DESIGN GUIDELINE

DG452
Structural Design Guidelines

Issue No. 5

October 5, 2009

Approved By:

Peter Torres, P.E., Chief Civil/Structural Engineer

Issue Record

No.	Date	Description of Change	Entered By	Formal Review	Intermediate Review
0	01/15/75	Initial Issue	Khan	x	
1	08/04/94	Reissue of existing guideline	Khan		x
2	08/04/01	Chapter 3 updated	McMorrow	x	
3	08/12/04	Added reference to DG452A	McMorrow		x
4	12/29/06	Reissue of existing guideline – 5 yr formal review interval	Kenkre	x	
5	10/05/09	Reissue of existing guidelines – 3 yr review	Mitchell	x	
	05/28/14	Madan Naik's Title Change			

Division of Engineering Services
Madan Naik, P.E.
Vice President and Deputy Chief Engineer

FOREWORD:

MTA New York City Transit is, as a minimum, required to comply with the provisions of the Building Code of New York State. In addition to the above, applicable provisions of the following Design Guidelines (DG) must be considered on all projects:

- **DG 452** **Structural Design Guidelines**

  This guideline provides information relating to train clearances and track alignment and loads. In addition, design criteria are provided for both elevated and subway structures. This document is primarily intended for use on projects involving modifications and/or rehabilitation of existing transit structures or facilities.

- **DG 452A** **Structural Design Guidelines**
  Subway and Underground Structures

  This guideline provides information relating to loads and in addition, design criteria are provided for subway structures. This document is primarily intended for use on projects involving construction of new below ground transit structures or facilities.

Each project is to be evaluated at the initiation of design to determine its applicability to the provisions of each of the aforementioned Design Guidelines.

STRUCTURAL DESIGN GUIDELINES

INDEX

STRUCTURAL DESIGN GUIDELINES

INDEX

Chapter 1

Chapter 1

DATUM TABLE

TABLE OF CONTENTS

Chapter 1

DATUM TABLE

COMPARISON OF DATUM PLANES

	A	B	DESCRIPTION
1	100.539	+3.192	BOROUGH PRESIDENT OF RICHMOND
2	100.097	+2.750	N.Y. CENTRAL R.R.
3	100.097	+2.750	BOROUGH PRESIDENT OF MANHATTAN
4	100.072	+2.725	L.I.R.R. EXCEPT BAY RIDGE DIV. & TUNNELS
5	100.072	+2.725	BOROUGH PRESIDENT OF QUEENS
6	99.972	+2.625	WILLIAMSBURGH BRIDGE
7	99.955	+2.608	BOROUGH PRESIDENT OF BRONX
8	99.955	+2.608	NEW YORK, NEW HAVEN & HARTFORD R.R.
9	99.907	+2.560	BOROUGH PRESIDENT OF BROOKLYN-HIGHWAYS
10	99.646	+2.299	QUEENSBOROUGH BRIDGE
11	99.234	+1.877	BROOKLYN BRIDGE
12	99.067	+1.720	BOROUGH PRESIDENT OF BROOKLYN-SEWERS
13	99.027	+1.680	BROOKLYN WATER SUPPLY
14	99.025	+1.678	L.I.R.R. - BAY RIDGE DIVISION
15	99.024	+1.677	MANHATTAN BRIDGE
16	97.347	0	BOARD OF WATER SUPPLY
17	97.347	0	U.S.C & G. SURVEY (MEAN SEA LEVEL, SANDY HOOK)
18	97.347	0	WESTCHESTER COUNTY PARK COMM.
19	96.561	-0.786	CROTON AQUEDUCT
20	95.244	-2.103	DEPT. OF MARINE & AVIATION
21	95.244	-2.103	NEW YORK CONNECTING R.R.
22	0	-97.347	N.Y.C. TRANSIT AUTHORITY*
23	-199.928	-297.275	PENN. TUNNELS (2.725=300)
24	-200.163	-297.275	L.I. TUNNELS OF THE PENN. R.R. (2.490=300)
25	-202.653	-300.000	PORT AUTHORITY TRANS. - HUDSON
26	-202.653	-300.000	NARROWS TUNNELS

To obtain from any given foreign elevation the corresponding elevation referred to the Transit Authority datum plane: add to the given elevation the figure shown in column A.

Column B shows datum in relation to U.S.C. & G. survey datum.

*Elevation 100 of Transit Authority is 2.653 ft. above Mean Sea Level at Sandy Hook.

Chapter 2

Chapter 2

CLEARANCES

TABLE OF CONTENTS

CHAPTER 2

CLEARANCES

I - LATERAL CLEARANCE

1. The net line of a structure shall preferably be not less than 1'-6" from the building line. Where this distance is inadequate for construction work, a temporary easement may be necessary, unless underpinning is provided under the building of such a type that it will act as a sheeting. In the latter case, the subway design and specifications shall be such as to definitely require the Contractor to do the work on this basis.

2. Where a sewer is located between the subway and building walls or vaults, the clear distance shall not be less than the maximum external diameter of the sewer and in no case less than 3'-6".

3. The amount of permanent easement to be taken outside of the net line of a structure encroaching on private property shall in each case be discussed with the Construction Division, but shall generally be not less than 1 ft.

4. The standard distances from centerline of tangent track to ℄ of subway columns, benches and platform edge are shown in Figs. 1A to 1F. The distances are to be increased on curves to allow for super elevation, center and end excesses.

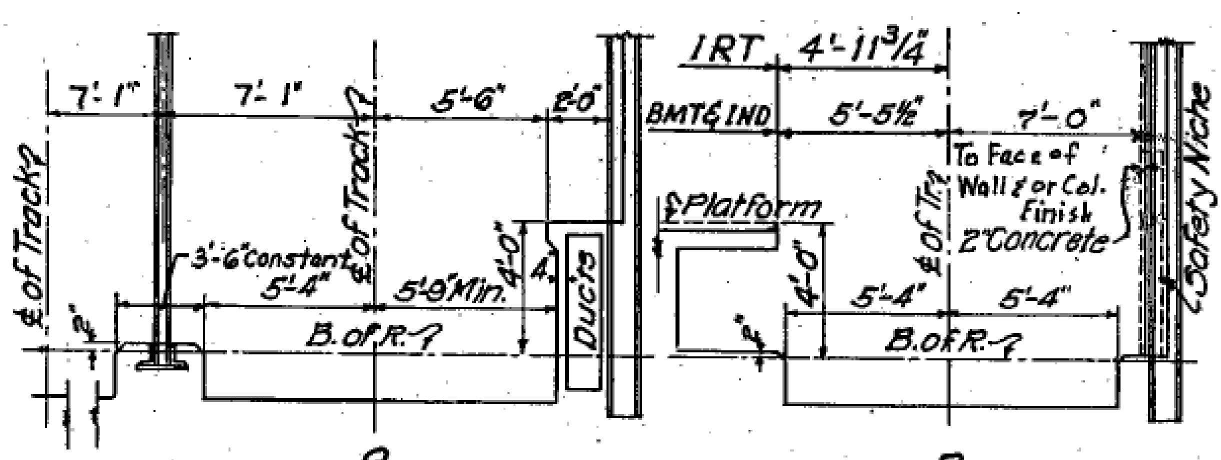


Figure 1
SD-2-1

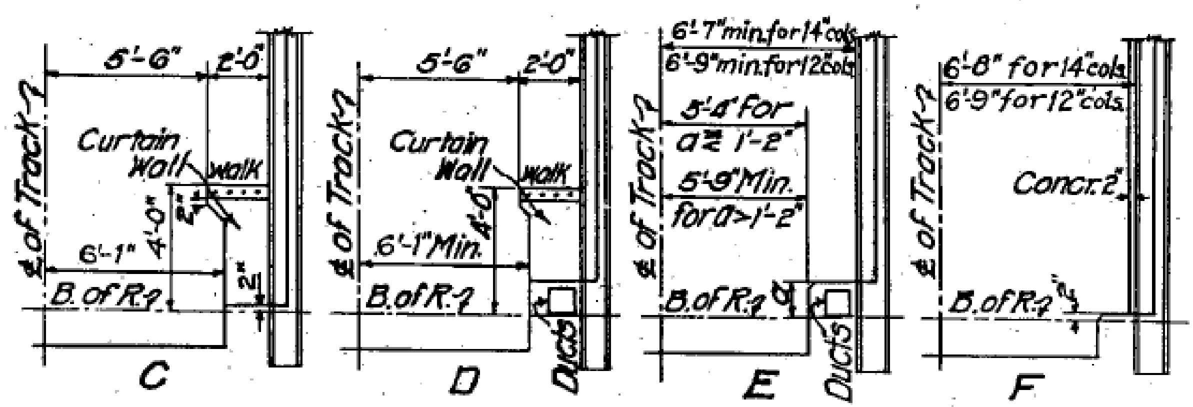
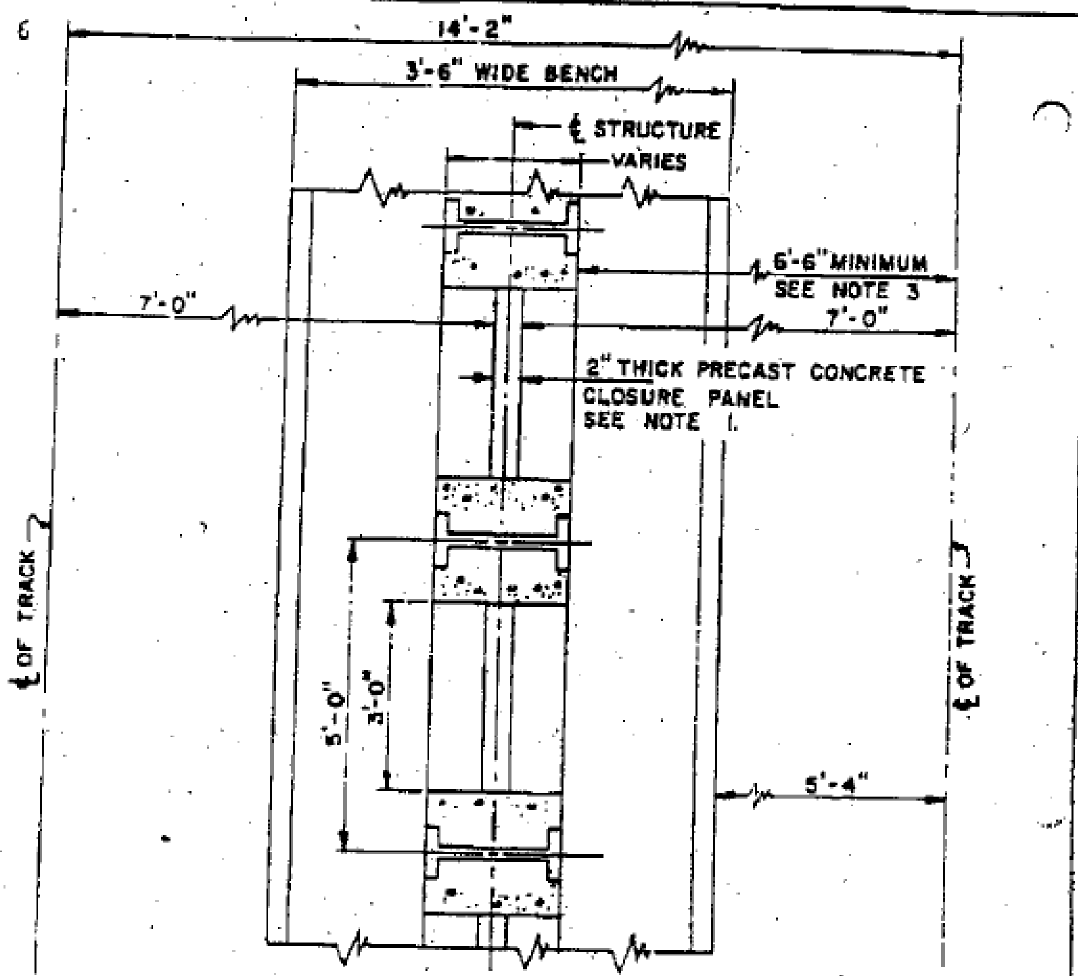


Figure 1 (Cont'd.)

5. Where safety niches cannot be provided in an interior wall for a length of more than three consecutive bays, or 15 ft., the minimum distance from centerline of track to face of wall shall be 7'-0" on tangent, to be increased by the required amounts on curves.



TYPICAL PART PLAN OF CENTER WALL

NOT TO SCALE

NOTES

1. ELIMINATE CLOSURE PANEL FOR ACCESS (APPROXIMATELY EVERY 300 FT.) AND AS REQUIRED FOR EQUIPMENT PURPOSES.
2. DISTANCES FROM CENTERLINE OF TANGENT TRACK ARE TO BE INCREASED ON CURVES TO ALLOW FOR SUPERELEVATION, CENTER AND END EXCESSES.
3. ALLOWANCE FOR ENCROACHMENT NOT INCLUDED. SEE SECTION II

MODIFICATION TO CLEARANCES BETWEEN CENTER WALL AND CENTERLINE OF TANGENT TRACK FOR IND-BMT STRUCTURES

II - PERMISSIBLE ENCROACHMENTS ON LATERAL CLEARANCE

7. The value given in Fig. 1A for the distance from centerline of track to an interior column provides for a normal clearance of 1'-6" on tangent. This clearance shall generally be maintained on curves but may, in special cases, be encroached upon to the extent specified under Subsects. a to d below. Only where the encroachment would exceed the amounts therein given, shall the distance between running tracks be increased.

Encroachment is permitted as follows:

a. At turnouts and crossovers, to the extent of 6 in. on the turnout or crossover track. Additional encroachment shall be taken care of by setting out the columns to encroach on the adjacent running track up to a maximum of 3 in. (See Fig. 2).

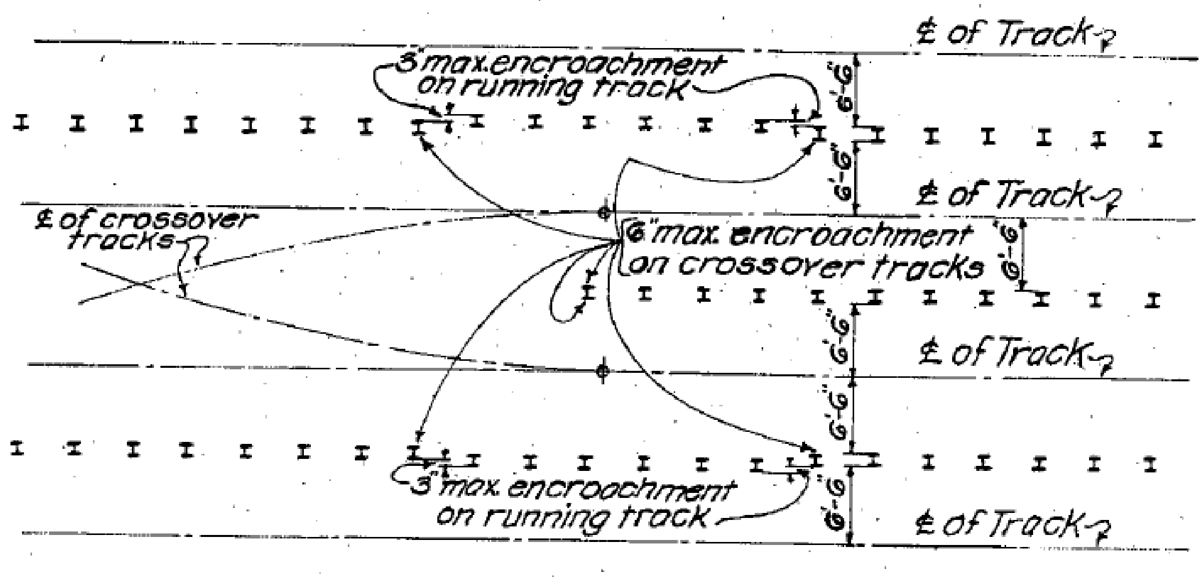


Figure 2

b. At turnouts back to back, to the extent of 9 in. on each turnout track (See Fig. 3). Encroachments exceeding 6 in. shall, however, be avoided wherever practicable by staggering the switch points and offsetting the columns, or by using higher frog numbers or long span roof beams.

