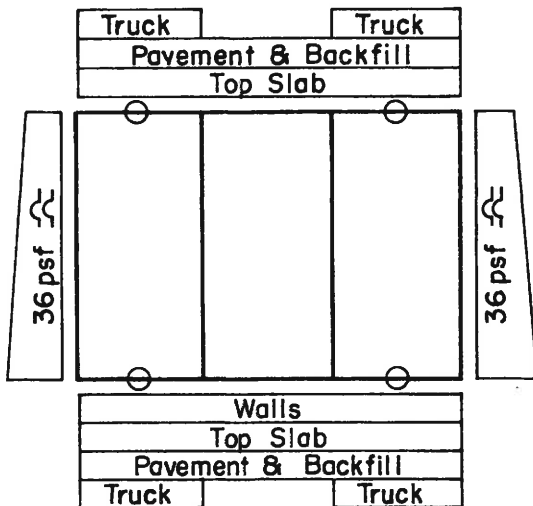
CASE III

MAX. (+) MOMENT OUTSIDE
BARRELS, TOP AND INVERT SLABS

CASE IV

Truck

Pavement & Backfill

Top Slab

36 psf | 36 psf | Side Earth 3

Walls

Top Slab

Pavement & Backfill

Truck
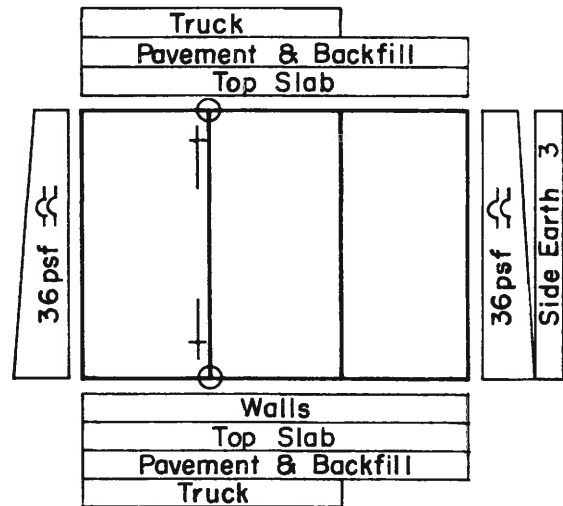
CASE IV

MAX. (−) MOMENT AT CENTERWALL
MAX. SHEAR AT CENTERWALL

NOTES

1. ○ Point of critical moment.
   † Point of critical shear.

2. (+) Indicates tension on inside of box.
   (−) Indicates tension on outside of box.

3. Additional uniform side earth if indicated by soils report.

See Detail A

Sym. About $\mathcal{C}_L$

NOTE: Steel Clearances Are Minimum

h_1

h

G-Bar

$1\frac{1}{2}$

T

$2''$

$1\frac{1}{2}$

B-Bars

C_1-Bar

Optional Construction Joint

4" or 6"

Distribution Bars

0.1W

0.2W

$1\frac{1}{2}$

$1\frac{1}{2}''$

v_1

$T_2/2$

C-Bar

H

D-Bar

Longitudinal Bars Equally Spaced

Construction Joint

GRADE LINE

4" to 12"

FLOW LINE

Distribution Bars

0.2 W

0.1W

C_2-Bar

C_3-Bar

v_2

v_3

3"

T_3

$\frac{3}{4}''$/Ft.

$2''$

B_1-Bars

Vee

$2''$

$2''$

H-Bar

h_2

h_3

T_2

W/2

TYPICAL SECTION

15 Diameters

2"

See Detail B ST-39

TRANSVERSE CONSTRUCTION JOINT DETAILS

B-Bars or B_1-Bars

B-Bars or B_1-Bars

Spacing in Table

DETAIL A

DESIGN DATA:

LOADS
Live loads
Dead loads
Depth of cover
Lateral E.F.P.
DESIGN METHOD
(WSD OR USD)
ALLOWABLE STRESSES

ORANGE COUNTY FLOOD CONTROL DISTRICT		
SINGLE BOX CONDUIT TYPICAL DETAILS		
1972	1 OF 2	ST-38

Sym. About ₵

NOTE: Steel Clearances
Are Minimum

G-Bar
G₁-Bar

#4 @18"

$1\frac{1}{2}$"

T_1

2"

4" or 6"

$T_2/2$

Optional
Construction
Joint

$1\frac{1}{2}$"

B-Bar
B₁-Bar

$1\frac{1}{2}$"

Dist. Bars   .25 W

v_1

$1\frac{1}{2}$"

C_1-Bar

H

GRADE LINE

3"

Construction
Joint

2"

Vee

4" to 12"

T_3

FLOW LINE

3/4"/Ft.

D-Bar

Longitudinal Bars
Equally Spaced

D-Bar

C-Bar

Roughened
Joint

2"

F-Bar
F₁-Bar

C_2-Bar

v_2 v_3

C_3-Bar

T_2 W T_4

2"

H₁-Bar
H-Bar

h_2

h_3

h

h_1

TYPICAL SECTION

15
Diameters

2"

See Detail B

TRANSVERSE CONSTRUCTION
JOINT DETAILS

1"

T

2/3 T

T/3 T/3

T/3

T/3

Chamfer, Channel
Face

DETAIL B

DESIGN DATA:
   See Single Box
   Conduit ST-38

ADDENDA

ADDENDUM NO. 1

Criteria: ☐ New ☐ Replacement ☒ Amendment

CHAPTER I

ITEM: Friction

The section on Friction Losses, pages 3, 4 and 5 of the July 1972 Orange County Flood Control District Design Manual has been revised to reflect more realistic conditions when the hydraulic radius "R" of a channel is 5 or greater.

The last paragraph on page 3 and its continuation at the top of page 4, which reads . . .

> "While most district projects fall within a relatively narrow range of hydraulic radii and adjustment of "n" is not usually required, for large improved channels the values of "n" tabled below may be inadequate. In channels where the hydraulic radius is 5 or greater the tabled values should be increased approximately 15 percent and rounded to the nearest thousandth."

Shall be changed to . . .

> "While most district projects fall within a relatively narrow range of hydraulic radii and adjustment of "n" is not usually required, for large improved channels the values of "n" tabled below may be inadequate. In channels where the hydraulic radius is 5 or greater the tabled value should be increased using the criteria below and rounded to the nearest thousandth."

R	"n"
5 to 10	Increase by 5%
10 to 15	Increase by 10%
15 or Greater	Increase by 15%

ADDENDUM NO. 2

Criteria: ☒ New ☐ Replacement ☐ Amendment

CHAPTER I

<u>ITEM</u>: Criteria For Retarding Basins

A.   POLICY

Retarding basins may be included as a component of the overall flood protection system, along with storm drains, channels, and flood plain management. It is the policy of the Orange County Flood Control District to include retarding basins as part of the master plan of a total watershed when the criteria set forth in this memo are satisfied.

B.   INTRODUCTION

This Addendum provides criteria for design of all retarding basins. For those retarding basins under the jurisdiction of the State Division of Safety of Dams (see Standard Plan 327, sheet 2, for definition of State jurisdiction), these criteria apply to the extent that State dam safety criteria do not control.

It is intended that the criteria not apply retroactively to any existing basin or the current planned Galivan, Edison, and Bartlett Park basins, except insofar as the criteria can be accommodated within the existing site and planned volumes.

In considering the feasibility of a retarding basin/channel system (hereinafter called retarding basin), it should be recognized that a retarding basin may not provide service equivalent to an all-channel alternative. Whereas an all-channel alternative provides significant benefits for floods of greater than its design capacity, a retarding basin loses effectiveness after it becomes full. Thus for floods exceeding the capacity of the retarding basin, damages for the retarding basin alternative may exceed damages for the all-channel alternative.

Furthermore, an all-channel system is designed for an estimate of peak flow rate whereas a retarding basin is designed for an estimate of both peak flow and

volume of flow: thus producing a degree of uncertainty not present in the all-channel alternative.

The sensitivity of the retarding basin alternative when subjected to a storm greater than its design storm suggests that the designer should be particularly concerned, in a poorly defined watercourse such as an alluvial fan (i.e., the coastal floodplain covering roughly half of Orange County) about excess flows escaping from the watershed and cascading to adjacent watersheds and compounding the anticipated damage. This is particularly important in the case of covered channels.

Proponents of a retarding basin alternative should be aware that the benefits of the retarding basin may diminish with distance downstream from the retarding basin. This reduction in benefits may be caused by the following factors:

1. The watershed downstream of the retarding basin, when analyzed separately, has a shorter time of concentration and higher intensity rainfall.

2. The reduction in discharge does not produce a proportionate reduction in new facility cost – i.e, within normal ranges of reduced discharge, the reduction produces a saving only in ROW cost and in invert cost (which is the least expense element of the channel), except for underground facilities or where the issue is replacement of existing facilities.

3. In some situations, the retarding of storm flow may increase the downstream discharge by contributing more to the other tributary inflow due to the later delivery of retarded flow.