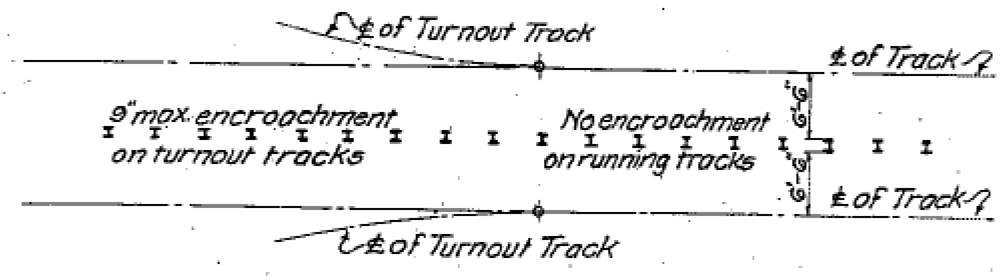


Fig. 3

c. At the junctions of curved and tangent tracks, to the extent of 3in. on each track (see Fig.6)

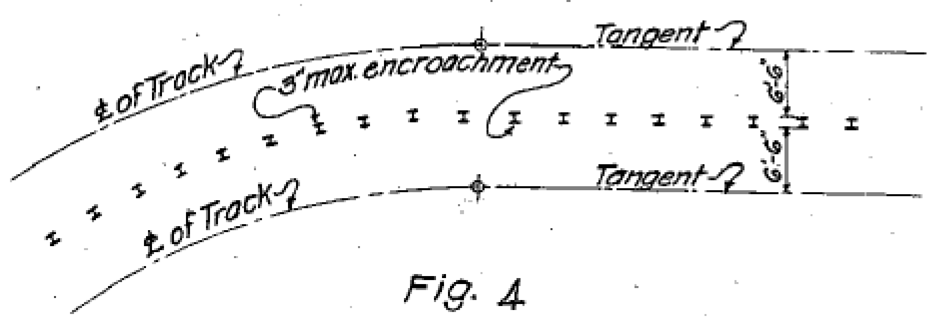
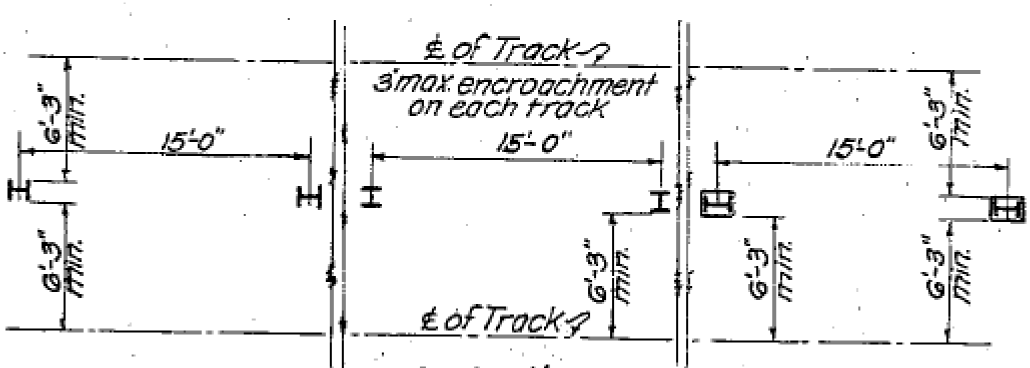


Fig. 4

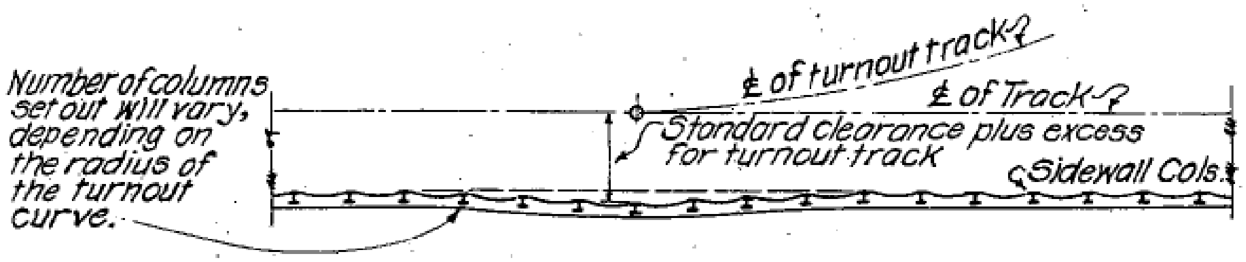
d. Where columns of increased depth are required between tracks, or where columns are encased in concrete, the encroachment may be 3 in. on each track (see Fig.7), except as provided in Sect. 29.



At Stations

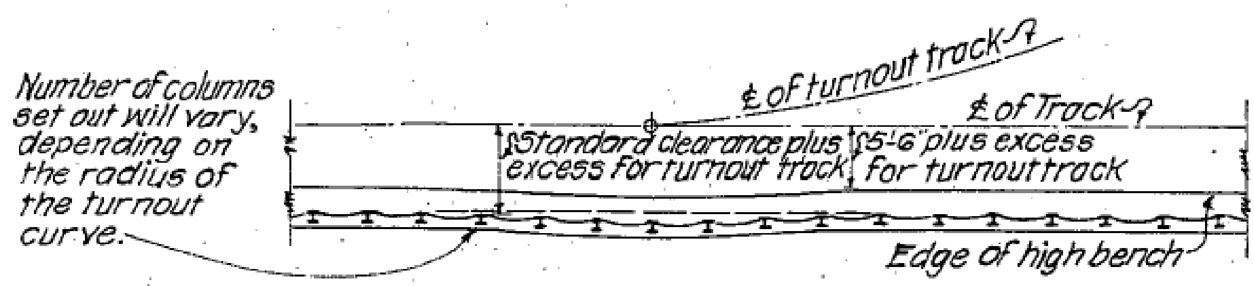
Fig. 5

8. No encroachment is permitted at sidewalls facing turnouts or crossovers. Such sidewalls shall be set out to allow for the full amount of clearance. (See Figs. 6 and 7)



Sidewall with Low Bench

Fig 6



Sidewall with High Bench.

Fig. 7

III - VERTICAL CLEARANCE

9. The minimum distance from street or ground surface to top of roof of main subway structure shall generally be 6 ft. over tracks and 5 ft. over mezzanines.

For special conditions, and under private property, these minima may be reduced as follows: Auxiliary structures, such as fan chambers, rectifier stations, entrances, passageways, etc., occupying relatively small areas of the roof surface, may have 2 ft. minimum cover, provided conditions are such as to permit a satisfactory relocation of subsurface structures. The minimum cover for auxiliary structures under sidewalks shall be 6 in.

The Supplementary drawings and the Construction Division shall be consulted to provide for additional clearance, where necessary, for sewers, street ducts and other subsurface structures. Raising the street or ground surface shall be considered as a possible economic measure.

10. In general, the depth of the Base of Rail below street or ground surface shall be about 21 ft. At mezzanine stations it shall be about 42 ft. The distance between Bases of Rail in double deck structures or at subway crossings shall be about 16 ft.

11. The distance from Base of Rail to underside of subway roof or mezzanine floor shall be not less than 13’-2”. Where roof members have lower cover plates which are not embedded in concrete, the distance shall be not less than 13’-2 3/4” to underside of lowest cover plate so as to allow for rivet heads. At vertical curves, additional allowance shall be made for vertical center or end excess (See Sect. 21).

12. In elevated structures, the minimum vertical clearance for girders shall be 14 ft. over roadways.

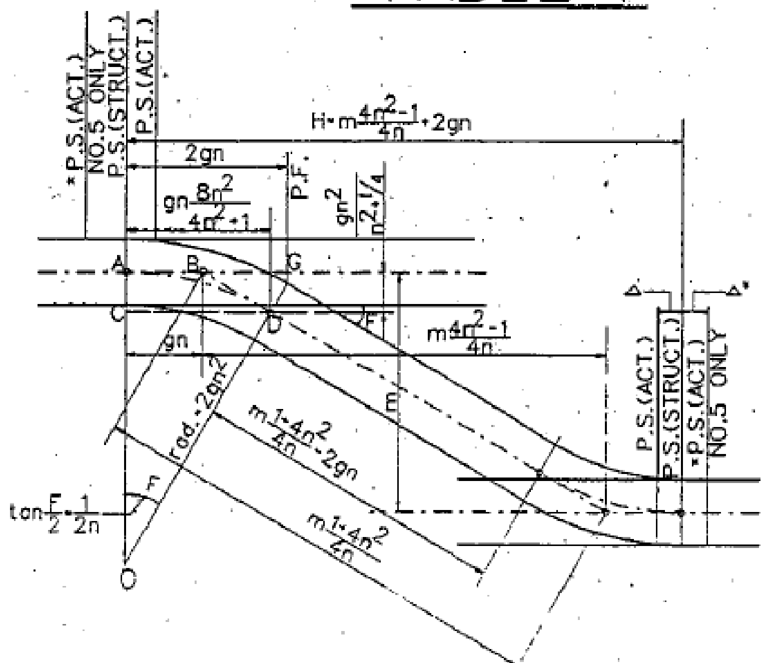
For clearance to be provided over railroad tracks, the railroad company concerned shall be consulted.

IV - FORMULAS FOR TURNOUTS AND CROSSOVERS

13. Formula for turnouts from tangent tracks, as well as for crossovers, are given in Fig. 8, and the corresponding values for standard frog numbers are given in Table 1, page SD-2-8.

FROG		RADIUS		TANGENT	LEAD	LENGTH OF CROSSOVER "H" FOR M=			PS (STRUCT) TO PS (ACTUAL)
NO.	ANGLE	A-O	C-D	A-B	A-G	13'-6"	14'-0"	14'-6"	△
5	11°-25'-16"	235.417	46.617	23.542	47.083	113.909	116.384	118.859	1.067*
6	9°-31'-38"	339.000	56.110	28.250	56.500	136.938	139.917	142.896	3.872
7	8°-10'-16"	461.417	65.582	32.958	65.917	159.936	163.418	166.900	10.852
8	7°-09'-10"	602.667	75.040	37.667	75.333	182.910	186.894	190.878	16.626
9	6°-21'-35"	762.750	84.489	42.375	84.750	205.874	210.360	214.846	10.493
10	5°-43'-29"	941.667	93.932	47.083	94.167	228.831	233.819	238.806	14.972
12	4°-46'-19"	1356.000	112.804	56.500	113.000	274.717	280.706	286.696	25.072
15	3°-49'-06"	2118.750	141.093	70.625	141.250	343.523	351.015	358.507	19.133
16	3°-34'-37"	2410.667	150.520	75.333	150.667	366.462	374.454	382.447	23.921
18	3°-10'-56"	3051.000	169.369	84.750	169.500	412.317	421.310	430.303	33.940
20	2°-51'-51"	3766.667	188.216	94.167	188.333	458.167	468.161	478.155	48.417

TABLE I



F=Frog angle
n=Frog number
g=Gauge of rails=4'-8½"
=4.70833'
log=0.6728672
m=Distance between center lines of tangent tracks.

FIGURE 8

V - CRANDALL'S TRANSITION CURVE

14. Crandall's transition curve is a double spiral coiling in opposite directions, as shown in Fig. 9. Its curvature at any point is proportional to the distance from its point of zero curvature as measured along the curve. Transitions are used for effecting a gradual change in centrifugal force when trains are entering or leaving curves, and also for securing additional clearance between concentric circular tracks.

15. Notation (See Fig. 10)

P.T.C.	=	Junction between tangent and transition at beginning of curve.
P.C.C.	=	Junction between transition and circular curve.
P.T.T.	=	Junction between transition and tangent at end of curve.
x, y	=	Coordinates of a point on transition curve.
i, i^o	=	Angle turned by curve at point x, y (in radians and degrees).
ℓ	=	Length of curve from P.T.C. to point x, y.
r	=	Radius of curvature at point x, y.
X,Y,I,I^o,L,R	=	Corresponding values at P.C.C.
F	=	Lateral throw of circular curve due to transition.
D	=	Abscissa of center of circular curve.
T	=	Abscissa of point of intersection of tangents, P.I.
V	=	Angle formed by tangents.
k	=	$l/\ell r$ (a constant)
n	=	ℓ/L (quotient of lengths ℓ and L)
Q	=	Value in degrees of one radian = 57.29578....(log = 1.7581226).

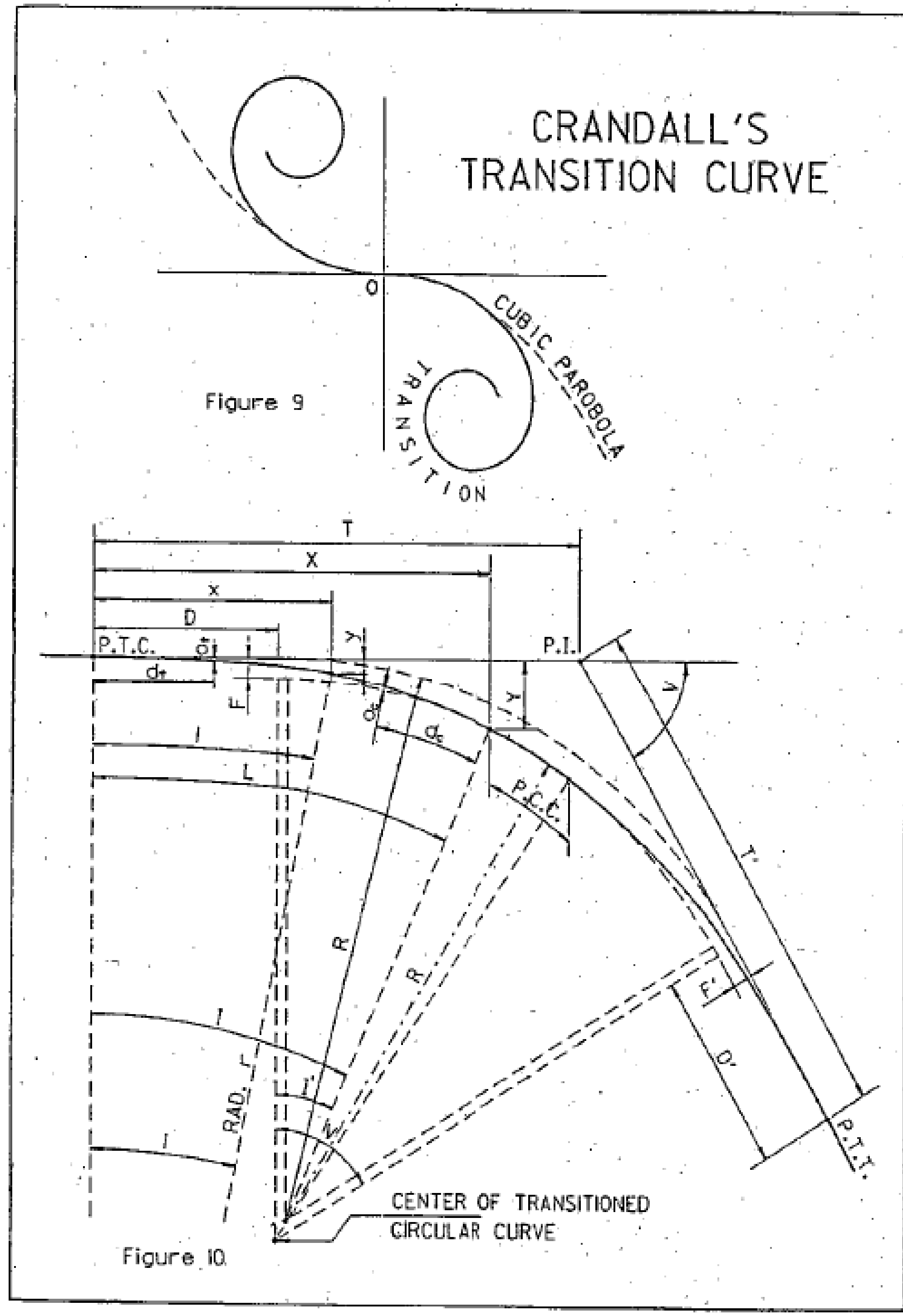


Figure 9

Figure 10

DERIVATION OF FORMULAS

16. BY DEFINITION:

$$\frac{1}{T} = \frac{di}{d\ell} = K\ell \ldots (1), \quad \frac{1}{R} = KL \ldots (2), \quad K = \frac{1}{RL} \ldots (3)$$

SUBSTITUTING (3) IN (1) AND INTEGRATING; WE HAVE

$$i = \frac{\ell^2}{2RL} \quad (4) \qquad I = \frac{L}{2R} \quad (5)$$

$$i^\circ = \frac{0\ell^2}{2RL} \quad (6) \qquad I^\circ = \frac{0L}{2R} \quad (7)$$

$$i^\circ = \frac{\lambda^2}{L^2} \; I^\circ = n \, I^\circ \quad (8)$$

ACCORDING TO GENERAL THEORY:

$$d\,x = d\lambda \cos i \quad \text{and} \quad dy = d\lambda \sin i$$

OR, SUBSTITUTING FROM (4):

$$d\,x = d\lambda \cos \frac{\lambda^2}{2RL} \quad \text{and} \quad dy = d\lambda \sin \frac{\lambda^2}{2RL} \quad (9)$$

BY EXPANDING THE COSINE AND SINE FUNCTIONS IN (9) INTO SERIES AND INTEGRATING, WE HAVE, AFTER SUBSTITUTING FROM (6):

$$x = \lambda - \lambda \left(\frac{1^{\circ 2}}{10Q^2} - \frac{1^{\circ 4}}{216Q^4} + \frac{1^{\circ 6}}{9360Q^6} - \ldots\ldots \right) \quad (10)$$

and

$$y = \lambda \left(\frac{1^\circ}{3Q} - \frac{1^{\circ 3}}{42Q^3} + \frac{1^{\circ 5}}{1320\,Q^5} - \ldots \right) \quad (10)$$

17. LATERAL THROW, LENGTH OF TANGENTS. THE TRANSITION CAUSES THE CIRCULAR CURVE TO BE THROWN OFF LATERALLY FROM THE TANGENT BY AN AMOUNT:

$$F = y - R\,(1 - \cos I^\circ) \quad (11)$$

THE ABSCISSA OF THE CENTER OF THE CIRCULAR CURVE BEING:

$$D = X - R \sin I^\circ \quad (12)$$

FOR THE TANGENT LENGTHS T AND T′ (FIG. 10) OF A CURVE TRANSITIONED AT BOTH ENDS WE HAVE:

$$T = D + (R + F) \tan \frac{V}{2} + \frac{F' - F}{\sin V} \quad (13)$$

$$T' = D' + (R + F') \tan \frac{V}{2} + \frac{F - F'}{\sin V} \quad (13)$$

VI - VERTICAL CURVES

18. Vertical curves are used in changing from one grade to another. They are parabolas with their axes vertical and are rated by the change r of grade affected in a distance of 100 ft., the change being expressed in percent:

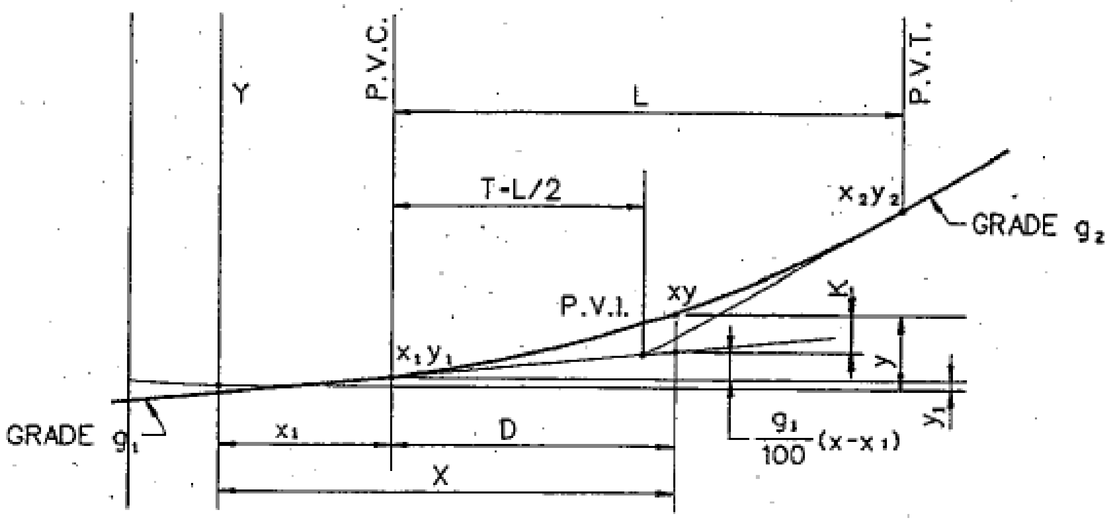


Figure 11

19. Notation (See Fig. 11)

P.V.C.	=	Junction between tangent grade and V.C. at beginning of curve.
P.V.T.	=	Junction between V.C. and tangent grade at end of curve.
P.V.I.	=	Point of intersection of tangent grades, produced.
r	=	Rate of vertical curve - in percent per 100 ft.
x,y	=	Coordinates of a point on vertical curve-in ft.
x_1, y_1	=	Coordinates of P.V.C. - in ft.
g	=	Grade at point x,y - in percent.
g_1, g_2	=	Tangent grades at P.V.C. and P.V.T., respectively - in percent.
D	=	Horizontal distance from P.V.C. to point x,y - in ft.
T	=	Horizontal distance from P.V.C. to P.V.I. - in ft.
L	=	Horizontal length of vertical curve – in ft.
K	=	Vertical offset of point x,y from tangent grade g, produced in ft.

20. THE EQUATION OF A VERTICAL CURVE, REFERRED TO ITS APEX O AS ORIGIN, IS:

$$y = \frac{x^2}{2} \quad \frac{r}{10000} \tag{1}$$

GIVING FOR THE GRADE AT ANY POINT ON THE CURVE:

$$\frac{g}{100} = \frac{dy}{dx} = x \quad \frac{r}{10000} \tag{2}$$

AT P.V.C., ACCORDINGLY,

$$y_1 = \frac{x_1^2}{2} \quad \frac{r}{10000}; \quad \frac{g_1}{100} = x_1 \frac{r}{10000} \tag{3}$$

CONSIDERING EQS. (1) AND (3), WE HAVE FOR THE VERTICAL OFFSET OF POINT x, y FROM TANGENT GRADE g_1 (SEE FIG. 11)

$$K = y - y_1 - \frac{g_1}{100} (x - x_1)$$

$$= \frac{x^2}{2} \quad \frac{r}{10000} - \frac{x_1^2}{2} \quad \frac{r}{10000} - x_1 \frac{r}{10000} \quad (x - x_1)$$

$$= \frac{r}{10000} \quad \frac{(x - x_1)^2}{2}$$

OR $K = \frac{r}{2} \left(\frac{D}{100}\right)^2$

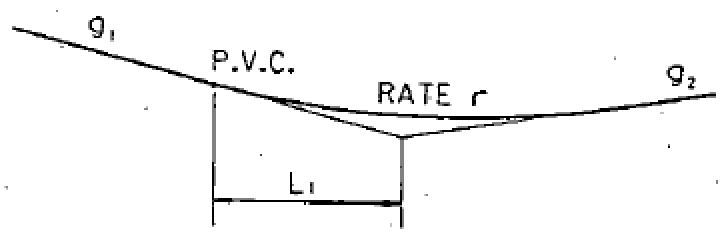
THE LENGTH L OF VERTICAL CURVE REQUIRED FOR JOINING GRADES g_1 AND g_2, FIG. 11, IS EQUAL TO $x_2 - x_1$, or, ACCORDING TO EQ. (2).

$$L = \frac{g_1 - g_2}{r} \quad 100 \text{ ft.}$$

SINCE IN ANY PARABOLA A DIAMETER THROUGH THE POINT OF INTERSECTION P.V.I. OF TWO TANGENTS BISECTS THE CHORD THROUGH THEIR POINTS OF CONTACT WITH THE CURVE, WE ALSO HAVE:

$$T = \frac{L}{2}$$

21. In figuring vertical center and end excess, the radius of the vertical curve shall be taken as $\frac{10000}{R}$ ft.



$L_1 = \frac{g_1}{r} 100$, where

L, is distance from P.V.C. to low or high point of vertical curve when grades g_1 and g_2 reverse direction or one grade is level.

VII - EXCESSES

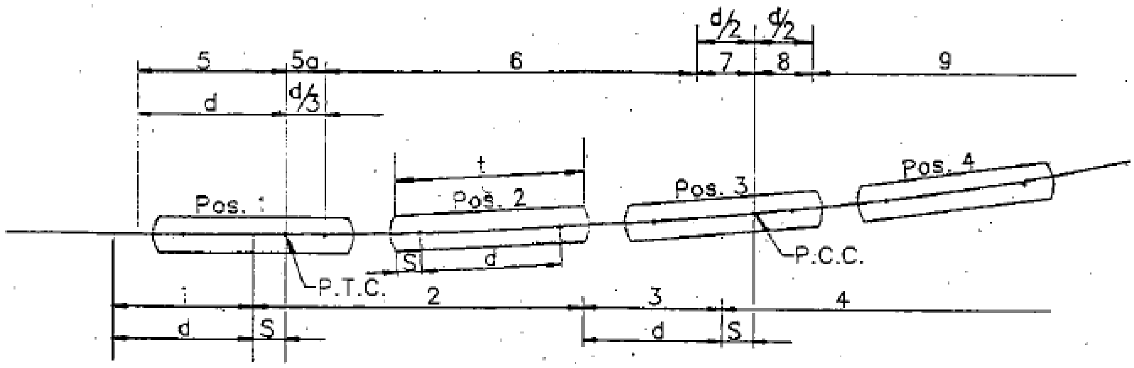
Superelevation excess on transition curves when superelevation at P.C.C. is known:

$$E_S = S\left(1 - \frac{B - \Delta}{L_S}\right) \frac{h}{4.7083}$$

<u>where:</u>

E_s	=	Excess in ft. due to superelevation.
S	=	Superelevation in ft. at P.C.C.
B	=	Distance in ft. from P.C.C.
L_s	=	Length of Track from O Superelevation to P.C.C.
h	=	Height above B. of R. to be considered for excess.
Δ	=	20 for 51’ (IRT) cars
Δ	=	25 for 67’ (BMT-IND) cars
Δ	=	30 for 75’ (BMT-IND) cars

FORMULAS FOR END & CENTER EXCESSES ON TRANSITION CURVES



1) $E.E. = \frac{(d+S+x)^3}{6LR}\,\frac{S}{d}$ (sign considered)

2) $E.E. = \frac{(d+S)S}{6LR}(3x+t)$

3) $E.E. = \frac{S}{6LR}\left[(S+d)(3x+t) - \frac{(x-L+S+d)^3}{d}\right]$

4) $E.E. = \frac{t^2 - d^2}{8R}$

5) $C.E. = \frac{9}{512}\,\frac{(x+d)^4}{RLd}$ (sign considered)

5a) $C.E. = \frac{9}{512}\,\frac{(x+d)^4}{RLd} - \frac{x^3}{6LR}$

6) $C.E. = \frac{1}{12RL}\left[x(d^2 - x^2) + (x^2 + \frac{d^2}{3})^{3/2}\right]$

7) $C.E. = \frac{1}{24RL}\left[3d^2 x + 2(L - x - \frac{d}{2})^3\right]$

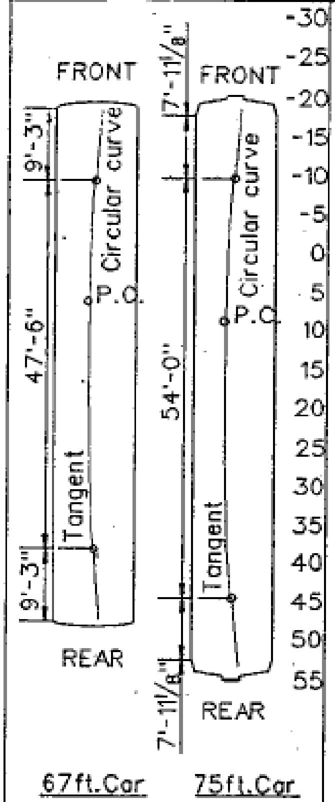
8) $C.E. = \frac{1}{24RL}\left[3d^2 L - 2(L - x - \frac{d}{2})^3\right]$

8) $C.E. = \frac{d^2}{8R}$

E.E. = End Excess
C.E. = Center Excess
x = Distance from P.T.C. to point considered (plus to right, minus to left)
t = Length of side of car
d = Distance between Truck Pivots
S = Overhang of end of Car
L = Length of Transition
R = Radius of Circular Curve

NOTE:
Above values expressed in ft.

EXCESS CLEARANCES FOR CARS ENTERING CIRCULAR CURVES—BMT-IND 67FT. & 75 FT. CARS



I End Excess	II Distance in ft. from P.C. B	III Center Excess High * Duct Bench	IV Center Excess Wall	V For 30 Miles per Hour Local Tracks High * Duct Bench	VI Maximum Speed 30 Miles per Hour Local Tracks Wall	VII of 40 Miles per Hour Express Tracks High * Duct Bench	VIII 40 Miles per Hour Express Tracks Wall
262	-30	365	282	605	913	792	1404
262	-25	364	282	604	913	791	1404
262	-20	356	279	596	910	783	1401
262	-15	338	268	578	899	765	1390
262	-10	310	246	550	877	737	1368
262	-5	269	213	509	844	696	1335
262	0	216	167	456	798	643	1289
250	5	161	120	401	751	588	1242
213	10	117	82	357	713	544	1204
170	15	81	54	321	685	508	1176
131	20	54	32	294	663	481	1154
98	25	34	18	274	649	461	1140
70	30	19	8	259	639-.02	446	1130-.02
46	35	9	3	249-.01	634-.05	436-.01	1125-.05
27	40	4	1	244-.02	632-.07	431-.02	1123-.07
13	45	1	0	241-.03	631-.09	428-.03	1122-.09
4	50	0		240-.04	631-.12	427-.04	1122-.12
0	55			240-.04	631-.14	427-.04	1122-.14
				For any station not given, located B ft. from P.C., amounts to be subtracted are:			
				$\frac{B-30}{565}$	$\frac{B-25}{215}$	$\frac{B-30}{565}$	$\frac{B-25}{215}$

To find End Excess or Center Excess at any station, divide corresponding figure in Col. I, III or IV by R (radius of curve in ft.).

To find Total Center Excess at Duct Bench or Well, divide left-hand figure in Col. V or VI (VII or VIII) by R and subtract right-hand figure, where given.

Values thus obtained are expressed in ft.

Full superelevation assumed at P.C., tapering 1 in. in every 40 feet. This presupposes that a sufficient length of tangent track is available for the run-off. (Minimum radius = 230 feet)

The entire car is assumed tilted same amount as front track. This table does not apply to sharp curves, where transitions are required.

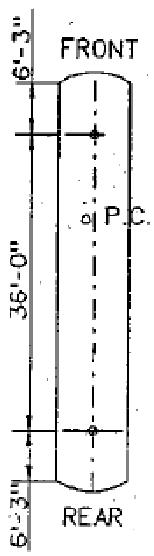
* Top of Duct Bench is 4'-0" above Base of Rail.

NOTE: Columns I, IV, VI and VIII refer to the 67 ft. car. Columns III, V and VII refer to the 75 ft. car.

EXCESS CLEARANCES FOR CARS ENTERING CIRCULAR CURVES

IRT CAR (51'-4" CARS)

I End Excess	II Distance in ft. from P.C. B	III Center Excess	IV Total Center Excess: High Duct Bench *	V Total Center Excess: Wall
132	-25	162	402	793
132	-20	162	402	793
132	-15	160	400	791
132	-10	150	390	781
132	- 5	129	369	760
132	0	96	336	727
120	5	61	301	692
90	10	36	276	667
64	15	18	259	650
43	20	8	248	639
26	25	3	243	634
13	30	0	240-.03	631-.05
5	35		240-.03	631-.07
1	40		240-.04	631-.09
0	45		240-.04	631-.12
	50		240-.05	631-.14
For any station not given, located B ft. from P.C., amounts to be subtracted in Columns IV and V are:			$\frac{B-20}{565}$	$\frac{B-20}{215}$



To find End Excess or Center Excess at any station, divide corresponding figure in Column I or III by R (radius of curve in ft.).

To find Total Center Excess at Duct Bench or Wall, divide left-hand figure in Column IV or V by R and subtract right-hand figure, where given.

Values thus obtained are expressed in ft.