All of the above suggests that a retarding basin alternative should not be selected over the all-channel alternative on the sole basis of being marginally less costly. Flood Control District experience suggests that retarding basins have their greatest feasibility when existing concrete-lined facilities downstream of the retarding basin site can be made adequate by the addition of retarding.

Considering the lesser degree of service that may be provided by the retarding basin, there should be a substantial cost advantage to the retarding basin before it should be considered for construction with Flood Control District funds. It is suggested that the minimum cost advantage for consideration of Flood Control District funding of a retarding basin should be a one-third reduction in cost as compared to the all-channel alternative. However, when the retarding basin is to be constructed by others, including assessment districts, and donated to the Flood Control District, the basin will be accepted provided it meets all the other criteria enumerated herein.

C.   LEVEL OF PROTECTION

The level of protection provided by a retarding basin should be designed to be equivalent to that provided by a channel. During a 100-year flood, the outflow or bypass flow from a retarding basin must be contained within the downstream channel with normal freeboard.

It is also intended that during floods greater than the 100-year flood, the protection of the community downstream from a retarding basin be equivalent to that which would have been provided by a 100-year channel alternative. The performance of the retarding basin and downstream channel shall be tested against the performance of a 100-year channel alternative using a 24-hour 1,000-year flood. Property damage under the retarding basin alternative must be no greater than that under the 100-year channel alternative. The 24-hour 1,000-year design flood will be computed in accordance with the numerical method outlined in the Orange County Hydrology Manual. Point rainfall for the 1,000-year return interval will be based on the amounts tabled for the various durations in the Department of Water Resources publication entitled "Rainfall Depth-Duration-Frequency for California".

For this test, a 100-year channel alternative shall be developed using normal channel design criteria. The 24-hour 1,000 year flood peak shall be computed and compared with the actual channel hydraulic capacity. If there is overflow from the channel, the overflow area shall be mapped and the resulting property damage shall be estimated using the highest level of development permitted under the General Plan or in the case of interim land use (i.e., agricultural or undeveloped land), future land use projected by reasonable planning procedures and assumptions and concurred by Orange County staff.

The 24-hour 1,000-year flood shall be routed through the retarding basin system. Any overflow from the downstream channel within the zone of influence of the retarding basin shall be mapped and any property damage shall be estimated using the same level of development used for the channel alternative. If the property damage under the retarding basin alternative exceeds that under the channel alternative, the retarding basin alternative as proposed will not be approved.

The retarding basin alternative may be approved if the retarding basin storage capacity and/or the outlet channel capacity are enlarged so that damages do not exceed those under the channel alternative. Damages may also be reduced by modifying development in the flood plain. If the 24-hour 1,000-year flood plain within the zone of influence of the retarding basin is undeveloped, the planned development may be revised to reduce flood damages. The grading, street pattern, land use, and density may be adjusted. Assurances satisfactory to the Flood Control District must be provided that permanently limit development to the approved plan. The assurances may take the form of easements, irrevocable

offers of dedication, deed restrictions, or other forms satisfactory to the Flood Control District.

D. LAND USE AND RIGHT-OF-WAY

Use of land for a retarding basin shall be consistent with the General Plan. Potential joint uses of the basin property shall be identified and design shall be compatible with reasonable joint uses. Right-of-way shall be obtained in fee unless a joint use is approved, prior to design, which suggests that a flood control easement is more desirable.

E. HYDROLOGY

A retarding basin shall be designed for a 100-year multiple-day design storm as prescribed by the Orange County Hydrology Manual. The critical storm duration shall be determined by adding preceding days of rainfall before the peak 24-hour rainfall until no increase in retarding basin volume or outlet requirements is obtained.

Design of the downstream channel may be dependent upon the hydraulics of the retarding basin and therefore shall be analyzed after sizing of the retarding basin features. Design of the downstream channel shall be based on analysis of two conditions given below. The condition which produces the larger peak discharge shall be used. Also, additional analyses may be required where the watershed or sub-area has an unusual shape, where there are multiple retarding basins or other special circumstances having timing impacts.

1. The design storm shall be applied over the entire watershed, during the modified Puls routing through the retarding basin and convex routing from the retarding basin to the concentration point of interest. The District may accept some other routing techniques such as Muskingum Routing only if the results are comparable to those obtained by convex routing. Downstream tributary discharge shall be added at the concentration point using depth-area-duration (DAD) factors for the watershed as a whole.

2. The design storm shall be applied only to the sub-area downstream of the retarding basin and no outflow from the retarding basin. Downstream tributary discharge shall be added at the concentration point using DAD factors for only the contributing watershed.

Retarding basin hydrology should also take into account the escape of floodwater from adjacent watersheds particularly in interim condition and should be sized to capture the escaped water without allowing escape to the next watershed.

Addendum No. 2

4

F.   DESIGN DETAILS

Outlet spillway design shall consider the consequences of failure. If there is no physical limitation to the flows which can enter the basin, the outlet/spillway shall be designed for not less than a 1,000-year flood based on the design storm computed as described in Section III. If there is a physical restriction to the amount of flow which can enter the basin, the spillway shall be designed for the maximum restricted flow, using a weir coefficient at the high end of the expected range or using a Mannings n at the low side of expected range in n. For the purpose of spillway design, the flood retarding volume (i.e., exclusive of water conservation, recreation pool, and debris) may be assumed to be empty at the beginning of the spillway flood if the outlet is ungated but should be assumed full if gated. The outlet spillway shall deliver its discharge to the downstream channel. If mitigation of the impacts of the spillway flood is required, the spillway should be designed in conjunction with additional volume above the spillway crest to retard spillway flows. If the configuration is such that basin overflow cannot be caused by inflow, an outlet spillway need not be provided.

The following criteria are maximum times allowed for depletion of temporary flood detention in situations not involving water conservation. These criteria should be used as rules-of-thumb suitable for preliminary design. Final design should start with the rules-of-thumb and optimize (by sensitivity analysis) cost of a larger release structure against its benefits (both to the detention facility and downstream).

1.   Facilities which must be emptied by pumping. Size pumps to discharge 80% of the design storage volume within 10-days.

2.   Facilities which empty by gravity flow.

     a.   Off-stream (by-pass). Discharge 80% of the design storage volume within 2 days.

     b.   On-stream (flow through). Provide a gated facility capable of discharge 80% of the design storage volume and the recession hydrograph within 2 days. For purpose of preliminary design, the capacity may be assumed to be 2 times the capacity needed to discharge 80% of the design storage volume without consideration of the recession hydrograph.

3.   Combination gravity and pumped facilities. Use above gravity criteria for gravity portion and pumped criteria for pumped portion.

Cut slopes or embankment slopes shall consider the identified potential joint uses. Where a park is a potential joint use, slopes shall not be steeper than 5:1 unless approved by the park agency. Slopes shall be stable during the maximum credible earthquake as determined by the geotechnical report for the site.

Addendum No. 2

5

Among other factors, the geotechnical report shall consider liquefaction and the "sudden drawdown" condition. In no event shall slopes be steeper than 2 to 1.

The minimum slope of a basin floor shall be 1 percent to a paved drain. Paved drains shall have slopes a minimum of 0.5 percent.

A berm shall be provided around the entire periphery of the basin with a minimum width of 15 feet, a minimum centerline curve radius of 45 feet, all-weather surface, and with one paved access ramp to the nearest street and to the invert of the basin with a maximum slope of 10 percent. In cases where a weir interrupts the continuity of the berm, the roadway shall provide a 50' x 50' turnaround or a cul-de-sac of 35 foot radius turnaround at each side of the weir or shall bridge the weir. A further description of roadway requirements can be found in Design Manual Addendum No. 4 (Public Works Maintenance Requirements).

Either weir or gates may be used for the inlet control. The inlet structure design shall be based upon routing the design flood through the inlet and basin or shall be sized to deliver peak discharge with the basin one-half full. If a weir is proposed, the weir elevation and length shall be optimized considering: need to limit premature spill to the basin, relationship of weir length to basin volume, and cost. The weir shall either be designed for stable subcritical flow in the feeder channel or the design must be verified by an approved hydraulic model. Gates may be applicable where basin volume is limited and must be conserved by preventing premature spill to the basin, where close downstream control is required (i.e., the gates are controlled by downstream water surface), and where sediment and debris are not problems. If gates are proposed, standby power and telemetry of gate, basin and channel status shall be provided and control shall have one level of redundancy. The basin volume shall include consideration of the uncertainty in estimating weir and gate flow rates, or else the inlet design must be verified by a mathematical, and, if necessary, a hydraulic model.

Retarding basin proposals for flow-through facilities shall include a sediment analysis and the design of the retarding basin shall provide sufficient additional volume to store the sediment expected to be deposited in the basin in a 100-year storm. Bypass channel low flows shall be confined to a pilot channel or vee to facilitate sediment movement.

Because of the uncertainties in design flood volume estimation, and estimating rating of inlet structures, confining levees shall include two feet of freeboard over the water surface at outlet spillway surcharge unless the spillway is designed for the Probable Maximum Flood, in which case no freeboard is required.