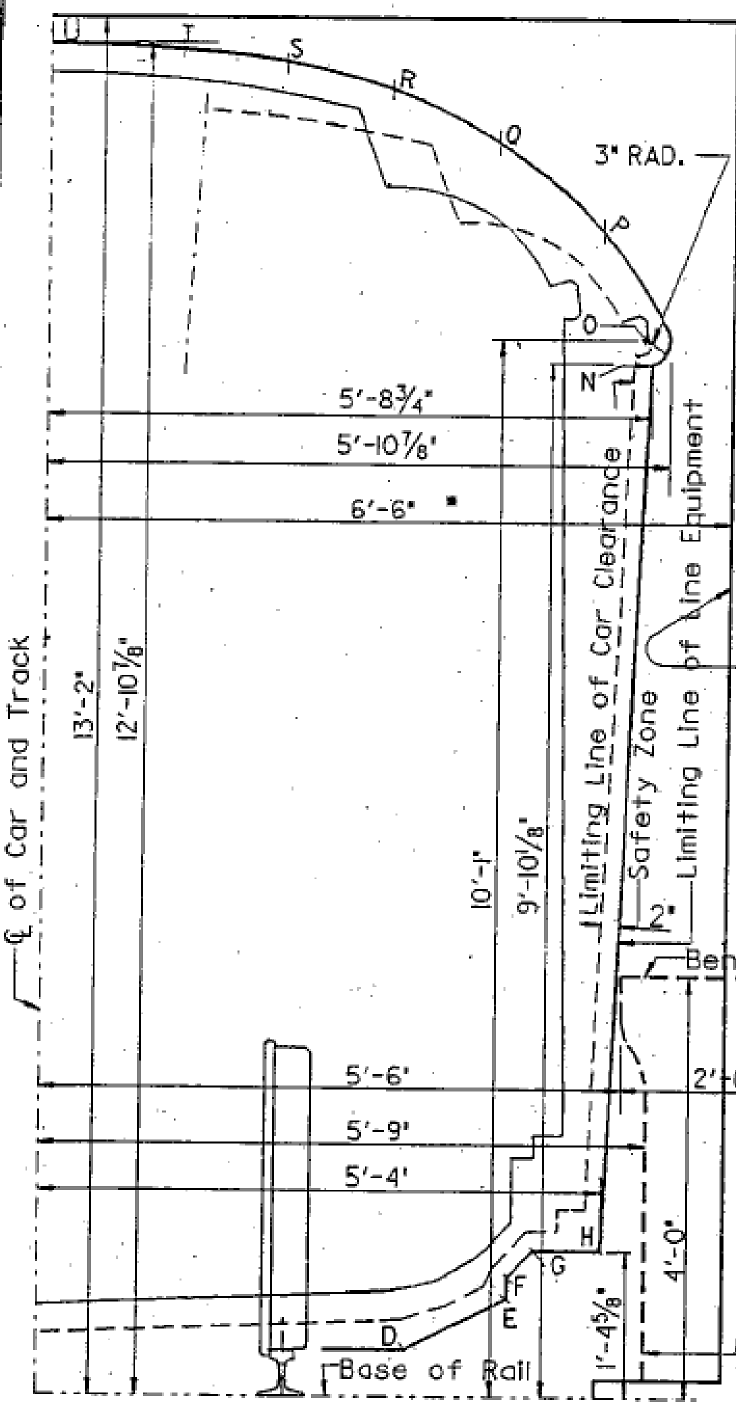
Full superelevation assumed at P.C., tapering 1 in. in every 40 feet. This presupposes that a sufficient length of tangent track is available for the run-off.

The entire car is assumed tilted same amount as front truck. The table does not apply to sharp curves, where transitions are required.

* Top of Duct Bench is 4'-0" above Base of Rail.

FOR CURRENT CAR CLEARANCES SEE **STANDARD DRAWINGS**

DIAGRAM SHOWING NORMAL CLEARANCE FOR 67ft. CAR (BMT & IND DIVISIONS)



	FROM ℄ OF CAR	FROM BASE OF RAIL
D	3'-6"	0'-5 41/64"
E	4'-5 1/4"	0'-11 1/8"
F	4'-5 1/4"	1'-5/8"
G	4'-8 1/4"	1'-4 5/8"
H	5'-4"	1'-4 5/8"
N	5'-8 3/4"	9'-10 1/8"
O	5'-7 7/8"	10'-1"
P	5'-3"	11'-1 3/8"
Q	4'-3"	11'-11 5/8"
R	3'-3"	12'-5 3/4"
S	2'-3"	12'-8 7/8"
T	1'-3"	12'-10 1/4"
U	0'-0"	12'-10 7/8"

Structural Clearance Line for Intermediate Walls and Columns and for Side Walls without Duct Bench (Maximum Encroachment of 3" Permitted along Intermediate Walls and Columns at Isolated Points. No Encroachment Permitted at Sidewalls.)

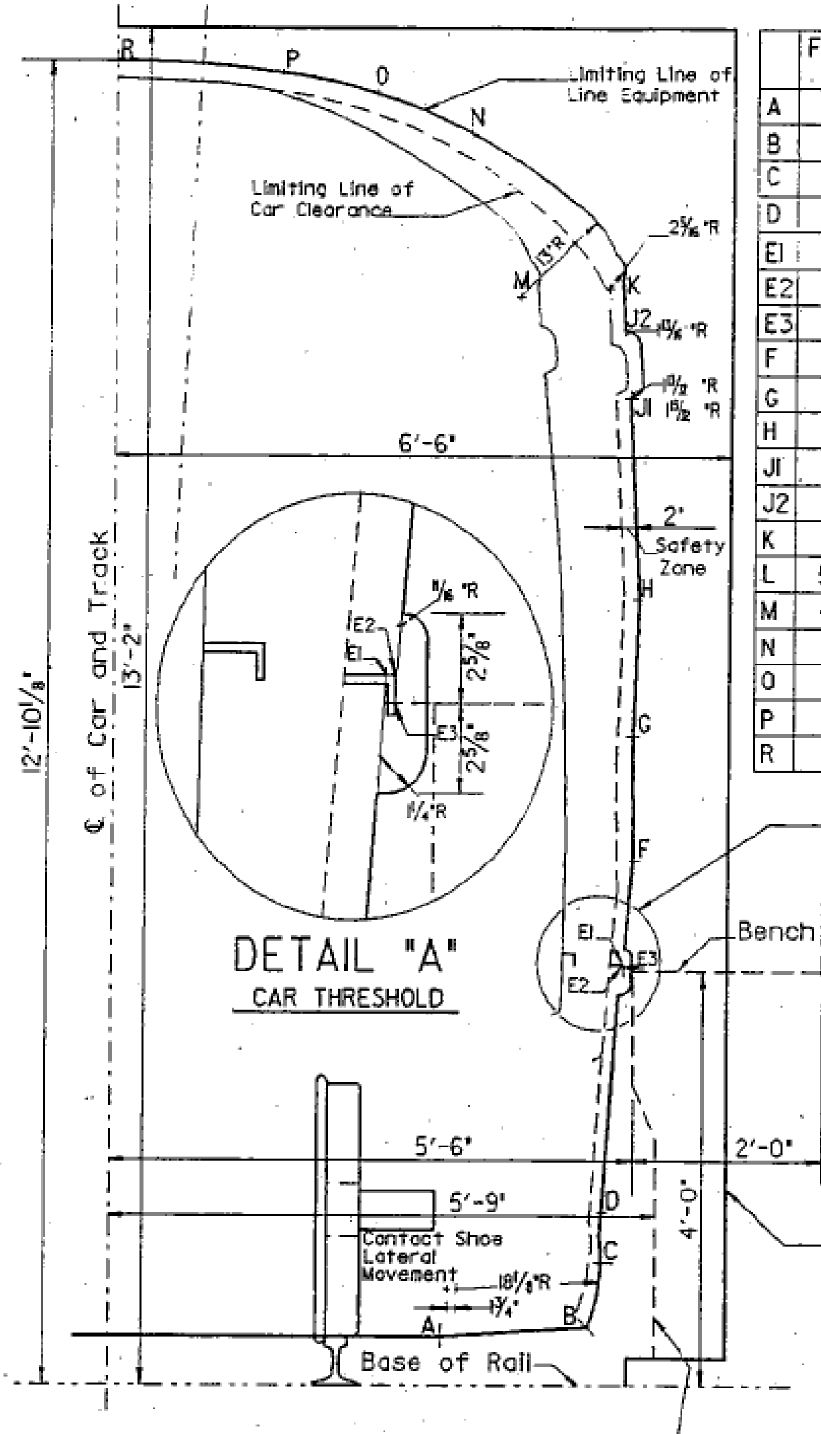
* Where Safety Niches cannot be provided for three Consecutive Bays or 15 feet, Distance from ℄ of Track to Face of Interior Wall shall be increased to 7 feet on Tangent Track. Distance is to be increased as necessary on Curved Track to allow for Excess Clearances. If this Criteria cannot be met, "NO CLEARANCE" Signs are to be posted.

Structural Clearance Line for Sidewall with Duct Bench. (No Encroachment Permitted.)

Note: All vertical dimensions measured from the Base of Rail are established based upon 5 41/64" height for 100 LB. Rail.

FOR CURRENT CAR CLEARANCES SEE **STANDARD DRAWINGS**

DIAGRAM SHOWING NORMAL CLEARANCE FOR 75ft. CAR (BMT & IND DIVISIONS)



	FROM ℄ OF CAR	FROM BASE OF RAIL
A	3'-6'	0'-5 41/64"
B	5'-1 1/16"	0'-6 3/4"
C	5'-2 11/16"	1'-2 1/4"
D	5'-2 1/4"	1'-8 1/8"
E1	5'-4 7/16"	4'-0 21/32"
E2	5'-4 13/16"	4'-0 21/32"
E3	5'-4 13/16"	4'-0 9/16"
F	5'-5 7/8"	5'-0 27/32"
G	5'-6 3/4"	6'-3 13/32"
H	5'-6 3/8"	7'-7 7/16"
J1	5'-5 5/32"	9'-6 15/16"
J2	5'-4 3/8"	10'-2 11/16"
K	5'-4 3/16"	10'-6 21/32"
L	5'-2 3/8"	10'-7 25/32"
M	4'-3 1/8"	10'-6 21/32"
N	3'-9"	12'-1 3/4"
O	2'-9"	12'-6 3/8"
P	1'-9"	12'-8 13/16"
R	0'-0"	12'-10 1/8"

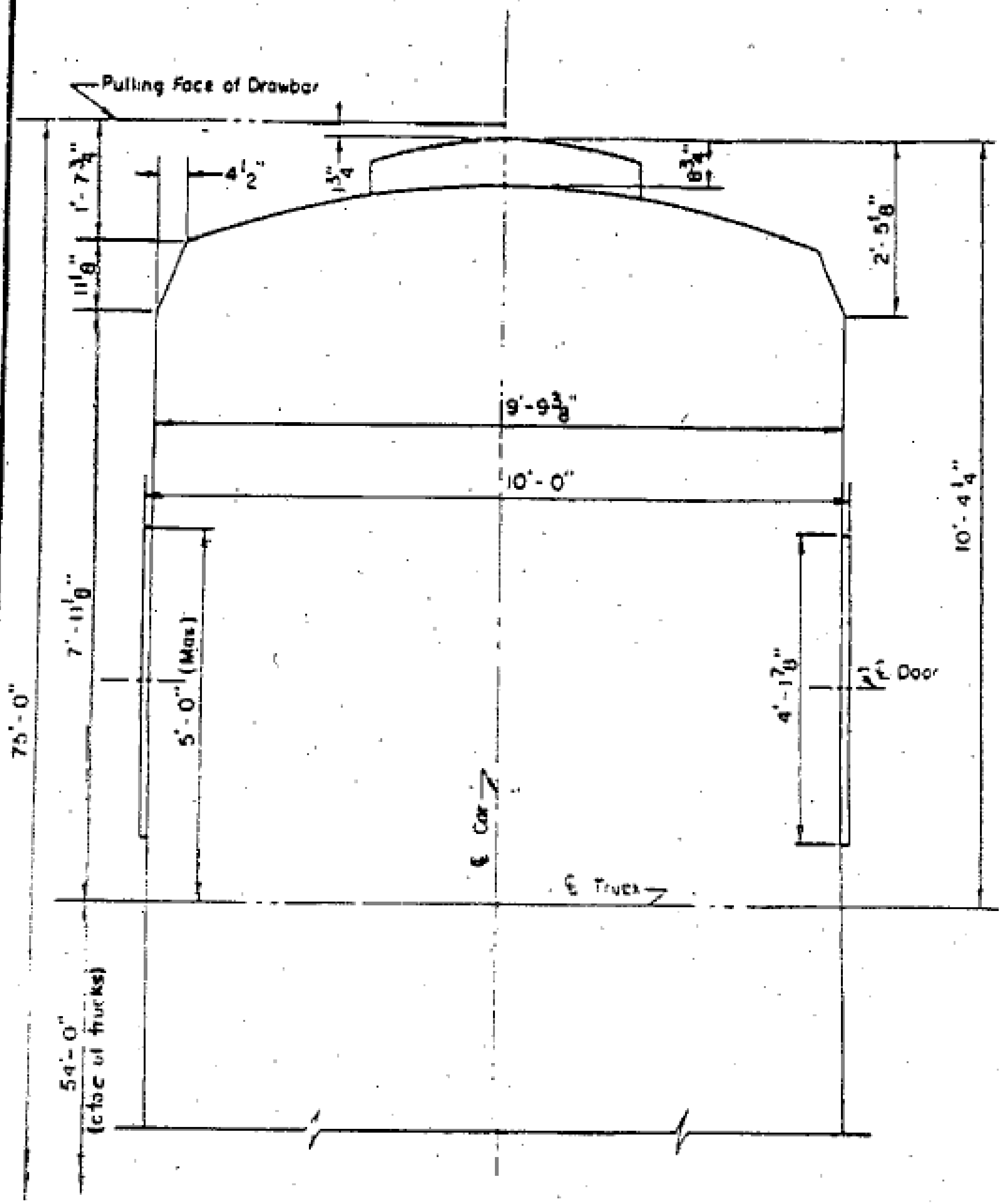
SEE DETAIL "A"

WHERE SAFETY NICHES CANNOT BE PROVIDED FOR THREE CONSECUTIVE BAYS ON 15 FEET, DISTANCE FROM ℄ OF TRACK TO FACE OF INTERIOR WALL SHALL BE INCREASED TO 7 FEET ON TANGENT TRACK. DISTANCE IS TO BE INCREASED AS NECESSARY ON CURVED TRACK TO ALLOW FOR EXCESS CLEARANCES. IF THIS CRITERIA CANNOT BE MET "NO CLEARANCE" SIGNS ARE TO BE POSTED.

STRUCTURAL CLEARANCE LINE FOR INTERMEDIATE WALLS AND COLUMNS AND FOR SIDE WALLS WITHOUT DUCT BENCH (MAXIMUM ENCROACHMENT OF 3" PERMITTED ALONG INTERMEDIATE WALLS AND COLUMNS AT ISOLATED POINTS. NO ENCROACHMENT PERMITED AT SIDEWALLS)

STRUCTURAL CLEARANCE LINE FOR SIDEWALL WITH DUCT BENCH (NO ENCROACHMENT PERMITTED)

NOTE: ALL VERTICAL DIMENSIONS MEASURED FROM THE BASE OF RAIL ARE ESTABLISHED BASED UPON 5 41/64" HEIGHT FOR 100 LB. RAIL.

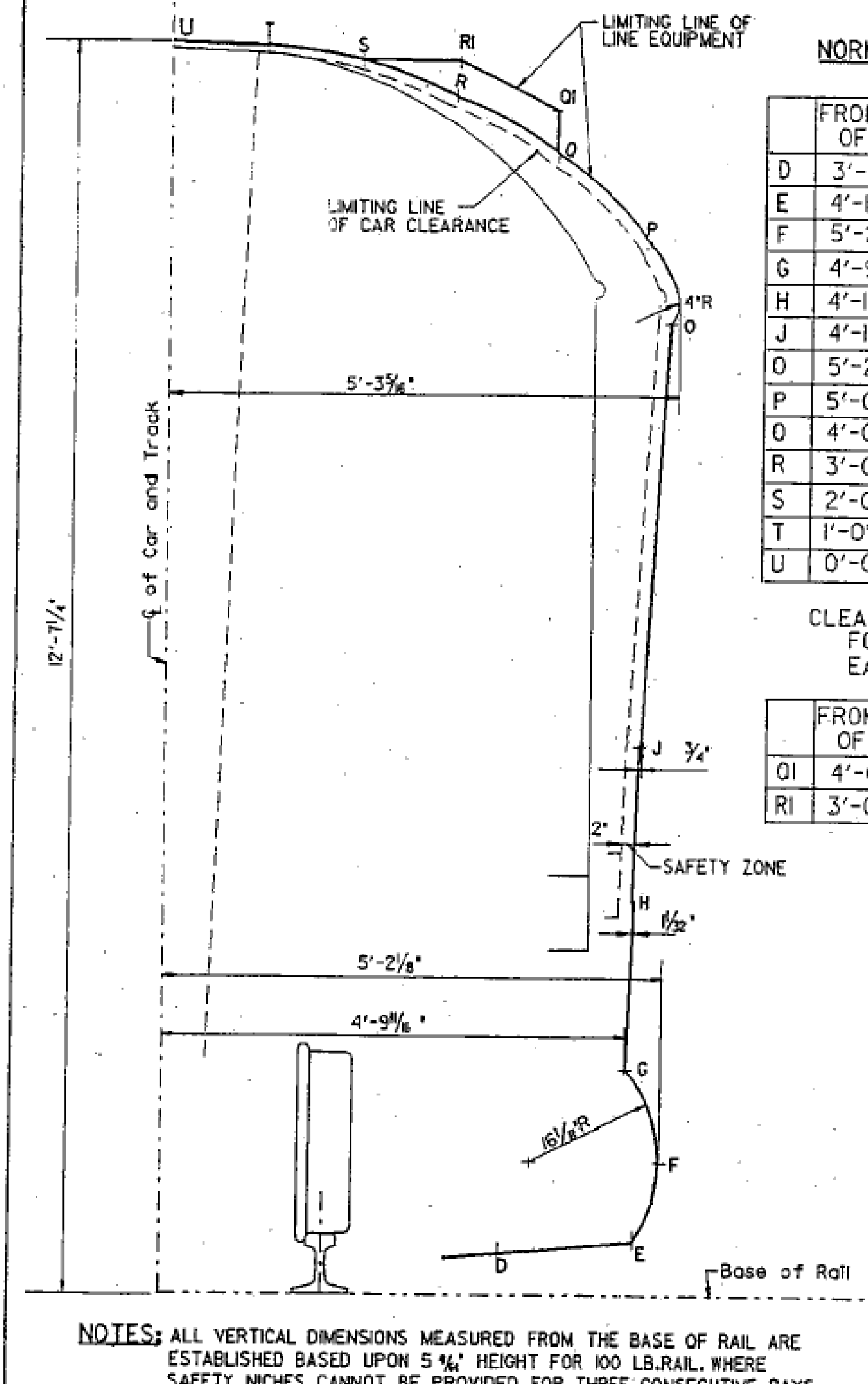


PLAN
(Not to Scale)

GENERAL CONFIGURATION OF 75ft. CAR

FOR CURRENT CAR CLEARANCES SEE **STANDARD DRAWINGS**

DIAGRAM SHOWING NORMAL CLEARANCE FOR IRT DIVISION



NORMAL CLEARANCE

	FROM ℄ OF CAR	FROM BASE OF RAIL
D	3'-6'	0'-5 4/64 "
E	4'-11/16"	0'-6 1/2"
F	5'-2 1/8"	1'-4"
G	4'-9 11/16"	2'-3 1/16"
H	4'-10 3/4"	3'-11 1/4"
J	4'-11 3/8"	5'-6 5/16"
O	5'-2 1/2"	9'-9 1/2"
P	5'-0"	10'-7'
Q	4'-0"	11'-6 1/8"
R	3'-0"	12'-0 3/4"
S	2'-0"	12'-5 5/16"
T	1'-0"	12'-6 15/16"
U	0'-0"	12'-7 1/4"

CLEARANCE MODIFIED FOR 1938 AND EARLIER CARS

	FROM ℄ OF CAR	FROM BASE OF RAIL
Q1	4'-0"	11'-11 1/4"
R1	3'-0"	12'-5 1/2"

NOTES: ALL VERTICAL DIMENSIONS MEASURED FROM THE BASE OF RAIL ARE ESTABLISHED BASED UPON 5 3/4" HEIGHT FOR 100 LB. RAIL. WHERE SAFETY NICHES CANNOT BE PROVIDED FOR THREE CONSECUTIVE BAYS OR 15 FEET, DISTANCE FROM CL. OF TRACK TO FACE OF INTERIOR WALL SHALL BE 7 FEET ON TANGENT TRACK. DISTANCE IS TO BE INCREASED BY REQUIRED AMOUNTS ON CURVED TRACK TO ALLOW FOR EXCESS CLEARANCES. IF THIS CRITERIA CANNOT BE MET, "NO CLEARANCE" SIGNS ARE TO BE POSTED.

Chapter 3

CHAPTER 3A

LOADS

TABLE OF CONTENTS

I - DEAD LOADS

1. **DEAD LOAD** CONSISTS OF THE WEIGHT OF THE STRUCTURE PLUS ANY STATIONARY LOAD IT MAY HAVE TO CARRY.

2. **COVER LOAD** ON SUBWAYS IS DEFINED AS THE COMBINED WEIGHT OF THE ROOF CONSTRUCTION AND THE OVERLYING EARTH AND IS CONSIDERED EQUAL TO THAT OF A LAYER OF DRY EARTH OF A DEPTH AS FROM THE UNDERSIDE OF THE TRANSVERSE ROOF MEMBERS TO THE GROUND OR STREET SURFACE. IF THE TOP OF THE ROOF IS BELOW WATER LEVEL, AN ADDITIONAL ALLOWANCE SHALL BE MADE FOR THE GREATER WEIGHT OF SATURATED EARTH. FOR ROOFS WITH CEILING APPROXIMATELY FLUSH WITH THE UNDERSIDE OF THE ROOF BEAM, ADD TO THE COVER LOAD 75 PSF OF ROOF SURFACE FOR EVERY FOOT OF DEPTH OF THE ROOF SLAB.

3. THE WEIGHT OF **MATERIALS** SHALL BE ESTIMATED AS FOLLOWS:

MATERIAL	**WEIGHT (PCF)**
STEEL	490
CAST IRON	450
STONE CONCRETE (REINFORCED)	150
STONE OR ASPHALTIC CONCRETE (PLAIN)	144
EARTH, DRY	100
EARTH, SATURATED	125
ROCK	150
BRICK MASONRY	120
STONE MASONRY	150
CRUSHED STONE, GRAVEL	110
WATER	62.5

4. **LOAD ON TUNNELS**: IN THE DESIGN OF SUBAQUEOUS AND ROCK TUNNELS, EACH CASE SHALL BE CONSIDERED ON ITS OWN MERITS. IN NO CASE SHALL THE LOAD ON TOP OF ROCK TUNNEL ROOFS BE TAKEN AS LESS THAN THAT OF A 10 FOOT LAYER OF ROCK.

5. **ELEVATED STRUCTURES**:

CANOPY ROOF	20 PSF
WOOD FLOORING FOR PLATFORMS AND MEZZANINES	25 PSF
TIES, RAIL AND SERVICE WALK IN OPEN FLOOR STRUCTURES, FOR EACH RAIL	220 PLF

II -TRAIN AXLE LOAD

6. THE **TRAIN LOAD** ON SUBWAY AND ELEVATED TRACKS SHALL BE TAKEN AS A CONTINUOUS TRAIN OF CARS WITH AXLE LOADS OF THE AMOUNTS AND SPACINGS GIVEN IN FIGURES 1A TO 1C. FOR MAXIMUM VALUES OF SHEAR, MOMENT AND FLOOR BEAM REACTION, SEE TABLE 1A FOR A DIVISION (IRT) LOADINGS AND TABLE 1B FOR B DIVISION (IND/BMT) LOADINGS.

7. IN DESIGNING STRUCTURAL MEMBERS, THE EFFECT OF DEAD LOAD AND IMPACT SHALL BE ADDED TO THE VALUES GIVEN IN TABLES 1A AND 1B. WHERE STRINGERS ARE SPLICED FOR MOMENT AT SUPPORTS, SHEARS AND FLOOR BEAM REACTIONS SHALL BE INCREASED AN ADDITIONAL 25 PERCENT.

8. THE TRAIN AXLE LOADS SHOWN IN FIGURES 1A, 1B AND 1C WERE UTILIZED AS THE STANDARD DESIGN LOADINGS FOR NEW YORK CITY TRANSIT STRUCTURES. THE CARS FOR WHICH THESE LOADINGS WERE DEVELOPED ARE IN MOST CASES NO LONGER IN SERVICE. HOWEVER, CURRENT AND ALL FUTURE NON-REVENUE AND PASSENGER CAR DESIGNS MUST BE RESTRAINED IN THE MAGNITUDE AND SPACING OF MAXIMUM AXLE LOADS SO THAT THE SHEARS, MOMENTS AND FLOOR BEAM REACTIONS PRODUCED BY THESE LOADS IN SUBWAY AND ELEVATED STRUCTURES DO NOT EXCEED THE VALUES PRODUCED BY THE STANDARD DESIGN LOADINGS.

9. FOR STRUCTURAL REHABILITATION OR MODIFICATION OF ALL STRUCTURES, EACH STRUCTURAL MEMBER AS A MINIMUM SHALL BE REPLACED IN KIND OR BY AN EQUIVALENT MEMBER AND IN ADDITION, THE STRUCTURES SHALL BE ANALYZED FOR THE RESPECTIVE DIVISION LOADINGS AND MEMBERS OF INCREASED STRENGTH SHALL BE PROVIDED, IF REQUIRED.

10. WHERE THE STRUCTURE SUPPORTS RAILROAD TRAINS, THE DESIGN SHALL BE IN ACCORDANCE WITH THE REQUIREMENTS OF THE RAILROAD COMPANY CONCERNED, PROVIDED THOSE OF THE NEW YORK CITY TRANSIT ARE NOT MORE SEVERE.

11. INTERMEDIATE TRACK FLOORS IN SUBWAYS WITH BEAMS PLACED TRANSVERSELY, SHALL BE DESIGNED FOR A LIVE LOAD OF 1100 PSF STATIC EQUIVALENT, APPLIED OVER A WIDTH OF 10 FEET SYMMETRICAL TO THE CENTERLINE OF TRACK, IN ADDITION TO THE DEAD LOAD, AS SHOWN IN FIGURE 2. THE DIRECT COMPRESSION DUE TO SIDE PRESSURE SHALL BE CONSIDERED IN DESIGNING TRACK FLOOR AND OTHER INTERMEDIATE BEAMS. SPECIAL TRACK FLOOR CONSTRUCTION AT CROSSINGS, WITH RAIL SUPPORTS PLACED LONGITUDINALLY, SHALL BE DESIGNED IN ACCORDANCE WITH TABLES 1 AND 2.

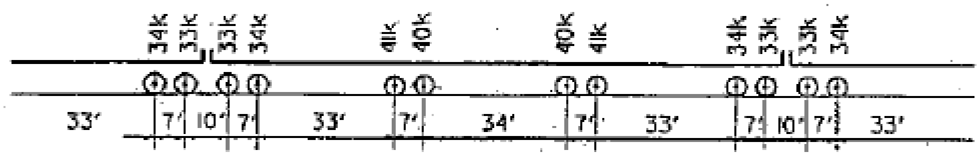


FIGURE 1A
(BMT / B DIVISION CAR LOADING)

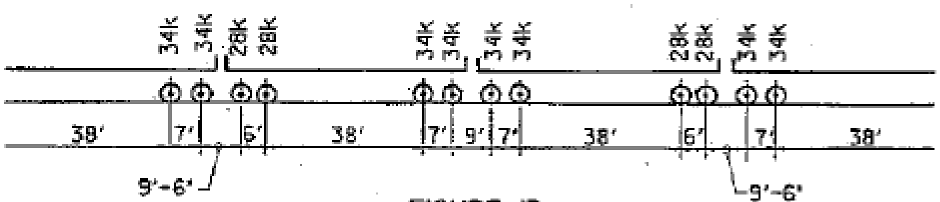


FIGURE 1B
(IND / B DIVISION CAR LOADING)

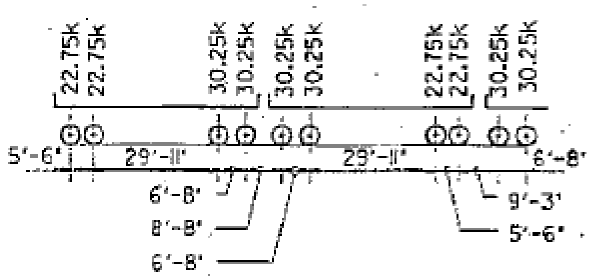


FIGURE 1C
(IRT / A DIVISION CAR LOADING)

FIGURE 1

TABLE 1A

"A" DIVISION (IRT) CAR LOADING
SHEARS, BENDING MOMENTS AND FLOOR BEAM REACTIONS
FOR STRINGERS ON STRAIGHT TRACK DUE TO TRAIN LOAD ON ONE RAIL
(IMPACT NOT INCLUDED)

SPAN FT.	SHEAR K.	MOMENT FT.-K.	FBR* K.
6	15.13	22.69	15.13
8	17.65	30.26	17.65
10	20.17	37.83	22.18
12	21.85	47.33	26.05
14	23.05	61.46	28.81
16	24.58	75.84	31.51
18	26.89	90.39	34.73
20	28.74	111.16	37.31
22	30.25	133.83	39.42
24	32.77	156.50	41.18
26	34.90	179.18	42.67
28	36.73	201.86	43.94
30	38.32	231.28	45.05
32	39.71	260.95	46.01
34	40.93	290.69	46.87
36	42.02	320.48	47.63
38	42.99	350.33	48.73
40	43.87	380.21	49.89
42	44.66	410.14	50.93
44	45.38	440.09	52.38

SPAN FT.	SHEAR K.	MOMENT FT.-K.	FBR* K.
46	46.04	470.07	53.91
48	46.64	500.07	55.61
50	47.20	530.09	57.17
52	47.73	560.13	59.07
54	48.62	590.19	61.37
56	49.45	620.26	63.51
58	50.34	650.35	65.50
60	51.44	680.44	67.85
62	52.46	710.55	70.55
64	53.43	740.67	73.08
66	54.33	770.79	75.45
68	55.48	800.93	77.98
70	56.70	831.07	80.51
75	59.82	908.93	86.25
80	63.18	1000.85	91.27
85	66.14	1104.07	95.69
90	68.77	1207.38	99.89
95	71.13	1322.22	104.19
100	73.25	1458.94	108.83

* For two adjacent spans of equal length.

ADJACENT SPANS FT.	FBR K.
30 and 35	46.78
30 and 40	48.07
30 and 45	49.08
30 and 50	51.25
30 and 55	54.06
30 and 60	57.25
30 and 65	60.65
30 and 70	63.56
30 and 75	66.09
30 and 80	68.30
30 and 85	70.50
30 and 90	72.70
30 and 95	74.67
30 and 100	77.04

ADJACENT SPANS FT.	FBR K.
35 and 35	47.26
35 and 40	48.59
35 and 45	50.76
35 and 50	52.97
35 and 55	55.79
35 and 60	58.98
35 and 65	62.38
35 and 70	65.29
35 and 75	67.82
35 and 80	70.03
35 and 85	71.98
35 and 90	73.97
35 and 95	76.29
35 and 100	78.77

TABLE 1B

"B" DIVISION (IND/BMT) CAR LOADING
SHEARS, BENDING MOMENTS AND FLOOR BEAM REACTIONS
FOR STRINGERS ON STRAIGHT TRACK DUE TO TRAIN LOAD ON ONE RAIL
(IMPACT NOT INCLUDED)

SPAN FT.	SHEAR K.	MOMENT FT.-K.	FBR* K.
6	20.50	30.75	20.50
8	23.00	41.00	23.00
10	26.50	51.25	26.50
12	28.83	61.58	28.83
14	30.50	80.39	31.57
16	31.75	99.56	34.00
18	32.72	118.97	37.78
20	33.50	138.55	40.80
22	34.14	158.25	43.27
24	35.42	178.04	45.33
26	37.92	197.90	47.08
28	40.07	221.20	48.57
30	41.93	249.48	49.87
32	43.56	282.76	51.00
34	45.00	316.13	52.00
36	46.28	349.56	52.89
38	47.42	383.06	53.86
40	48.45	416.61	54.40
42	49.38	450.20	55.05
44	50.23	483.82	55.84

SPAN FT.	SHEAR K.	MOMENT FT.-K.	FBR* K.
46	51.00	517.48	57.62
48	51.71	551.17	59.99
50	52.36	584.89	61.89
52	52.96	618.62	63.64
54	53.52	652.37	65.81
56	54.04	686.15	68.04
58	54.52	719.94	70.45
60	54.97	753.74	73.03
62	55.69	787.55	75.45
64	56.68	821.38	77.72
66	58.22	855.22	79.85
68	59.67	889.06	81.85
70	61.04	922.92	83.74
75	64.13	1007.59	88.03
80	66.84	1092.30	91.78
85	69.24	1177.05	96.21
90	71.36	1304.72	101.53
95	74.27	1436.33	106.29
100	77.06	1568.22	111.49

* For two adjacent spans of equal length.