Except where a joint-use partner is responsible for security, the basin site shall be fenced with chain-link fence in accordance with Standard Plan 600-0-OC.

Addendum No. 2

An approved landscape plan or erosion control planting plan shall be prepared for outside berms and slopes. The basin interior berms, berm slopes, and basin floor shall be bare earth unless special arrangements are made for maintenance as provided in Section VII.

Groundwater conditions shall be evaluated. The basin invert shall not be lower than the estimated average high groundwater table. The need for subdrains to stabilize the basin invert shall be investigated. Lowering of the regional groundwater table outside of the boundary of the basin site shall not be permitted without a special study of the impacts of the changed groundwater table.

The inlet and outlet spillways shall be suitably protected with cutoff walls and rock to prevent erosion.

G.   MAINTENANCE COST ANNUITY

Unless a joint-use partner is responsible for maintenance, the basin interior berms, berm slopes, and basin floor shall be bare earth. If the basin is proposed to be dedicated to the Flood Control District, a maintenance cost analysis shall be performed of the retarding basin alternative as compared to an all-channel alternative using historical cost data representative of the basin and approved by the District. In the analysis, any savings in future OCFCD downstream maintenance costs will be credited against the present worth of the basin maintenance costs prior to determining the annuity. If the analysis shows that the retarding basin alternative is more costly to maintain than the all-channel alternative, either the present worth of additional maintenance costs over the project life shall be estimated using 3% interest rate, and a cash deposit or annuity donated to the Flood Control District to cover the additional costs; or annual payments shall be provided by agreement backed by adequate surety.

H.   WATER CONSERVATION

A geotechnical/geohydrologic report shall be prepared on the feasibility of water conservation. If water conservation is economically feasible, an additional basin volume, established by an economic analysis, shall be provided and any additional facilities needed for diversion of storm flows or low flows. In the absence of an analysis, volume shall be increased by 20%.

I.   OPERATING MANUAL

An operating manual describing all features of basin operation, including rating curves of all facilities, an elevation-capacity curve for the basin, instruction manuals for all equipment, and a maintenance schedule shall be provided.

Addendum No. 2

J.   ADDITIONAL CRITERIA

If any criteria are proposed that are not in accordance with this Addendum or are in addition to the criteria in this Addendum, they shall be clearly identified in a preliminary submittal, the reason for their proposed use set forth, and a sensitivity analysis made of the impact of variation in the criteria.

K.   PHASING

Where a retarding basin is included in an approved master plan, and a developer proposes to construct facilities downstream from the basin, either the basin shall be constructed concurrently, or the retarding basin right-of-way and construction shall be assured by an agreement legally enforceable on the land or other surety, or the downstream facilities shall be designed and constructed on the basis of unretarded Qs.

ADDENDUM NO. 3

Criteria: ☒ **New** ☐ **Replacement** ☐ **Amendment**

CHAPTER I

<u>ITEM</u>: Freeboard

A. **PURPOSE**

Freeboard is provided to insure that the desired degree of protection will not be reduced by unaccounted for factors which affect channel hydraulics but which are not required to be specifically analyzed in design. These factors include but are not limited to: variations in Manning's n with channel bottom conditions, uncertainties in the selection of Manning's n, variation in stage-discharge relationships, variation in velocity from average velocity, sedimentation, debris, bulking, and air entrainment. When any of the above factors are expected to be significant, its effect shall be separately estimated.

B. **DEFINITION**

Freeboard is the vertical distance from the design hydraulic-grade-line (plus wave height, super-elevation, and any other factors required to be separately evaluated):

1. to the top of levee in ultimate unlined earth levee channels.

2. to the top of channel in the ultimate unlined earth channels.

3. to the top of wall engineered channel lining (examples: riprap, concrete, sheet piles, etc.).

4. to the soffit where box-conduits or culverts are designed as open channels.

5. to the low point of the soffit of bridges.

Under some supercritical conditions freeboard may be required above conjugate depth –refer to Other Considerations – Stable depth.

C. MINIMUM VALUES

The Minimum Values addressed herein shall comply with the current Federal Emergency Management Agency (FEMA) Regulations for levees.

The following are minimum acceptable freeboard for Froude number below 0.90 (for larger Froude numbers refer to Other Considerations – Stable depth):

1. Leveed channels and flood-walls*. Levees or floodwalls shall be used only when there is no reasonable alternative conveyance section. "Reasonable" shall be determined by an analysis of alternatives approved by the Engineer. The magnitude of freeboard for levees and flood walls shall consider the degree of hazard to the protected area. When levees or flood walls are used, the following minimum criteria shall apply.

    a. Two feet of freeboard shall be provided for facilities with the water surface not more than 2-feet above surrounding ground.

    b. Where the water surface is more than 2 ft. above surrounding ground, a minimum freeboard of 3 ft. above the water surface level of the design discharge shall be provided. An additional 1 ft. is required within 100-ft on either side of structures (such as bridges) riverward of the levee. An additional 1½ ft. above the minimum at the upstream end of the levee, tapering over 500 ft. to not less than the minimum is also required.

    c. For super-critical flow, refer to Other Considerations – Stable depth.

    d. For levees subject to tidal conditions, the freeboard shall be established one-foot above the height determined by a special study which shall consider all factors affecting water surface, including but not limited to astronomical tides, wave set-up, wave run-up (if applicable), storm surge, and tsunamis. Tsunamis shall be considered as an independent event unrelated to the other factors.

    * A flood wall is a wall, in lieu of a levee, which projects above the surrounding ground for the purpose of conveying flood waters.

Addendum No. 3

2. Non-leveed channels with design frequency of 100-years – 1.5 ft.

3. Non-leveed channels with a design frequency of less than 100-years.

    a. drainage area between 500 acres and 4,000 acres – 1.0 ft.

    b. drainage area less than 500 acres – 0.5 ft.

D. OTHER CONSIDERATIONS

1. Overtopping

Flows exceeding the design discharge shall be considered and the design shall include measures to assure the least hazardous (damaging) initial overtopping location or to assure initial overtopping of levee on least hazardous (damaging) side. Future land use shall be considered and whenever possible, the overflow location should be into parks, wetlands, flood-plain easements or streets.

2. Hydraulic gradeline

Freeboard may also be determined by local drainage considerations. Unless pumping stations are an economical alternative for tributary drainage, the design hydraulic gradeline in the main trunk shall be sufficiently below surrounding ground to accommodate local drainage.

Where an entire tributary system has been designed, including all inlets, a design hydraulic-gradeline at least 0.5 ft. below street gutter grade shall be provided within catch-basins. In all other cases, or in preliminary studies which do not consider inlet hydraulics, the design hydraulic-gradeline of the conveyance facility shall be at least two feet below street gutter grade. Design in undeveloped areas shall consider the possibility that future street elevations may be one or more (usually 2) feet below average existing ground.

3. Alluvial fans

On alluvial fans where overflow may escape (cascade) to an adjoining watershed, a minimum of one-foot additional freeboard shall be provided.

Addendum No. 3

3

4.   Stable depths and supercritical flow

Stable depths (Froude number below 0.90 or above 1.20) shall be provided. Where a channel must be designed for Froude number between 1.00 and 1.20 or where piers or confluences introduce potential instability, freeboard shall be added to the conjugate depth. For leveed channels, freeboard shall be added to conjugate depth when Froude number is between 1.00 and 2.00. For non-leveed channels with Froude number greater than 1.2, the freeboard shall be as follows:

design frequency of 100-yrs.	3.0 ft.
drainage area between 500 acres and 4000 acres	2.0 ft.
drainage area less than 500 acres	1.0 ft.

5.   Exceptions

A request for less than the minimum specified herein may be submitted to the Engineer for approval. The request shall include appropriate engineering analysis demonstrating adequate freeboard. The engineering analysis shall address those factors listed in paragraph A, Purpose. Under no circumstances will freeboard less than two feet be accepted for levees and floodwalls. In the case of coastal levees no exceptions will be made because Minimum Values requires a special study.

ADDENDUM NO. 4

ISSUED: **January 6, 1995**

Criteria: ☒ **New** ☐ **Replacement** ☐ **Amendment**

CHAPTER II

ITEM: Public Works Maintenance Requirements

A. **CHANNEL, PERIMETER AND ACCESS ROADS**

1. "Channel road" includes all roads along channels, "perimeter road" includes all roads around retarding basins, "access road" includes short access to outlets, inlets, and pump stations.

2. A 14 ft. wide all-weather road, aggregate base, or equal shall be located adjacent to the channel within a 20 ft. horizontal clear area.

3. All roads shall be continuous where practical. Where a continuous road is not practical and a turnaround is authorized by Manager, Public Works Operations, a 50 ft. x 50 ft. turnaround or a cul-de-sac of R=35 ft. shall be provided.

4. For perimeter and access roads, the minimum radius horizontal curve shall be R=35 ft. (for inside edge of track). Geometric standards for channel roads shall be in accord with the Orange County Highway Design Manual with a design speed of 20 mph.