ADJACENT SPANS FT.	FBR K.
30 and 35	51.89
30 and 40	53.41
30 and 45	54.59
30 and 50	55.53
30 and 55	58.20
30 and 60	61.05
30 and 65	63.46
30 and 70	66.12
30 and 75	68.61
30 and 80	70.80
30 and 85	73.06
30 and 90	76.00
30 and 95	78.63
30 and 100	81.00

ADJACENT SPANS FT.	FBR K.
35 and 35	52.46
35 and 40	53.98
35 and 45	55.16
35 and 50	56.95
35 and 55	60.37
35 and 60	63.21
35 and 65	65.62
35 and 70	67.69
35 and 75	69.48
35 and 80	72.37
35 and 85	75.04
35 and 90	77.42
35 and 95	79.55
35 and 100	82.00

III - IMPACT

12. IMPACT IS CONSIDERED FOR TRAINS ONLY. FOR SUBWAY AND ELEVATED STRUCTURES, THE TRAIN LOADS SPECIFIED IN SECTION 6 SHALL BE INCREASED BY A PERCENTAGE I, AS GIVEN BY THE FOLLOWING FORMULA:

$$I = \frac{150-(L/6)*100}{(450+L)} \quad \text{--} \quad (1)$$

WHERE: I = INCREASE IN PERCENT OF THE LIVE LOAD ON A SINGLE TRACK.

L = LENGTH OF SPAN IN FEET.

FOR MEMBERS SUPPORTING SEVERAL TRACKS, SUCH AS CROSS GIRDERS AND COLUMNS, L = LENGTH OF ADJACENT SPANS FOR ONE TRACK ONLY. SEE TABLE 2 FOR VALUES OF I.

13. WHERE A MEMBER SUPPORTS MORE THAN ONE TRACK THE NUMBER OF TRACKS ASSUMED LOADED SHALL BE SUCH AS WILL PRODUCE THE MAXIMUM STRESS IN THE MEMBER, BUT THE IMPACT INCREASE SHALL BE APPLIED ONLY TO THAT TRACK WHICH, WHEN LOADED, CONTRIBUTES MOST TO THE LIVE LOAD STRESS.

14. IN DESIGNING STEEL MEMBERS OF STRUCTURES SUPPORTING ELEVATED COLUMNS, USE ACTUAL COMPUTED LOADS AND APPLY IMPACT FORMULAS AND STRESSES AS SPECIFIED HEREIN.

15. THE LIVE LOAD AND IMPACT ON HIGHWAY BRIDGES SHALL BE TAKEN FROM THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO) STANDARD SPECIFICATION FOR HIGHWAY BRIDGES OR THOSE SPECIFIED BY THE AGENCY HAVING JURISDICTION IF MORE SEVERE.

TABLE 2

IMPACT INCREASES (%) FOR TRAIN LOADS
AS DETERMINED FROM FORMULA (1)

L	I
5	33
10	32
15	32
20	31
25	31
30	30

L	I
40	29
50	28
60	28
70	27
80	26
90	25

L	I
100	24
150	21
200	18
250	16
300	14
350	12

L	I
400	10
500	7
600	5
700	3
800	2
900	0

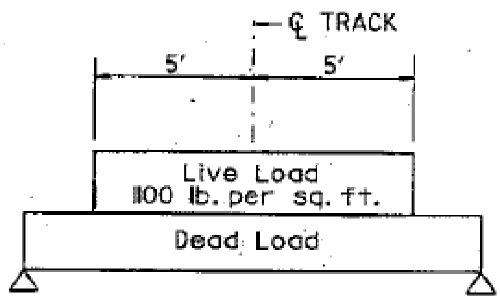


FIGURE 2

IV - TRACTIVE FORCE

16. **TRACTIVE FORCE** SHALL BE TAKEN AT 200 PLF OF TRACK FOR A LENGTH EQUAL TO THE DISTANCE BETWEEN EXPANSION JOINTS AND SHALL BE ASSUMED TO STRESS THE COLUMNS RESISTING SUCH TRACTIVE FORCE IN BENDING ONLY. IN STRUCTURES SUPPORTING SEVERAL TRACKS, THE TRACTIVE FORCE DUE TO ALL THE TRACKS SHALL BE CONSIDERED AT EXPRESS STATIONS ONLY. AT LOCAL STATIONS AND BETWEEN STATIONS, CONSIDER FORCE DUE TO 1-2/3 TRACKS FOR A TWO TRACK LINE, 2 TRACKS FOR A 3 TRACK LINE AND 2-1/3 TRACKS FOR A FOUR TRACK LINE. THE FORCE PER COLUMN SHALL BE TAKEN AS THE TOTAL TRACTIVE FORCE DIVIDED BY THE NUMBER OF COLUMNS RESISTING IT, EXCEPT THAT WHERE THERE IS A CONSIDERABLE DIFFERENCE IN THE STIFFNESS OF COLUMNS, THE FORCE RESISTED BY EACH COLUMN SHALL BE ASSUMED TO BE IN PROPORTION TO ITS STIFFNESS WHICH IS DEFINED AS MOMENT OF INERTIA DIVIDED BY LENGTH.

V - CENTRIFUGAL FORCE

17. WHERE PROPER SUPERELEVATION IS PROVIDED, THE CENTRIFUGAL FORCE SHALL BE CONSIDERED AS STRESSING COLUMNS (IN BENDING ONLY), BRACING AND STEEL COLUMN BASES, AND SHALL BE ASSUMED TO ACT AT THE LEVEL OF THE BASE OF RAIL IN THE DIRECTION OUTWARD AND RADIAL TO THE CURVE. IN THIS CASE, ONLY ITS HORIZONTAL EFFECTS NEED BE CONSIDERED.

    WHERE NO SUPERELEVATION IS PROVIDED, THE CENTRIFUGAL FORCE SHALL BE ASSUMED TO ACT 5 FEET ABOVE THE BASE OF RAIL AND THE RESULTING VERTICAL FORCES SHALL BE TAKEN INTO ACCOUNT IN DESIGNING STRINGERS AND COLUMNS.

    IN COMPUTING STRESSES, ASSUME TRAINS ON ALL TRACKS AND USE THE FOLLOWING FORMULA:

F = CW……………………………………………………………….(2)

WHERE: F = CENTRIFUGAL FORCE (KIPS)

C = COEFFICIENT DEPENDING ON DEGREE OF CURVATURE, AS PER TABLE 3

W = WEIGHT OF TRAIN, ASSUMED AS 2 KLF OF EACH TRACK MEASURED BETWEEN CENTERLINES OF SPANS.

TABLE 3

DEGREE OF CURVATURE	RADIUS FT.	C
1	5730	0.020
2	2865	0.040
3	1910	0.060
4	1433	0.076
5	1146	0.090
6	955	0.102
7	819	0.112
8	717	0.120
9	637	0.126
10	574	0.130

DEGREE OF CURVATURE	RADIUS FT.	C
11	522	0.132
12	478	0.132
13	442	0.130
14	410	0.126
15	383	0.120
16	359	0.112
17	338	0.102
18	320	0.090
19	303	0.076
20	288	0.060

CENTRIFUGAL FORCE SHALL BE NEGLECTED FOR CURVES OF LESS THAN 1 DEGPEE,

WHILE FOR CURVES EXCEEDING 20 DEGREES, THE VALUE OF C SHALL BE TAKEN AS 0.060.

VI - WIND LOADS

18. WIND FORCE SHALL BE COMPUTED AT 30 PSF ON THE VERTICAL PROJECTION OF EXPOSED SURFACE. FOR TWO AND THREE TRACK STRUCTURES, THE EXPOSED SURFACE OF 1 TRAIN, FOR FOUR TRACK STRUCTURES THAT OF 1-1/4 TRAINS AND FOR FIVE TRACK STRUCTURES THAT OF 1-1/2 TRAINS SHALL BE INCLUDED. THE WIND FORCE ON TRAINS SHALL BE CONSIDERED AS APPLIED 6 FEET ABOVE THE BASE OF RAIL. WHERE WIND SCREENS AT PLATFORMS ARE CONSIDERED AS PART OF THE EXPOSED SURFACE, THE WIND FORCE ON ONE TRAIN SHALL BE DISREGARDED.

VII - TEMPERATURE EFFECTS

19. IN THE DESIGN AND ANALYSIS OF STEEL STRUCTURES, PROVISION SHALL BE MADE FOR THE STRESSES OR MOVEMENTS RESULTING FROM A VARIATION IN TEMPERATURE FROM -10 TO + 110 DEGREES FAHRENHEIT. FOR CONCRETE STRUCTURES, SUCH AS VIADUCTS, PROVISION SHALL BE MADE FOR A VARIATION FROM +20 TO +80 DEGREES FAHRENHEIT.

20. THE COEFFICIENTS OF LINEAR EXPANSION FOR COMMON CONSTRUCTION MATERIALS ARE PROVIDED IN TABLE 4.

TABLE 4

MATERIAL	COEFFICIENT OF LINEAR EXPANSION PER UNIT OF LENGTH, PER DEGREE F
STEEL, MILD (STRUCTURAL)	6.5×10^{-6}
STEEL, STAINLESS	9.6×10^{-6}
IRON, CAST, GRAY	6.0×10^{-6}
CONCRETE, NORMAL WEIGHT	5.5 to 7.0×10^{-6}

VIII - LOADING COMBINATIONS

21. IN THE DESIGN AND ANALYSIS OF ELEVATED STRUCTURES, THE FOLLOWING LOADING COMBINATIONS AND PERCENTAGES OF ALLOWABLE STRESS ARE TO BE UTILIZED:

NOTATION:

- D = DEAD LOAD
- L = LIVE LOAD, INCLUDING IMPACT
- F = CENTRIFUGAL FORCE
- T = TRACTIVE FORCE
- W = WIND
- TP = TEMPERATURE

LOADING COMBINATIONS:

CASE	LOADING	% OF ALLOWABLE STRESS NEW MATERIAL	% OF ALLOWABLE STRESS DAMAGED MATERIAL
I	D+L+F	100	125
II	D+L+T	125	150
III	D+L+F+W	125	150
IV	D+L+W+1/2 TP	150	170
V	D+T+TP	150	170

22. IN THE DESIGN FOR NEW MATERIALS, THE PROVISIONS OF PARAGRAPH 9, REQUIRING INSTALLATION OF, EQUIVALENT STRENGTH MEMBERS AS A MINIMUM, IS TO BE CONSIDERED.

23. THE CRITERIA FOR DAMAGED MATERIALS IS TO BE APPLIED ONLY AS AN INTERIM STEP WHERE FREQUENT INSPECTIONS AND EARLY REPAIRS ARE ANTICIPATED.

IX - DESIGN LIVE LOADS

24. THE DESIGN LIVE LOAD ON SURFACES OF VARIOUS SUBWAY AND ELEVATED STRUCTURES ARE TABULATED IN TABLE 5.

25. THE ENGINEER MUST CHECK WITH THE APPROPRIATE DEPARTMENT OR DIVISION WHETHER THERE HAVE BEEN ANY MAJOR CHANGES IN THE EQUIPMENT DESIGN, WITH RESPECT TO SIZE, WEIGHT, METHOD OF SUPPORT AND VIBRATION OR SHOCK LOADING CHARACTERISTICS SUBSEQUENT TO THE DERIVATION OF THESE DESIGN LIVE LOADS.

26. IN THE EVALUATION OF EXISTING STRUCTURES FOR PLACEMENT OF NEW EQUIPMENT OR CHANGES IN FUNCTION, THE INITIAL ANALYSIS IS TO BE CARRIED OUT UTILIZING THE LIVE LOADS IN TABLE 5. IF THIS ANALYSIS INDICATES AN UNACCEPTABLE OVERSTRESS, A MORE EXACT ANALYSIS, INCLUDING SITE SPECIFIC LOAD INTENSITIES AND LOCATIONS MAY BE UTILIZED. ANY AREA SO DESIGNED MUST BE CLEARLY POSTED WITH SIGNS INDICATING LOADING LIMITATIONS.

TABLE 5

DESIGN LIVE LOAD

U = UNIFORM LOAD IN PSF AND C = CONCENTRATED LOAD IN KIPS

NO.	DESCRIPTION	U	C	NOTE
1	CANOPY ROOF	30	0	1
2	SERVICE WALK	150	0	1
3	STAIRS, ON HORIZONTAL PROJECTION	150	0	1
4	PLATFORMS & MEZZANINES			
	(a) ELEVATED	100	0	1
	(b) SUBWAY	150	0	-
5	CHILLER ROOM	150	15	2
6	AIR COOLING UNIT ROOM	150	1	2
7	FAN AREA	150	5	2, 3
8	CONTROL ROOM	150	0	-
9	ELEVATOR MACHINE ROOM	-	-	4
10	ELEVATOR PIT	-	-	4
11	ESCALATOR MACHINE ROOM	150	2	2, 5
12	ESCALATOR PIT	150	0	5
13	EJECTOR ROOM	150	1	2
14	PUMP ROOM	250	2	2
15	SUMPS	-	-	6
16	CIRCUIT BREAKER HOUSE	200	2	2
17	ELECTRICAL DISTRIBUTION ROOM	250	5	2
18	ELECTRICAL PANEL ROOM	150	0	-
19	RELAY ROOM	150	1	2
20	CENTRAL INSTRUMENT ROOM	150	1	2
21	SIGNAL TOWER CONTROL ROOM	150	1	2
22	COMMUNICATION ROOM	150	0	2
23	TELEPHONE COMPARTMENT ROOM	150	0	-
24	COMPRESSOR ROOM	150	0	-
25	SUBSTATION			
	TRANSFORMER AREA	300	15	2
	CIRCUIT BREAKER PLATFORM	300	6	2
26	TRACK LUBRICATION ROOM	150	0	-
27	VARIOUS QUARTERS	150	0	7
28	SUBWAY STORAGE SPACES	400	0	-
29	MAINTENANCE SERVICE ROOMS & DUCT MANHOLES	150	0	-
30	PASSAGEWAYS	150	0	8

NOTES ON TABLE 5:

1. IN DESIGNING ELEVATED COLUMNS, USE ONE-HALF OF THE UNIFORMLY DISTRIBUTED LIVE LOAD.

2a. IN DESIGNING FLOOR SLABS (ONE-WAY OR TWO-WAY) OR FLOOR BEAMS, USE THE UNIFORMLY DISTRIBUTED LIVE LOAD OVER THE ENTIRE FLOOR AREA PLUS THE CONCENTRATED LIVE LOAD LOCATED SO AS TO PRODUCE: (1) MAXIMUM SHEAR, AND (2) MAXIMUM MOMENT. FOR ONE-WAY SLABS, APPLY THE DISTRIBUTED LIVE LOAD PLUS THE CONCENTRATED LIVE LOAD TO A SLAB WITH A WIDTH THAT IS TWICE THE EFFECTIVE DEPTH.

2b. IN THE DESIGN OF COLUMNS, USE ONLY THE UNIFORMLY DISTRIBUTED LIVE LOAD.

3. THIS LOADING IS TO BE USED FOR UNDER PLATFORM EXHAUST FAN ROOMS, FAN CHAMBERS, FAN WORK AREAS AND ANY OTHER AREAS SUPPORTING SIMILAR SIZE FANS.

4a. DESIGN LIVE LOADS MUST BE DETERMINED FOR EACH LOCATION IN CONSULTATION WITH THE MECHANICAL DESIGN DISCIPLINE.

4b. A MINIMUM UNIFORMLY DISTRIBUTED LIVE LOAD OF 150 PSF SHALL BE USED ON ALL FLOORS.

4c. THE EQUIPMENT LIVE LOAD SHALL BE INCREASED SUFFICIENTLY TO PROVIDE FOR IMPACT. IF NOT OTHERWISE SPECIFIED, THIS INCREASE SHALL BE TAKEN AS 100 PERCENT.

5. DESIGN LIVE LOADS GIVEN ARE FOR ESCALATORS WITH A MAXIMUM RISE OF 33 FEET. FOR LONGER ESCALATORS, NOTE 4 APPLIES.

6. DESIGN LIVE LOADS MUST BE DETERMINED ON THE BASIS OF MAXIMUM HYDROSTATIC PRESSURE (AND EXTERNAL EARTH PRESSURE, WHERE APPLICABLE).

7. THIS DESIGN LIVE LOAD APPLIES TO QUARTERS OF THE FOLLOWING PERSONNEL, INCLUDING TOOL ROOMS, WORK SHOPS AND "LIGHT" STORAGE ROOMS.

   a. FOREMAN
   b. DISPATCHERS
   c. TRAINMEN
   d. MOTORMEN
   e. 3RD RAILMEN
   f. TRACKMEN

8. PASSAGEWAYS AND OTHER AREAS ON WHICH EQUIPMENT IS TO BE TEMPORARILY SUPPORTED MUST BE DESIGNED FOR THE DESIGN LIVE LOADS OF THE APPROPRIATE ROOMS.

X - SIDEWALK AND ROADWAY LOADS

27. THE SIDEWALK LIVE LOAD OVER SUBWAYS SHALL BE TAKEN AT 600 PSF. THIS VALUE ALSO APPLIES TO SIDEWALK GRATINGS.

28. THE ROADWAY LIVE LOAD OVER SUBWAYS SHALL BE COMPUTED BY EITHER OF THE FOLLOWING METHODS, WHICHEVER GIVES THE MORE SEVERE CONDITION:

    a. FOR UNIFORM LOAD AS GIVEN IN TABLE 6. THIS LIVE LOAD SHALL ALWAYS BE ASSUMED FOR THE DESIGN OF INTERIOR COLUMNS AND LONGITUDINAL ROOF MEMBERS IN SUBWAYS, AS IT ALWAYS GOVERNS WHERE A LARGE AREA IS SUPPORTED.

    b. AS A LOCAL CONCENTRATION OF 200 KIPS ON FOUR WHEELS, 12 FT. BETWEEN AXLES AND 6 FT. GAUGE. EACH OF THESE WHEEL LOADS SHALL BE CONSIDERED DISTRIBUTED OVER AN AREA OF 2 FT. BY 2 FT. ON THE PAVEMENT AND THENCE THROUGH THE SOIL AND ROOF AT A SLOPE OF 1 HORIZONTAL TO 2 VERTICAL.

    TABLE 7 GIVES THE EQUIVALENT TOTAL LOAD PER SQ. FT. WHICH FOR VARIOUS COVERS, SPANS AND SPACINGS OF ROOF BEAMS PRODUCES IN THE LATTER THE SAME MOMENT OR SHEAR AS DOES THE LOCAL CONCENTRATION TOGETHER WITH THE CORRESPONDING DEAD LOAD, FOR VARIOUS COVERS OF DRY EARTH.

29. SUBWAY ROOFS SUPPORTING NO OTHER STRUCTURES SHALL BE DESIGNED FOR THE LOADS SPECIFIED IN TABLES 6 OR 7.

30. IN DETERMINING THE ROOF LOAD CARRIED BY A COLUMN, THE PORTION DIRECTLY OVER THE COLUMN SHALL BE INCLUDED.

31. IN DOUBLE DECK STRUCTURES, WHERE THE UPPER AND LOWER SIDEWALL COLUMNS ARE VERTICALLY ALIGNED, ONE-HALF THE T0TAL LOAD ON THE UPPER COLUMN SHALL BE CONSIDERED GIVEN OVER TO THE LOWER COLUMN, EXCEPT WHERE THE TOTAL EXCEEDS 200 KIPS OR THE BOND VALUE OF THE COLUMN, IN WHICH CASE ALL OF IT SHALL BE CONSIDERED GIVEN OVER. TO THIS SHALL BE ADDED THE INTERMEDIATE FLOOR LOAD. HOWEVER, IN NO CASE SHALL THE LOWER COLUMN BE OF LESS STRENGTH THAN THE UPPER ONE.

TABLE 6

SIDEWALK AND ROADWAY LOAD OVER SUBWAYS (KSF)

COVER FT.	COVER LOAD	LIVE LOAD		TOTAL LOAD	
		SIDEWALK	ROADWAY	SIDEWALK	ROADWAY
2	0.2	0.6	1.3	0.8	(1.5)
3	0.3	0.6	1.2	0.9	(1.5)
4	0.4	0.6	1.1	1.0	(1.5)
5	0.5	0.6	1.0	1.1	1.5
6	0.6	0.6	0.9	1.2	1.5
7	0.7	0.6	0.8	1.3	1.5
8	0.8	0.6	0.7	1.4	1.5
9	0.9	0.6	0.6	1.5	1.5
10	1.0	0.6	0.6	1.6	1.6
11	1.1	0.6		1.7	
12	1.2	0.6		1.8	
13	1.3	0.6		1.9	
14	1.4	0.6		2.0	
15	1.5	0.5		2.0	
16	1.6	0.4		2.0	
17	1.7	0.3		2.0	
18	1.8	0.2		2.0	
19	1.9	0.1		2.0	
20	2.0	0.0		2.0	

NOTES:

1. FOR EACH ADDITIONAL FOOT OF COVER, INCREASE THE TOTAL LOAD BY 0.1 KSF.
2. VALUES IN BRACKETS ARE MINIMUM VALUES AND SHALL BE COMPARED TO THOSE GIVEN IN TABLE 7.
3. FOR ROOFS BELOW WATER OR WITH A DEPRESSED CEILING, INCREASE LOAD AS SPECIFIED IN PARAGRAPH 2.
4. FOR ROOF BEAMS BELOW THE SIDEWALK AT AREAS OF SHALLOW COVER, HIGHER LIVE LOADS SHALL BE CONSIDERED DUE TO THE CONCENTRATED LOAD DISTRIBUTION AS INDICATED IN SECTION 28 (b).

TABLE 7

LOCAL CONCENTRATION AS PER PARAGRAPH 28b
EQUIVALENT LOAD (INCLUDING COVER LOAD) IN KSF

(a) FOR 2 FOOT COVER

	SPAN FT.	**SPACING OF BEAMS (FT.)**				
		1	**2**	**3**	**4**	**5**
VALUES TO BE USED FOR SHEAR	5	3.20	3.20	2.85	2.45	2.20
	6	3.00	3.00	2.65	2.30	2.05
	7	2.75	2.75	2.45	2.10	1.90
	8	2.75	2.75	2.45	2.10	1.85
	9	2.70	2.70	2.45	2.10	1.80
	10	2.70	2.70	2.40	2.05	1.80
	11	2.70	2.70	2.40	2.05	1.80
	12	2.65	2.65	2.35	2.00	1.75
	13	2.55	2.55	2.30	2.00	1.70
	14	2.50	2.50	2.25	1.95	1.70
	15	2.45	2.45	2.15	1.85	1.65
	16	2.35	2.35	2.10	1.80	1.60
	17	2.30	2.30	2.05	1.75	1.55
	18	2.20	2.20	2.00	1.70	1.50
	19	2.15	2.15	1.95	1.65	1.50
	20	2.10	2.10	1.90	1.60	1.50
VALUES TO BE USED FOR MOMENT	5	3.20	3.20	2.85	2.45	2.20
	6	3.00	3.00	2.65	2.30	2.05
	7	2.75	2.75	2.45	2.10	1.90
	8	2.55	2.55	2.30	1.95	1.75
	9	2.35	2.35	2.10	1.80	1.65
	10	2.35	2.35	2.10	1.80	1.60
	11	2.30	2.30	2.05	1.75	1.55
	12	2.25	2.25	2.05	1.75	1.50
	13	2.20	2.20	2.00	1.70	1.50
	14	2.20	2.20	1.95	1.70	1.50
	15	2.15	2.15	1.95	1.65	1.50
	16	2.10	2.10	1.90	1.65	1.50
	17	2.05	2.05	1.85	1.60	1.50
	18	2.00	2.00	1.80	1.55	1.50
	19	1.95	1.95	1.75	1.55	1.50
	20	1.95	1.95	1.75	1.50	1.50

TABLE 7

LOCAL CONCENTRATION AS PER PARAGRAPH 28b
EQUIVALENT LOAD (INCLUDING COVER LOAD) IN KSF

(b) FOR 3 FOOT COVER

	SPAN FT.	**SPACING OF BEAMS (FT.)**				
		1	**2**	**3**	**4**	**5**
VALUES TO BE USED FOR SHEAR AND MOMENT	5	2.30	2.30	2.25	2.05	1.95
	6	2.25	2.25	2.20	2.00	1.90
	7	2.20	2.20	2.15	1.90	1.80
	8	2.15	2.15	2.10	1.90	1.75
	9	2.10	2.10	2.10	1.85	1.70
	10	2.10	2.10	2.10	1.85	1.70
	11	2.10	2.10	2.10	1.85	1.70
	12	2.10	2.10	2.05	1.85	1.65
	13	2.10	2.10	2.05	1.85	1.65
	14	2.05	2.05	2.00	1.80	1.60
	15	2.00	2.00	1.95	1.75	1.60
	16	1.95	1.95	1.90	1.75	1.55
	17	1.90	1.90	1.85	1.70	1.50
	18	1.85	1.85	1.80	1.65	1.50
	19	1.80	1.80	1.75	1.60	1.50
	20	1.75	1.75	1.70	1.55	1.50

(c) FOR 4 FOOT COVER

	SPAN FT.	**SPACING OF BEAMS (FT.)**				
		1	**2**	**3**	**4**	**5**
VALUES TO BE USED FOR SHEAR AND MOMENT	5	1.80	1.80	1.80	1.80	1.80
	6	1.80	1.80	1.80	1.80	1.80
	7	1.80	1.80	1.80	1.75	1.75
	8	1.80	1.80	1.80	1.70	1.70
	9	1.80	1.80	1.80	1.70	1.65
	10	1.80	1.80	1.80	1.70	1.55
	11	1.80	1.80	1.80	1.70	1.55
	12	1.80	1.80	1.80	1.70	1.55
	13	1.80	1.80	1.80	1.70	1.55
	14	1.80	1.80	1.80	1.70	1.55
	15	1.75	1.75	1.75	1.65	1.55
	16	1.70	1.70	1.70	1.60	1.50
	17	1.65	1.65	1.65	1.60	1.50
	18	1.65	1.65	1.65	1.55	1.50
	19	1.60	1.60	1.60	1.55	1.50
	20	1.55	1.55	1.55	1.50	1.50

FOR COVER BETWEEN 4 FT. AND 9 FT., USE 1.50 KSF TOTAL LOAD IRRESPECTIVE OF SPAN AND SPACING OF BEAMS. TABLE 7 SHALL NOT BE USED IN DESIGNING A HEADER BEAM SUPPORTING A NUMBER OF OTHER BEAMS. IN SUCH CASES, WHERE COVER IS LESS THAN 9 FT., THE HEADER BEAM SHALL BE DESIGNED FOR A UNIFORM LOAD OF 1.5 KSF OF AREA SUPPORTED.

32. COLUMN FOOTINGS, GRILLAGES, LONGITUDINAL INVERT DISTRIBUTING GIRDERS AND INTERIOR SUBWAY COLUMNS SHALL BE DESIGNED FOR THE TOTAL LOAD GIVEN IN TABLE 6, TO WHICH SHALL BE ADDED PLATFORM AND MEZZANINE FLOOR LIVE LOAD AT 150 PSF. AND INTERMEDIATE TRACK FLOOR LIVE LOAD AS SPECIFIED IN PARAGRAPH 11, TOGETHER WITH THEIR RESPECTIVE DEAD LOADS. IN CONSIDERING LOADS TRANSMITTED FROM SIDEWALL COLUMNS TO STEEL FOOTINGS, NO DEDUCTION SHALL BE MADE FOR THE LOAD CARRIED BY THE WALL CONCRETE.

33. SUBWAY INVERTS DISTRIBUTING LOADS OVER THE ENTIRE SUBGRADE SHALL BE DESIGNED FOR THE "DEAD LOAD" GIVEN IN TABLE 6, TOGETHER WITH THE WEIGHT OF WALLS AND FLOORS, NOT INCLUDING INVERT, TO WHICH LOADS SHALL BE ADDED THE LIVE LOAD FROM PLATFORM, MEZZANINE AND UPPER TRACKS AT ONE-HALF THE VALUES SPECIFIED IN PARAGRAPHS 11 AND 32, BUT IN NO CASE SHALL INVERTS OF THIS TYPE BE DESIGNED FOR A TOTAL LOAD OF LESS THAN 900 PSF.

34. IN COMPUTING THE SHEAR AND BEARING STRESSES IN STEEL OR CONCRETE AT COLUMN SUPPORTS, THE LOADS TO BE PROVIDED FOR SHALL BE THOSE SPECIFIED IN PARAGRAPH 32.

XI - BUILDING LOAD

35. WHEN THE SUBWAY PASSES UNDER PRIVATE PROPERTY, IT SHALL BE DESIGNED FOR BUILDING LOAD. THE QUESTION AS TO HEIGHT AND CHARACTER OF BUILDING TO BE PROVIDED FOR IN EACH CASE, AS WELL AS THE LOCATION OF WALLS OR LOT LINES WHERE THE SAME HAS NOT ALREADY BEEN DETERMINED SHALL BE BASED UPON ZONING REQUIREMENTS.

36. BUILDING LOADS ON SUBWAYS SHALL BE COMPUTED FROM THE FOLLOWING VALUES, WHICH ARE GIVEN IN PSF, THE DEAD LOADS INCLUDING THE WEIGHT OF PARTITIONS:

CLASSIFICATION	LIVE LOAD	DEAD LOAD
ROOF, ALL TYPES OF BUILDINGS	40	60
FLOORS		
▪ A – APARTMENT HOUSES	40*	100
▪ B – OFFICE BUILDINGS	50*	100
▪ C – THEATRES, CONCERT HALLS	75*	100
▪ D – OTHER PLACES OF ASSEMBLY, SCHOOLS	60*	100
▪ C - LIGHT MANUFACTURING BUILDINGS	75*	100
▪ E - FACTORY BUILDINGS, GARAGES	120*	100
▪ F - STORAGE HOUSES	120*	100
▪ G - HEAVY MANUFACTURING BUILDINGS	250*	130
CELLAR, ALL TYPES OF BUILDINGS	300	
WALLS (12 INCH BRICK)		
▪ WITHOUT OPENINGS	120 LBS./VERT SQ. FT.	
▪ WITH OPENINGS	100 LBS./VERT SQ. FT.	

*LIVE LOAD ON THE TOP FLOOR SHALL BE TAKEN AT 85% OF THE VALUES GIVEN ABOVE, AND THAT ON SUCCEEDING LOWER FLOORS AT 80%, 75% ….. DOWN TO A MINIMUM OF 50% OF THE SAME VALUES. THIS REDUCTION DOES NOT, HOWEVER, APPLY TO STORAGE HOUSES WHERE THE FULL LIVE LOAD SHALL BE ASSUMED FOR ALL THE FLOORS.