5. A minimum 12 ft. clear road width shall be provided adjacent to and around access ramps.

6. For leveed conditions, satisfactory back-slope stabilization shall be used to control erosion and sloughing. Such stabilization may include but not be limited to walls, slope flattening, slope lining, drainage devices, landscaping, etc.

7. Road shall be provided on both sides for a channel with a top width greater than 30 ft. For a channel with a top width of 30 ft. or less, a 5 ft. walk path may be used on one side in lieu of one of the access roadways.

Addendum No. 4

1

8. The road and walk shall slope toward the channel at 2%. If an exception is approved by Manager, Public Works Operations, to slope away from channel, a drainage collection system shall be provided to control storm flow entry into the channel.

9. Where channel maintenance roads are designated to be paved, the structural section shall be determined by the Orange County Materials Engineer. The minimum paved width shall be 10 ft. A minimum 2 ft. wide all-weather shoulder measured from edge of pavement shall be provided. Where the sideslope adjacent to the graded shoulder is steeper than 4:1, the downslope side shall have a 4 ft. wide graded area measured from edge of pavement to hinge point. A minimum 2 ft. horizontal clearance to obstructions shall be provided adjacent to the pavement. The graded shoulders adjacent to the pavement shall slope away from the pavement at 2% minimum, 5% maximum.

10. Channel maintenance roads designated to be paved with asphalt concrete will meet the minimum requirements for horizontal curves and sag and crest vertical curves as specified in CALTRANS Highway Design Manual and based upon a 20 mph design speed.

11. Channel maintenance roads approved by Manager, Design Division, to be surfaced with decomposed granite shall be surfaced with a minimum of 2 in. of D.G. (Per Section 400-2.3 of Standard Specifications for Public Works Construction) over aggregate base as determined by the Orange County Materials Engineer.

B. **ACCESS RAMPS TO CHANNEL INVERT**

1. Ramps shall have a maximum 10% grade and should slope downstream. Ramps shall have a minimum of 2% cross-slope toward channel.

2. For a vertical-wall channel, the channel wall shall extend 1 foot above the ramp to form a 1-foot high curb. This curb shall transition to zero at the invert gradeline within a distance of five feet.

3. Access ramp width, excluding walls and curbs, shall be a minimum of 12 feet.

4. A 30-foot longitudinal landing pad shall be provided at the base of the ramp. The wall transition shall have an angle of 45 degrees, starting at the end of the 30 ft. landing pad (see Exhibit 1 of 2).

5. Ramps shall be located as close as possible to cross streets, but at least 50 feet from the street right-of-way. Where a ramp is located within 100

feet of a cross-street, the area from the street entry to the beginning of the access ramp shall be paved with a road structural section (.35 ft. AC/.50 ft. AB or equivalent).

6. Ramps shall be installed as needed to provide continuous access to the invert but in no case shall ramp spacing be greater than 1 mile. Access to the invert is normally required for, but not limited to, the following:

    a. Retarding basins
    b. Between grade stabilizers
    c. Between drop structures
    d. Where access under bridges, culverts, etc., is restricted (less than 12 ft. vertical clearance)
    e. All channels with a bottom width greater than 12 feet.

7. Ramps shall be Portland cement concrete and the ramp surface shall have a transverse raked finish for traction.

8. Storm drains shall not outlet onto the ramp surface.

C. ENTRANCES AND GATES

1. Minimum 16-foot opening, double swing gates shall be provided in accordance with EMA Standard Plan 600-0-OC. If required by Chief Engineer, Public Works, in addition to the 16-foot gate, a 5-foot un-gated opening will be provided; or as an alternative, the 16 ft. gate may be replaced with bollards.

2. Gates shall be set back a minimum of 20 feet from the street right-of-way.

3. A 14-foot flared depressed curb driveway approach shall be provided in accordance with EMA Standard Plan 1210.

4. Road structural section (.35 ft. AC/.50 ft. AB or equivalent) from drive approach to gate shall be provided and shall include provisions for adequate drainage.

5. Gates shall open inward (toward channel road).

6. Access onto channel maintenance roads shall be provided at all street crossings.

7. Where the intersecting street contains a curbed median and where no undercrossing is provided, a 12 ft. wide opening in the street median shall be required to facilitate crossing the street.

Addendum No. 4

D. **UNDERCROSSINGS**

Where the Chief Engineer determines that an undercrossing is necessary for circulation of flood control maintenance vehicles, the following criteria apply.

1. Minimum width shall be 16 ft.

2. The minimum clearance below soffit of bridge shall be 12 ft.

3. Roads shall be a minimum of 2.0 ft. above channel gradeline.

4. Portland cement concrete with a transverse raked finish shall be used for roadways under bridges.

5. CALTRANS warning signs posting height of clearance under bridges shall be installed.

6. Crest vertical curves of 110 ft. and sag vertical curves of 100 ft. shall be provided. Horizontal curves based upon a minimum 20 mph design speed shall be provided per CALTRANS Highway Design Manual and the Orange County Highway Design Manual.

7. Grades shall not exceed 7%.

E. **ACCESS OPENINGS FOR STRUCTURAL BOX CULVERTS**

1. Openings shall be 20 ft. minimum in length and the entire width of the structure.

2. Spacing between access openings shall be between 0.5mi. to 1.0 mi.

3. Openings shall be placed outside the public roadway and pedestrian travel way or within the raised median area. A minimum of 3 ft. clearance shall be provided from an above-ground opening to the curb face if used in a raised median condition.

4. H-20 loading shall be provided for grate covers on openings which project less than 30 inches above finished grade. For grate covers projecting 30 inches or greater, 10,000 lbs. loaded pick-up truck loading condition may be used in lieu of H-20 loading.

5. The maximum weight of any portion of the grate cover shall not exceed 6,000 lbs.

6.   Covers shall have lifting hooks or eyes for use by a crane in maintenance operations.
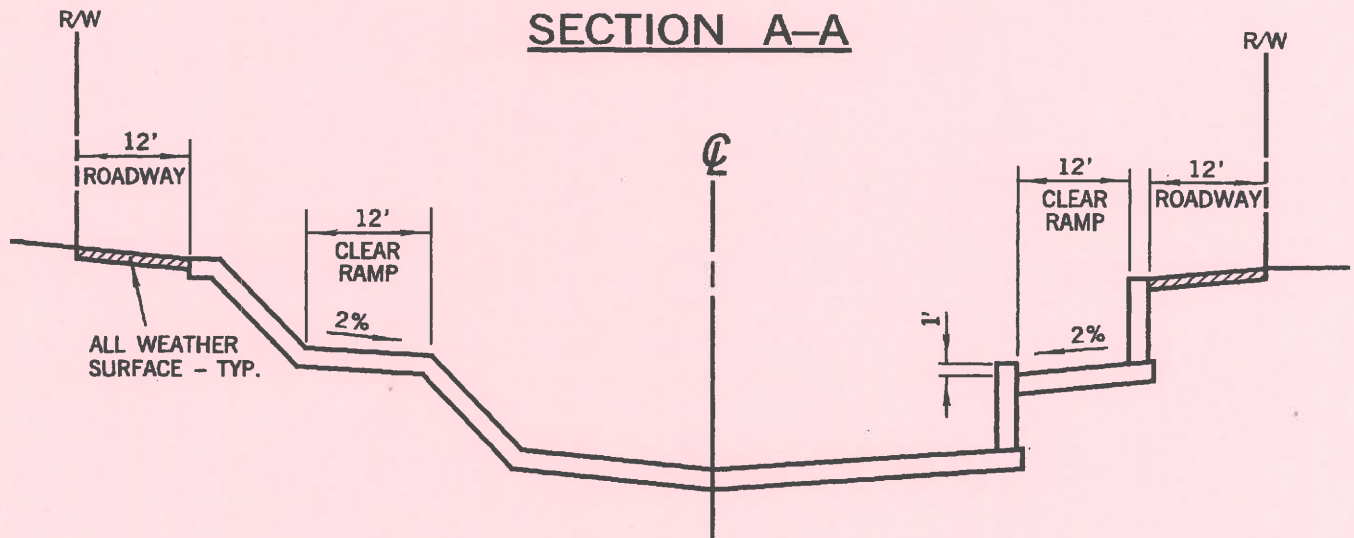
F.   ROCK RIPRAP

1.   Shall be placed a minimum of 25 ft. upstream and downstream of structures related to main channel flow. Drop structures shall require an independent design. Additional riprap may be required depending on flow and channel characteristics.

2.   Shall be placed where wave action, angle of attack or flow velocities may erode side slopes. Size, shape and location of riprap shall require independent design.

ALL DIMENSIONS SHOWN ARE MINIMUM

MAJOR OBSTACLE
FREEWAY, RAILROAD,
ETC. WHEN AN
UNDERCROSSING CAN
NOT BE CONSTRUCTED

R/W

50'

50'
TURNAROUND

FLOW

R/W

20'
CLEAR AREA

A

A

14' ALL
WEATHER
ROADWAY

20'
CLEAR AREA

CL

45°

12'
CLEAR

5' CURB TRANSITION
TO 0' HEIGHT AT
INVERT GRADE LINE

B

12'
CLEAR

2%
MIN.

B

30'
LANDING

30'
LANDING

10%
MAX.

10%
MAX.

2%
MIN.

2%
MIN.

VARIES

12'
CLEAR

12'
CLEAR

AROUND RAMPS TYP.

12'
CLEAR

12'
CLEAR

R/W

50'

R/W

R/W

STREET R/W

R/W

R/W

2'
TYP.

16' WIDE DOUBLE
SWING GATE

.35' AC/.50' AB FROM
ROADWAY TO GATE
(BEYOND GATE SEE PAR. B5)

20' TO
STREET R/W

14'

14' FLARED PCC DRIVE
APPROACH (STD. PLAN 1210)

DRIVE APPROACH AREA

ORANGE COUNTY FLOOD CONTROL DISTRICT

PUBLIC WORKS MAINTENANCE
REQUIREMENTS

| 1995 | 1 OF 2 | ADDENDUM NO. 4 |

R/W

20'
CLEAR AREA

TOP WIDTH

20'
CLEAR AREA

R/W

14'
ROADWAY

₵

14'
ROADWAY

2%

2%

ALL WEATHER
SURFACE – TYP.

GRADE LINE

BACK SLOPE
PROTECTION

SECTION A–A

R/W

12'
ROADWAY

₵

R/W

12'
CLEAR RAMP

12'
ROADWAY

12'
CLEAR
RAMP

2%

ALL WEATHER
SURFACE – TYP.

1'

2%

SECTION B–B

CALTRANS WARNING SIGN
POSTING HEIGHT OF CLEARANCE

16'

2%

12'
CLEAR

2' MIN.

GRADE LINE

BRIDGE UNDERCROSSINGS

ORANGE COUNTY FLOOD CONTROL DISTRICT		
PUBLIC WORKS MAINTENANCE REQUIREMENTS		
1995	2 OF 2	ADDENDUM NO. 4

ADDENDUM NO. 5

Criteria: ☒ **New** ☐ **Replacement** ☐ **Amendment**

CHAPTER I

ITEM: Hydraulic Criteria for Determination of Water Surface Profile

It is the policy of Orange County Flood Control District and the Public Facilities and Resources Department that the hydraulic design of all prismatically shaped flood control facilities (with the exception of circular and semicircular pipe conduits) shall neglect channel invert cross-fall in the determination of water surface profiles and hydraulic gradeline elevations.

All hydraulic design shall be based on gradeline elevation control not flowline elevations unless an exception is specifically authorized in writing by the Chief Engineer, Public Facilities and Resources Department, or his designee. See attached figures on next page.
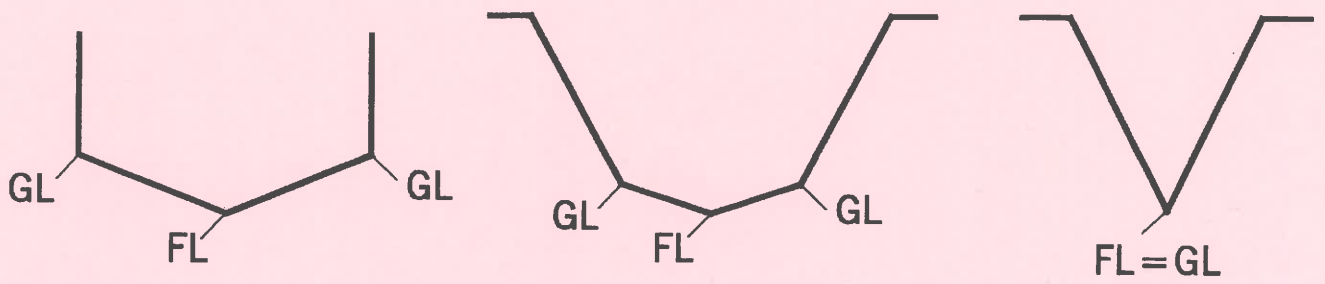
Addendum No. 5

Legend:

GL: Gradeline

FL: Flowline

ORANGE COUNTY FLOOD CONTROL DISTRICT

HYDRAULIC CRITERIA FOR DETERMINATION OF WATER SURFACE PROFILE

| 1997 | 1 OF 1 | ADDENDUM NO. 5 |

DESIGN DISCHARGE FOR NEW FACILITIES

INTRODUCTION

The 1986 Hydrology Manual specifies hydrologic methods and was not intended to establish design discharges. The purpose of this addendum to the Design Manual is to establish the discharge rate for the design of new EMA flood control facilities, hereinafter called the design discharge.

This addendum describes the 1973 Hydrology Manual criteria, which uses a 10 and 25-year design discharge in small watersheds, and explains why this criteria will provide 100-year protection to structures. Short-comings of the 1973 Hydrology Manual criteria are discussed. A design discharge table is presented which simulates the 1973 criteria but describes all design discharges in terms of Q-100 determined by the 1986 Hydrology Manual.

CURRENT CRITERIA

EMA's goals are to provide 100-year flood protection for habitable structures (Public Facilities Element of the General Plan) and provide useable streets and highways at 10-year floods as described in Addendum #1 to the EMA Design Manual (attached).

The 1986 Hydrology Manual specifies hydrologic methods that are to be used to determine discharges but does not specify what discharge is to be used for design. Since the 1986 Hydrology Manual does not specify design discharges, that portion of the 1973 Hydrology Manual which specifies design discharges remains as the currently effective EMA policy on design discharge.

The 1973 Hydrology Manual requires storm drains in small watersheds to be designed for a 10-year Rational Method Q; as the watershed becomes larger, a 25-year Rational Method Q is used for design; and in the lower portion a 100-year Unit Hydrograph Q is used for design. The design engineer may be required to test the 10 and 25-year designs to verify that they meet the desired 100-year protection goal for habitable structures. The criteria for design discharge used in the 1973 Hydrology Manual are as follows:

Design Discharge	Area
10-year Rational Method	less than 500 acres
25-year Rational Method	500 to 4000 acres
100-year Unit Hydrograph	4000 acres and larger

Although the 1973 Hydrology Manual specifies that the design discharge is to be applied to "open and underground channels and storm drains" practice has been to assume that the design discharge determined in accordance with the manual is the design discharge for the combined capacity of streets plus storm drain. The design discharge is carried entirely in the street until such location as the street capacity is exceeded. At that point, a storm drain is necessary for the excess with a minimum allowable storm drain diameter being 18".

Downstream from that point, the design discharge is carried by the combined capacity of street and storm drain. At some location further downstream, the street capacity becomes insignificant relative to the pipe and the entire design discharge is carried by the pipe.

The logic for the 1973 Hydrology Manual criteria is explained on Page 10 of the 1973 Hydrology Manual as follows: "First, consider less than 500-acre portion of the upper end of the drainage area. Assume this portion is served by underground storm drains designed to carry the peak flow indicated by the use of a 10-year frequency rainfall curve with the Rational Method. These storm drains will operate with water surfaces below street grade with a 10-year frequency storm and will discharge freely into a larger regional collection system. The occurence of a larger storm on the drainage area will produce run-off at a rate that can be accomodated by temporary storage and localized flooding, larger storm drain discharge by reason of greater head and escape by street or other flow into other drainage areas or into the regional collection system. A system designed to limit storage elevations to non-damaging heights (one-foot below improvements) would be the goal in this case."

The 1973 Hydrology Manual suggests that use of the Rational Method with "drainage areas up to 100 acres gives results approximately correct" and that the Rational Method may over-predict as the watershed increases and "when the tributary area has grown to 4000 acres the error has reached a magnitude that peak flows comparable with those obtained for a 100-year flood are common." The 1973 Hydrology Manual also points out an additional consideration in setting criteria: "the importance of the channel under consideration to the area being protected." "For example, should a 10-year storm drain be over-taxed, the quantity of flow and duration of flow is small and fewer actual structures are involved. However, for a 100-year channel, thousands and possibly tens of thousands of acres are involved which generally include emergency facilities, transportation facilities, commercial establishments and industry as well as residences."

The 1973 manual states that it "...will generally provide 100-year flood protection to structures in developments meeting current grading requirements even in small drainage areas where less than 100-year frequency criterion is employed for channel and storm drain design."

The 1973 Hydrology Manual criteria have been in use for many years (the 1973 Hydrology Manual merely used what had already been in use for many years) with no significant flooding problems in small Orange County watersheds.

SHORTCOMINGS OF THE 1973 MANUAL DESIGN DISCHARGE CRITERIA

There are several problems with the 1973 Hydrology Manual critera for design discharge:

- There is no clear relationship between the three different frequency Q's that are used.

- The relationship of design discharge to the EMA goal of 100-year protection is easily misunderstood. Only the most sophisticated hydraulic engineers understand that a 10-year or 25-year design is intended to provide 100-year protection. Many layman believe that the 100-year Q is ten times the 10-year Q and the 100-year Q is four times the 25-year Q.

- The Hydrology Manual requires the determination of three Q's by two different methods.

- The change from 10-25-100-year produces a discontinuity at 500 and 4000 acres: a point in the system where Q-in does not equal Q-out. The magnitude of the discontinuity is illustrated in Exhibit A. As explained on Page 10 of the 1973 Hydrology Manual, the Rational Method over-predicts as the watershed increases and it was believed that at about 4000 acres the over-prediction would yield a 100-year Q with a 25-year Rational Method. In reality, this smooth transition seldom exists and there was usually a discontinuity in Q at 4000 acres where the change was made from 25-year Rational Method to 100-year Unit Hydrograph. There was also a discontinuity in Q at the change from 10-year to 25-year.

NEW CRITERIA FOR DESIGN DISCHARGE

On balance the short-comings of the design discharge criteria set forth in the 1973 Hydrology Manual are minor. Therefor, the new criteria described herein follows the 1973 Hydrology Manual criteria with minor modifications intended to overcome the shortcomings of the 1973 Hydrology Manual criteria. The new criteria simulates the 1973 Hydrology Manual criteria but:

- Design Q's are expressed in terms of a percentage of Q-100. Thus, only a single hydrologic calculation, Q-100, is necessary.

- The percentage is gradually increased as the watershed increases to avoid discontinuities which formerly occurred at 500 and 4000 acres.

- Where a 10-year discharge is needed for street design, it may be obtained by using .62 Q-100.

The 1973 and new criteria presented herein are illustrated in Exhibit A. The similarity and minor differences are readily apparent.

The relationship to Q-100 (i.e. Q-10 = .62 Q-100 and Q-25 = .77 Q-100) is an approximation developed from an average of Orange County hydrology reports which included calculations of Q-10, Q-25, and Q-100. The actual relationship will vary substantially from watershed to watershed and within a watershed. However, the approximation and variation are consistent with the other approximations involved in estimations of the hydrologic process and the impact of variations is of less consequence with the smaller discharges (i.e. at the upper end of the watershed) to which they are applied.

The design Q for new facilities shall be in accordance with the values presented in the table below: the values are derived from Exhibit A. Q-100 is the discharge determined by the 1986 Hydrology Manual. The design Q shall be carried in the pipe (or channel) plus street (to top of curb). Calculations demonstrating that building pads are protected from Q-100 will be required except where street grades are 1% or greater, lot grading and street section conforms to current County standards, tributary areas are less than 40 acres, building pads are above curb elevation, and there are no sumps, floodplain or other unusual site conditions. When the design Q is between 0.77 and 1.00 Q-100, the 100-year freeboard criteria shall be used.

Drainage area-acres	Design Q	Approx. recurrence interval
less than 100	.62 Q-100	Q-10
100 to 199	.65	
200 to 299	.69	
300 to 399	.72	
400 to 499	.76	
500	.77	Q-25
501 to 699	.79	
700 to 999	.83	
1000 to 1499	.87	
1500 to 1999	.91	
2000 to 2999	.95	
3000 to 3999	.98	
greater than 4000	1.00 Q-100	Q-100

APPLICATION OF NEW CRITERIA

The design discharge criteria contained herein **apply only to new facilities proposed where no EMA facility currently exists.** For new facilities proposed to replace, extend, or fill gaps in existing EMA facilities, the project engineer shall recommend (for Director of Public Works approval) the design discharge after considering various levels of protection and related costs and benefits.

Enc.   Addendum #1
       Exhibit A

       JWW:jc(M-20)050

NEW CRITERIA

1973 HYDROLOGY MANUAL

AREA – ACRES

PERCENTAGE
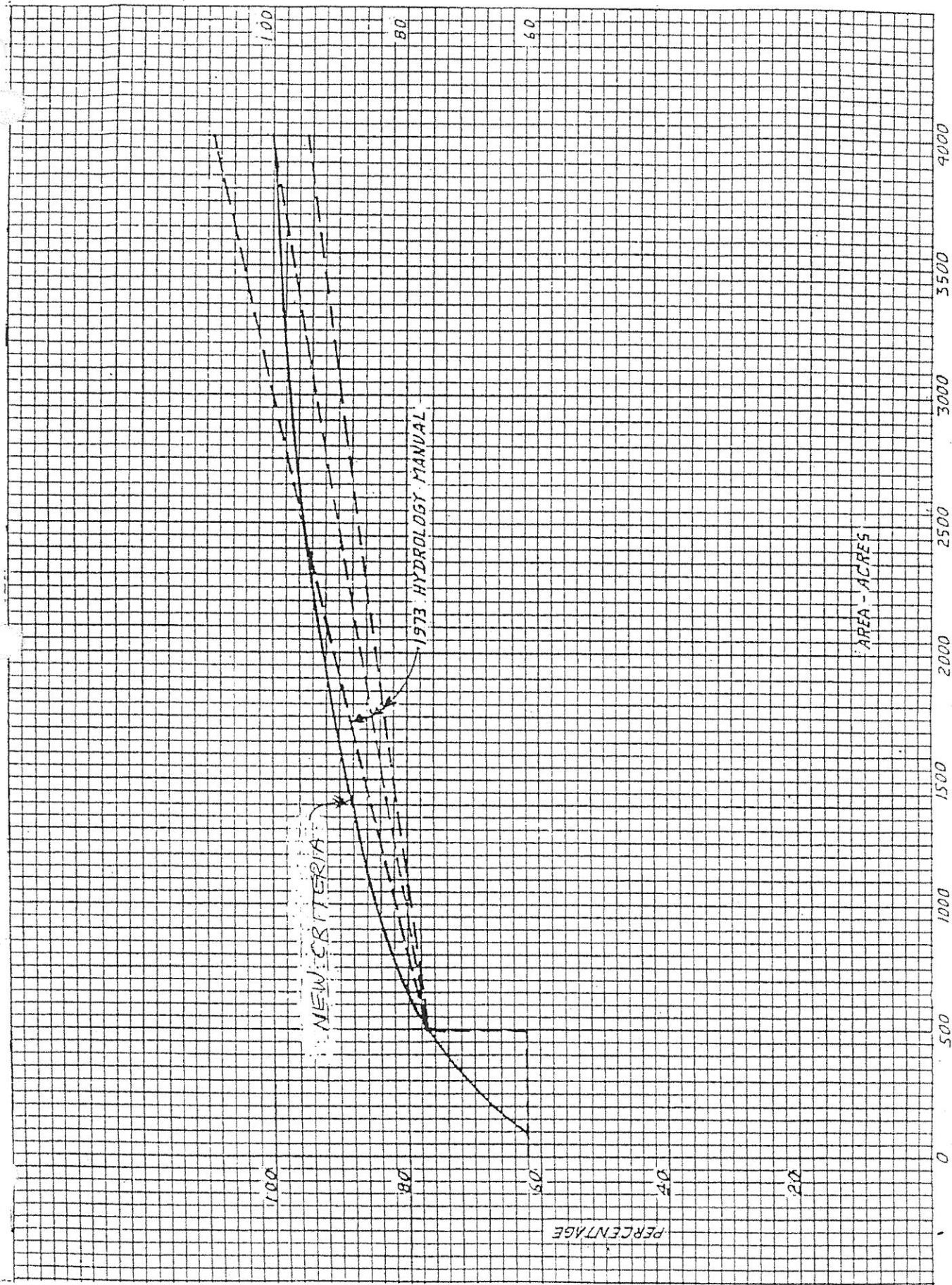
To: Distribution

From: J. M. Natsuhara, Manager   EMA/PW-Flood Control Program
           J. W. Williams, Manager   EMA/PW-Design Division
           D. N. Crouse, Manager   EMA/PW-Subdivision

Subject: EMA Drainage Policy - Resolution of Differing Qs

The 1973 Hydrology Manual was recently replaced by a 1986 Hydrology Manual. The purpose of this memo is to provide criteria for addressing situations where the new manual provides Qs at variance with the previous Qs.

A. GOALS AND CONDITIONS

1. EMA's goal is 100-year protection of structures (reference Public Facilities Element) and to provide for facilities to accommodate this goal where feasible. An 85% confidence level shall be used to determine 100-year discharge for new facilities. A lesser confidence level, but not less than 50%, may be considered to provide 100-year protection in an existing area.

2. Designs prepared under the 1973 Hydrology Manual proceeded on the assumption that conveyance based on 10/25/100-year discharges would achieve the goal of 100-year protection for structures. (Reference the 1973 Hydrology Manual page 10.):

- because 10/25 Rational Method was known to overpredict especially as drainage-area increased.

- because street flow was assumed to carry the excess not conveyed in the flood control facility.

3. The simplifying assumption made in paragraph 2 above was made because 100-year Qs statistically relevant to Orange County were not available.

4. The inventory of flood facilities include many facilities built to other criteria which existed prior to the 1973 Hydrology Manual.

5. We now have a new Hydrology Manual which produces 100-year Qs statistically relevant to Orange County and computer software for quick and complex analysis of systems.

6. Pay-as-you-go funding prevented building a single integrated flood control system and each channel typically includes many facilities built at different times and according to different criteria: it is a mixture of old (1973-1986), older (pre-1973), and future facilities.

7. One hundred-year Qs determined with the new Hydrology Manual may be larger, smaller, or the same as previous Qs.

8. If a new facility is built to a higher standard (i.e. higher Q) than the facilities upstream and/or downstream, the new construction may, at design discharges, cause over-flow to occur where it joins the downstream old facility unless either there is a flow restricting device built into the upstream end of the new facility or the tributary system cannot deliver the higher discharge.

9. If a new District-funded facility is built to a higher standard (i.e. higher Q) than facilities upstream and/or downstream, the result is excess investment with no economic return until upstream and/or downstream facilities are enlarged or replaced.

10. If a new District-funded facility is built to a lower standard (lower Q) than facilities upstream and/or downstream, the result is that a potential over-flow is created at the point of restriction and backwater or overflow may nullify the extra capacity provided by the old facilities and a portion of the old investment is wasted.

11. Concrete-lined facilities already built to a lesser standard typically will be accorded a low priority for reconstruciton and will not be enlarged or replaced for a long time (if ever):

- because funds are scarce, need is great, (a 1 + billion dollar backlog exists) and limited funds can be better used at other locations.

- because incremental increases in capacity are usually not feasible: capacity usually cannot be enlarged without demolition of the existing facility and replacement with a larger facility.

- because facilities already constructed to a lower standard appropriate many of the benefits and reduce the benefit-cost ratio for 100-year projects.

B. GUIDELINES

All of the above suggests that there is a new engineering issue which must be dealt with on a project-by-project basis. Guidelines are needed to determine what Q to design new facilities for. The following guidelines are provided:

1. Where a project report is needed, the issue should be resolved by the project report. Where a project report is not needed, preliminary design should resolve the issue. Where a request to cover (Resolution 72-1) is made by a developer, the issue should be resolved by Subdivision, after consultation with Flood Program Manager.

2. For all new facilities where no existing OCFCD facility exists, use new Hydrology Manual Qs.

3. For facilities where only a minor length of existing facilities need to be improved, and there is a high probability that the replacement of the deficiencies can be budgeted within the next ten years, use new Hydrology Manual Qs.

4. If there is a mix of existing and future facilities (the usual case) refer to the attached tree diagram, which is described by a, b, and c below.

    a. Determine the difference between new and prior Qs. If the difference is less than + or -10%, use the new Qs without further analysis, unless use of the new Q poses some special problem which needs further analysis.

    b. If the new Q exceeds the prior Q by more than 10%, determine the 100-year confidence level of the prior Q. If there are extensive facilities already built and if the confidence level exceeds 50% consider use of the prior Q, otherwise use new Q. If the confidence level is less than 50% and extensive facilities have already been built, consider the feasibility of using less than 100-year protection (i.e. the prior Q), otherwise use new Q.

    c. If the prior Q exceeds the new Q by more than 10%, and extensive facilities have already been built use the prior Q, otherwise use the new Q.
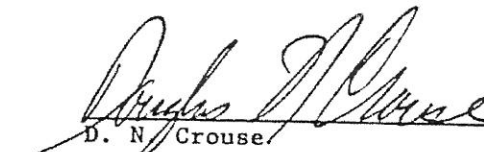
5. Analysis should consider the following:

- If upstream and/or downstream facilities are designed to smaller Qs than new Qs, consider the possibility of replacement/enlargement of upstream and/or downstream facilities. Consider factors such as extent of improvements which already exist, the cost of replacement, the difference in Q, availability of funds, water surface commitments, freeboard requirements, etc. If new Q is larger, include some way to deal with the over-flow concentration problem described in paragraph #8 of Goals and Conditions or temporarily limit new Q to downstream capacity.

- If upstream and/or downstream facilities are designed larger than the new Qs, in addition to the previous paragraph, determine whether an oversizing of the new Qs (i.e., increased level of protection) is desirable. In such a situation, it is believed that oversizing will be the usual choice in order to fully utilize the investment that has already been made in upstream and downstream facilities.

- Based on the preceding described analysis, recommend in the project report which Q to design for or, if the analysis is for a preliminary design, early in the design recommend to the Design Division Manager and Flood Programs Division Manager which Q to design for, or, if the analysis is for covering of a channel by Res. F-72-1 recommend to the Subdivision Manager and Flood Program Division Manager. There should be consultation with the Flood Program Division prior to formulating a recommendation. Obtain approvals of described manager prior to proceeding with final design.

0474

J. M. Natsuhara

J. W. Williams

D. N. Crouse

JWW:jc(M-13)019

Enclosure: tree diagram

Distribution:

J. Natsuhara
J. Williams
D. Crouse
K. E. Smith
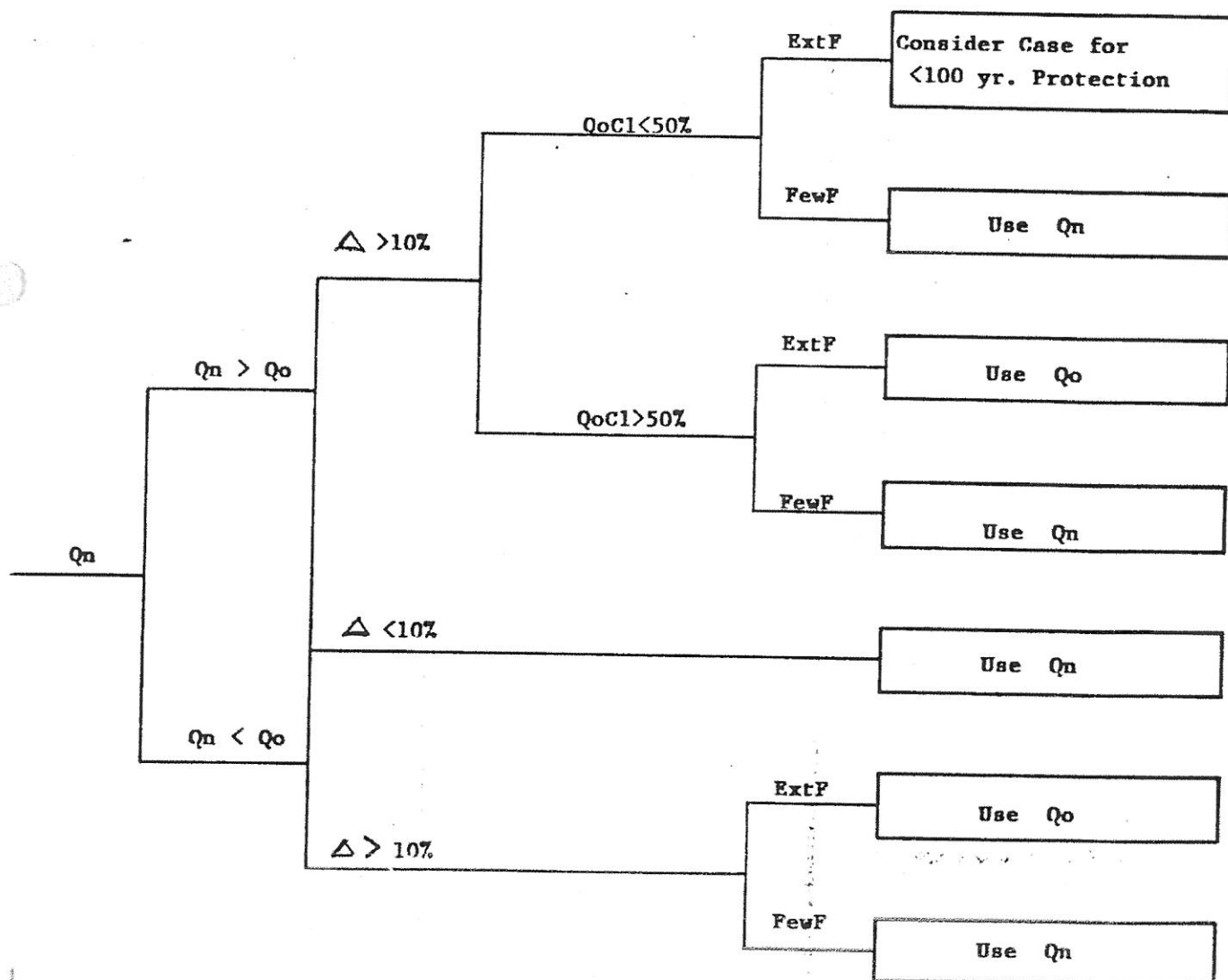J. Johansen
K. R. Smith
R. Rende
W. Reiter

Concurred: C. R. Nelson
Director of Public Works

TREE DIAGRAM - DIFFERING Qs

LEGEND

Qn	=	New Q
Qo	=	Old Q
Cl	=	Confidence Level
$\triangle$	=	Difference between Qn and Qo
FewF	=	Few facilities already built
ExtF	=	Extensive facilities already built

(F1/021)fgr

OC PublicWorks

Our Community. Our Commitment.

Memorandum

DATE: June 27, 2011

TO: Distribution

FROM: Ignacio G. Ochoa, P.E.
Director/Chief Engineer, OC Engineering

SUBJECT: OCFCD Design Manual, Chapter II, "Structures"

OC Public Works staff, on behalf of the Orange County Flood Control District (OCFCD), has completed a comparison of its current policy and standards for the design of reinforced concrete flood control structures with the structural design standards used by other agencies. These agencies included the Los Angeles County Flood Control District/Department of Public Works (LADPW), the U.S. Army Corps of Engineers (Corps), and the American Concrete Institute (ACI). Specifically, OCFCD staff completed a comparison between OCFCD criteria for design of flood control structures and that of the LA County 2008 Draft structural design criteria, the Greenbook pre-cast concrete culvert criteria, and the Corps criteria for hydraulic structures, EM 1110-2-2104.

OCFCD's Flood Control Design Manual has not been formally updated since 1971. However, a series of Design Manual addenda have been formally adopted by the Board of Supervisors over the years. In addition to the addenda, the Chief Engineer of OCFCD has issued Design Memoranda to OC Public Works staff on an as-needed basis, i.e. - the OCFCD Chief Engineer's memo on standards for reinforced concrete design within the OCFCD Design Manual, dated December 27, 2005, attached hereto.

OC Public Works staff concluded that for the last 30 years the OCFCD's Strength Design Criteria, as provided for in Chapter II of the current OCFCD Design Manual, has closely reflected the Corps strength design criteria as specified in the Corps Engineering Manual, EM 1110-2-2104, incorporating a 1.3 hydraulic structures magnification factor above and beyond those published by the ACI-318 Code for the Strength Design Method (Ultimate Strength Design). Other municipalities and Greenbook committees have in effect increasingly recognized this trend. Additionally, OC Public Works staff concluded that explicit application of a hydraulic structure magnification factor above and beyond that for ordinary structures is a common trend for hydraulic and sanitary structures. The hydraulic magnification factor is sometimes referred to as the resilience factor or the environmental durability factor.

Since it is the desire of OC Public Works to move in a direction where design and construction of its flood control infrastructure meet with and be eligible for Federal approval (i.e., FEMA Levee Certification, compliance with Corps PL 84-99 Program, and requests for Section 104 Credit associated with Federal Projects), OCFCD shall incorporate and specify by reference within the OCFCD Design Manual the use of the Corps Engineering Manual

(EM 1110-2-2104), "Strength Design for Reinforced-Concrete Hydraulic Structures," for design of all future reinforced concrete flood control structures.

The adoption of EM 1110-2-2104 is not expected to cause any measurable increase in the cost of construction of the OCFCD's infrastructure.

Ignacio G. Ochoa, P.E.

S:\Flood Control Design\Manager\Share\Projects-back up\Flood Control Policy\Structures\IGO, OCFCD Hydraulic Structures Factor.doc

Attachment: Memo dated December 27, 2005

Distribution: Kevin Onuma, Manager, OC Engineering/OC Flood
Khalid Bazmi, Manager, OC Engineering/OC Road
Vincent Gin, Manager, OC Engineering /OC Project Management
Victor Valdovinos, Manager, OC Engineering /Operations & Maintenance
Octavio Rivas, Manager, OC Engineering/OC Inspection
Mahrooz Ilkhanipour, Manager, OC Planning/OC Community Development
Mary Anne Skorpanich, Manager, OC Planning/OC Watersheds
Lance Natsuhara, OC Engineering/OC Flood/Santa Ana River Project
Phil Jones, Manager, OC Engineering/OC Flood/Flood Control Design
Mehdi Sobhani, OC Engineering/OC Flood/Flood Control Programs
Harry Persaud, Manager, OC Planning/OC Planned Communities
Scott Thomas, Manager, OC Community Resources/OC Parks Design

COUNTY OF ORANGE

RESOURCES & DEVELOPMENT MANAGEMENT DEPARTMENT

Bryan Speegle, Director
300 N. Flower Street
Santa Ana, CA

P.O. Box 4048
Santa Ana, CA 92702-4048

Telephone: (714) 834-2300
Fax: (714) 834-5188

DATE: DEC 2 7 2005

TO: Distribution

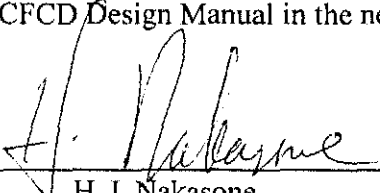**FROM:** H. I. Nakasone
Director Public Works/Chief Engineer

SUBJECT: OCFCD Design Manual, Chapter II, "Structures"

The Orange County Flood Control District criteria for design of flood control structures have not been modified since July 1972. During the early '70's, Working Stress Design was the method most commonly used by the District for the design of reinforced concrete flood control structures. In addition, the 1972 Manual set forth the relatively new Ultimate Strength Design method (now simply know as the Strength Design Method) as an alternate method for design of reinforced concrete structures.

In the last 35 years structural criteria and codes have evolved and have been significantly modified. The most popular and referenced code for design of reinforced concrete structures in the Country is the American Concrete Institute (ACI) code. The ACI 318 – 71 code is also referenced by our current design manual. The current ACI code for reinforced concrete design, ACI 318 – 2005, now specifies the Unified Design Provisions for the Strength Method as the preferred method for designing reinforced concrete structures. The conventional Strength Design Method has been moved to the appendices, and the Working Stress Design Method has been removed from the ACI code in its entirety.

Due to this evolution in code requirements, the Orange County Flood Control District shall now require that all newly proposed reinforced concrete flood control structures be designed by the Strength Design Method as provided for in Chapter II of the current OCFCD Design Manual. The Working Stress Design Method shall no longer be used for newly proposed flood control structures, but may be utilized for the modification of, connection to, and analysis of existing District owned concrete structures if so approved by the Chief Engineer or his designee.

The design criteria above shall prevail until such time as District staff completes and recommends further revisions to Chapter II of the OCFCD Design Manual in the near future.

H. I. Nakasone

Distribution: Tim Neely
Francisco Alonso
Nadeem Majaj
Nacho Ochoa
Jim Miller
Dave Marshall
Bill Tidwell

TO: C. R. Nelson, Director of Public Works

FROM: J. M. Natsuhara, Manager, Flood Program Division

SUBJECT: Policy Statement - Water Surface Control

Attached for your concurrence is a policy statement on water surface controls for side inlets. A peer review has been accomplished with many good comments. The Hydrologic and Hydraulic Criteria Committee has approved the policy statement.

Item I.A. gives water surface controls for side inlets. Case 1 is usually given, however, the engineer may opt for Case 2 or Case 3 depending on the circumstances. Item I.B. is covered in Manual of Policy and Procedures 4.2.111, but repeated here to make the water surface control policy complete. Item II is stated for guidance to designers.

If you concur, please sign in the signature block provided and return to Flood Program office for distribution to affected divisions.

J. M. Natsuhara

Concurr:

C. R. Nelson, Director of Public Works

JMN:lcc5-35

I. WATER SURFACE CONTROL FOR SIDE INLET

A. General

Confluences shall be designed in accordance with Chapter I, Section D(2) page 6 of the OCFCD Design Manual, dated July 1972. Where the incremental increase in flow does not exceed 10% the following shall apply:

Case 1

The peak discharge for main channel and side inflow shall be considered coincident where the time difference between peak discharge is 30 minutes or less. The water surface control to design the side inflow channel will be based on hydraulics for the peak discharge condition.

Case 2

The peak discharge occurs in the side inflow system when main channel has not approached its peak discharge. The water surface control for the side inflow channel will be controlled by the main channel hydraulics with lower than peak discharge in the main channel but in no case shall the water surface calculations be based on a discharge less than 77% of Q100 in the main channel. (Optional)

Case 3

The peak discharge occurs in the main channel and side inflow is not at its peak discharge. The water surface control for the side inflow system will be controlled by the main channel peak discharge. (Optional)

B. Water Surface Control for channels with interim (less than ultimate) conveyance sections and with no prior engineering study/project report.

Water surface control elevation is generally given as one foot below top of channel embankment or adjacent ground where unleveed area is other than flood plain Zone A. It should be noted, if the request is for a channel within Zone A (FIRM) no water surface control is given. (See Manual of Policy & Procedure 4.2.111)

C. Committed Water Surface Control

Committed water surface controls for channels calculated for 25-year discharges will be held for channels subsequently designed for 100-year discharges, where practical. If the committed water surface control is exceeded, the tributary drainage system shall be investigated for hydraulic efficiency of existing tributary system and watershed response.

II. WATER SURFACE CONTROL WITHIN BRIDGE/CULVERT

Side inlets should not be located at culverts and bridges except when there are no reasonable alternatives. Water surface control at culverts and bridges shall be established by design of the side inlets based on hydraulic analysis referenced to committed water surface upstream or downstream.

JMN:1cc(2-9)8-2-88