37. IN DESIGNING THE SUBWAY FOR UNIFORM BUILDING LOAD, FOOTINGS SUPPORTING PIERS AND COLUMNS ARE GENERALLY ASSUMED TO BE SPREAD OVER AN AREA EQUAL TO ONE-HALF THAT OF THE CORRESPONDING PANELS. THE LOAD PER SQUARE FOOT OF SUBWAY ROOF SHALL THEN BE ASSUMED EQUAL TO TWICE THE TOTAL FROM ROOF AND FLOORS AS DETERMINED FROM PARAGRAPH 36. ON THIS LOAD, WHICH IS ASSUMED TO BE APPLIED TO EVERY SQUARE FOOT OF SUBWAY ROOF WITHIN THE EASEMENT, SHALL BE SUPERIMPOSED THE FOLLOWING ADDITIONAL LOADS:

a. THE DEAD WEIGHT OF FRONT WALL SHALL BE CONSIDERED DISTRIBUTED OVER A STRIP 4 FEET WIDE OF WHICH 1 FOOT IS OUTSIDE THE BUILDING LINE, EXCEPT WHERE THE TOP OF SUBWAY ROOF IS LESS THAN 10 FEET BELOW THE STREET SURFACE WHEN A STRIP 3 FEET WIDE FLUSH WITH THE BUILDING LINE SHALL BE CONSIDERED. EXCEPTION IS ALSO MADE AS PER SUBPARAGRAPH c.

b. THE DEAD WEIGHT OF LOT LINE WALLS SHALL BE DISTRIBUTED OVER STRIPS 3 FEET WIDE FLUSH WITH THE LOT LINES, EXCEPT AS PROVIDED IN SUBPARAGRAPH c.

c. FOR BUILDINGS MORE THAN 6 STORIES HIGH, THE WIDTH OF THE STRIPS REFERRED TO UNDER SUBPARAGRAPHS a AND b, SHALL BE INCREASED BY BEING EXTENDED TO THE MIDDLE OF THE END PANELS.

d. THE LIVE LOAD ON THE CELLAR FLOOR SHALL BE CONSIDERED AS INDICATED IN PARAGRAPH 36.

b. THE WEIGHT OF SUBWAY ROOF AND OVERLYING SOIL SHALL BE TAKEN AT 100 PSF FOR EVERY FOOT BETWEEN THE CELLAR FLOOR LEVEL (USUALLY 10 FEET BELOW STREET LEVEL) AND THE UNDERSIDE OF TRANSVERSE ROOF MEMBERS. WHERE THE TOP OF THE SUBWAY ROOF IS BELOW WATER, 25 PSF SHALL BE ADDED FOR EVERY FOOT OF GROUND WATER HEAD.

TABLE 8 GIVES VALUES OF UNIFORM BUILDING LOAD FOR VARIOUS HEIGHTS OF BUILDINGS IN ACCORDANCE WITH THE ABOVE RULES.

TABLE 8

VALUES FOR BUILDING AND WALL LOADS
WHEN UNIFORMLY DISTRIBUTED OVER SUBWAY ROOF

STORIES	A	B	C	D	E	F	G	H	I
25	6.60	6.90	7.60	7.20	8.85	11.50	13.95	SEE NOTE 3	
24	6.40	6.65	7.30	6.90	8.55	11.05	13.45		
23	6.15	6.40	7.05	6.65	8.20	10.65	12.95		
22	5.90	6.15	6.75	6.40	7.90	10.20	12.45		
21	5.65	5.90	6.50	6.15	7.55	9.75	11.90		
20	5.40	5.65	6.20	5.90	7.25	9.30	11.40		
19	5.20	5.40	5.95	5.60	6.95	8.85	10.90		
18	4.95	5.15	5.65	5.35	6.60	8.45	10.40		
17	4.70	4.90	5.40	5.10	6.30	8.00	9.90		
16	4.45	4.65	5.10	4.85	5.95	7.55	9.35		
15	4.20	4.40	4.85	4.60	5.65	7.10	8.85		
14	4.00	4.15	4.55	4.30	5.35	6.65	8.35		
13	3.75	3.90	4.30	4.05	5.00	6.25	7.85		
12	3.50	3.65	4.00	3.80	4.70	5.80	7.35		
11	3.25	3.40	3.75	3.55	4.35	5.35	6.80		
10	3.00	3.15	3.45	3.30	4.95	4.90	6.30		
9	2.80	2.90	3.20	3.00	3.75	4.45	5.80		
8	2.55	2.65	2.90	2.75	3.40	4.05	5.30		
7	2.30	2.40	2.65	2.50	3.10	3.60	4.80		
6	2.05	2.15	2.35	2.25	2.75	3.15	4.25	2.10	3.35
5	1.80	1.90	2.05	1.95	2.40	2.70	3.70	1.80	2.90
4	1.55	1.60	1.80	1.70	2.05	2.25	3.10	1.50	2.40

NOTE:

1. FOR CLASSIFICATION OF BUILDING TYPES FOR COLUMNS DESIGNATED A THROUGH G SEE PAGE SD-3-21. COLUMNS DESIGNATED H AND I REFER TO "FRONT WALL" AND "LOT LINE WALL" LOADINGS, RESPECTIVELY.

2. COLUMNS A TO G GIVE LOADS IN KSF TO BE USED IN DESIGNING THE SUBWAY FOR UNIFORM BUILDING LOAD AS PER PARAGRAPH 37. THESE INCLUDE TWICE THE TOTAL FROM ROOF AND FLOORS PLUS LIVE LOAD ON CELLAR, BUT SHALL BE INCREASED BY THE WEIGHT OF SUBWAY ROOF AND OVERLYING SOIL AS PER PARAGRAPH 37e.

3. FOR BUILDINGS MORE THAN 6 STORIES HIGH, THE WALL LOADS GIVEN IN TABLE 9 UNDER COLUMNS H AND I RESPECTIVELY, SHALL BE CONSIDERED DISTRIBUTED OVER STRIPS AS SPECIFIED IN PARAGRAPH 37c.

4. FIGURES IN COLUMNS H AND I GIVE THE ADDITIONAL LOADS IN KSF DUE TO BUILDING WALLS DISTRIBUTED OVER STRIPS 4 FEET AND 3 FEET WIDE AS SPECIFIED IN PARAGRAPH 37a AND b RESPECTIVELY, ASSUMING EACH STORY HEIGHT AT 12 FT.

5. WHERE FRONT WALL IS ASSUMED TO BE DISTRIBUTED OVER A STRIP 3 FEET WIDE AS PER PARAGRAPH 37a, THE VALUES GIVEN IN COLUMN H SHALL BE INCREASED BY ONE-THIRD.

38. WHERE PROVISIONS ARE TO BE MADE FOR SPECIFIC COLUMN LOCATIONS, THE COLUMNS SHALL BE PLACED A UNIFORM DISTANCE APART, AS CLOSE TO 20 FT. BY 20 FT. AS CONDITIONS PERMIT, ODD INCHES TO BE TAKEN UP IN THE END PANELS. THE WALL COLUMNS SHALL BE SET BACK ON A CENTERLINE WHICH IS l'-8" FROM THE BUILDING LINE AND 1 -4" FROM LOT LINES, AND THE COLUMN LOADS SHALL BE COMPUTED IN ACCORDANCE WITH PARAGRAPH 36. THE SUBWAY, IN THIS CASE, SHALL BE DESIGNED FOR THE COLUMN LOADS AND FOR THE ADDITIONAL LOADS SPECIFIED IN PARAGRAPH 37d AND e. NO BASE PLATE OR GRILLAGE TO BE PROVIDED BY THE BULDING OWNER SHALL PROJECT BEYOND LOT LINES, OR MORE THAN 1 FT. BEYOND THE BUILDING LINE, AS PROVIDED BY THE BUILDING CODE, BUT WHERE GRILLAGES FOR BUILDING COLUMN SUPPORT ARE INCLUDED IN THE SUBWAY CONSTRUCTION NO SUCH LIMIT IS IMPOSED.

TABLE 9 GIVES VALUES FOR DETERMINING COLUMN CONCENTRATIONS FOR VARIOUS HEIGHTS OF BUILDINGS, TOGETHER WITH THE CORRESPONDING WEIGHTS OF WALLS PER LINEAR FOOT OF LENGTH.

TABLE 9

VALUES FOR BUILDING COLUMN CONCENTRATIONS
AND WEIGHT OF BUILDING WALLS

STORIES	A	B	C	D	E	F	G	H	I
25	3.15	3.30	3.65	3.45	4.30	5.60	6.85	31.20	37.45
24	3.05	3.20	3.50	3.30	4.10	5.40	6.60	30.00	36.00
23	2.95	3.05	3.40	3.20	3.95	5.15	6.35	28.80	34.55
22	2.80	2.95	3.25	3.05	3.80	4.95	6.05	27.60	33.15
21	2.70	2.80	3.10	2.95	3.65	4.75	5.80	26.40	31.70
20	2.55	2.70	2.95	2.80	3.50	4.50	5.55	25.20	30.25
19	2.45	2.55	2.85	2.65	3.30	4.30	5.30	24.00	28.80
18	2.35	2.45	2.70	2.55	3.15	4.05	5.05	22.80	27.35
17	2.20	2.30	2.55	2.40	3.00	3.85	4.80	21.60	25.95
16	2.10	2.20	2.40	2.30	2.85	3.65	4.55	20.40	24.50
15	1.95	2.05	2.30	2.15	2.70	3.40	4.30	19.20	23.05
14	1.85	1.95.	2.15	2.00	2.50	3.20	4.05	18.00	21.60
13	1.75	1.80	2.00	1.90	2.35	2.95	3.80	16.80	20.15
12	1.60	1.70	1.85	1.75	2.20	2.75	3.50	15.60	18.75
11	1.50	1.55	1.75	1.65	2.05	2.55	3.25	14.40	17.30
10	1.35	1.45	1.60	1.50	1.90	2.30	3.00	13.20	15.85
9	1.25	1.30	1.45	1.35	1.70	2.10	2.75	12.00	14.40
8	1.15	1.20.	1.30	1.25	1.55	1.85	2.50	10.80	12.95
7	1.00	1.05	1.20	1.10	1.40	1.65	2.25	9.60	11.55
6	0.90	0.95	1.05	0.95	1.25	1.45	2.00	8.40	10.10
5	0.75	0.80	0.90	0.85	1.05	1.20	1.70	7.20	8.65
4	0.65	0.65	0.75	0.70	0.90	1.00	1.40	6.00	7.20

NOTES:

1. FOR CLASSIFICATION OF BUILDING TYPES FOR COLUMNS DESIGNATED A THROUGH G, SEE PAGE SD-3-21. COLUMNS DESIGNATED H AND I REFER TO "FRONT WALL" AND "LOT LINE WALL" LOADING, RESPECTIVELY.

2. COLUMNS A TO G GIVE BUILDING LOADS IN KSF OF AREA SUPPORTED BY COLUMNS RESTING ON THE SUBWAY AS PER PARAGRAPH 38 WHILE COLUMNS H AND I GIVE ADDITIONAL LOADS IN KLF FOR BUILDING WALLS SUPPORTED BY SUCH EXTERIOR COLUMNS. IN ADDITION TO THESE COLUMN CONCENTRATIONS, THE SUBWAY SHALL BE DESIGNED FOR THE UNIFORM LOADS SPECIFIED IN PARAGRAPHS 37d AND e. HEIGHT OF EACH STORY ASSUMED AT 12 FEET.

XII - LATERAL PRESSURES ON SUBWAYS

39. LATERAL PRESSURE, AS CONSIDERED IN DESIGNING SUBWAY WALLS, IS GENERALLY DUE TO ONE OR MORE OF THE FOLLOWING CONDITIONS:

    a. EARTH ABUTTING AGAINST A VERTICAL PLANE FLUSH WITH BACK OF WALL.

    b. WATER PRODUCING HYDROSTATIC PRESSURE.

    c. LOADS RESTING ON ABUTTING EARTH.

40. FIGURES 3 AND 4 PROVIDE SIMPLIFIED LOADING DIAGRAMS FOR ESTABLISHING LATERAL PRESSURE ON SUBWAY STRUCTURES DUE TO EARTH AND HYDROSTATIC PRESSURES. ALL DIMENSIONS, D, ARE IN UNITS OF FEET AND COMPUTED LATERAL PRESSURES ARE IN UNITS OF PSF.

41. IN ESTABLISHING THE LATERAL PRESSURE DIAGRAMS, THE FOLLOWING GENERAL RULES ARE UTILIZED:

    a. ONLY DEAD LOAD IS CONSIDERED IN DETERMINING LATERAL PRESSURE.

    b. THE LATERAL PRESSURE SHALL AT NO POINT BE TAKEN AT LESS THAN 200 PSF.

    c. IN EARTH ABOVE WATER LEVEL, THE LATERAL PRESSURE IS COMPUTED AS ONE-THIRD THE VERTICAL PRESSURE, ASSUMING THE WEIGHT OF EARTH AS 100 PCF.

    d. IN EARTH BELOW WATER LEVEL, SAME AS ABOVE, AND, IN ADDITION, FULL WATER PRESSURE FROM WATER LEVEL.

    e. IN SOUND ROCK, WHERE CONCRETE CAN BE RAMMED INTO IT AND PRODUCE A SEAL FOR AT LEAST ONE-HALF THE AREA EXPOSED, THE LATERAL PRESSURE IS COMPUTED AS ONE-HALF WATER PRESSURE FROM WATER LEVEL OR TOP OF ROCK WHICHEVER IS HIGHEST. IN UNSOUND ROCK, WHERE NO SUCH SEAL CAN BE OBTAINED, FULL WATER PRESSURE SHALL BE ASSUMED.

    f. IN NO CASE SHALL THE LATERAL PRESSURE WITHIN SOUND ROCK BE TAKEN AS LESS THAN THE PRESSURE COMPUTED FOR THE OVERLYING EARTH AND WATER.

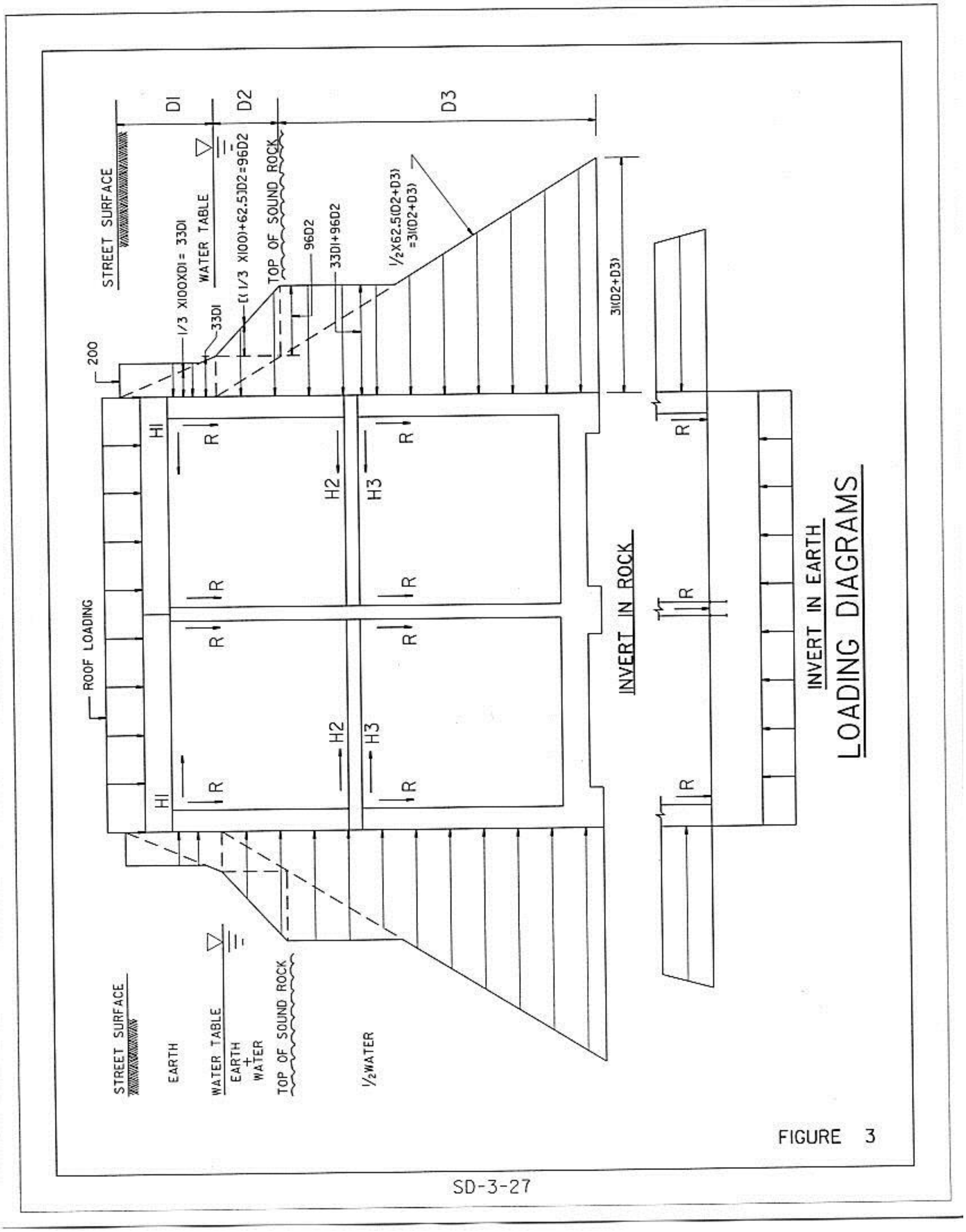
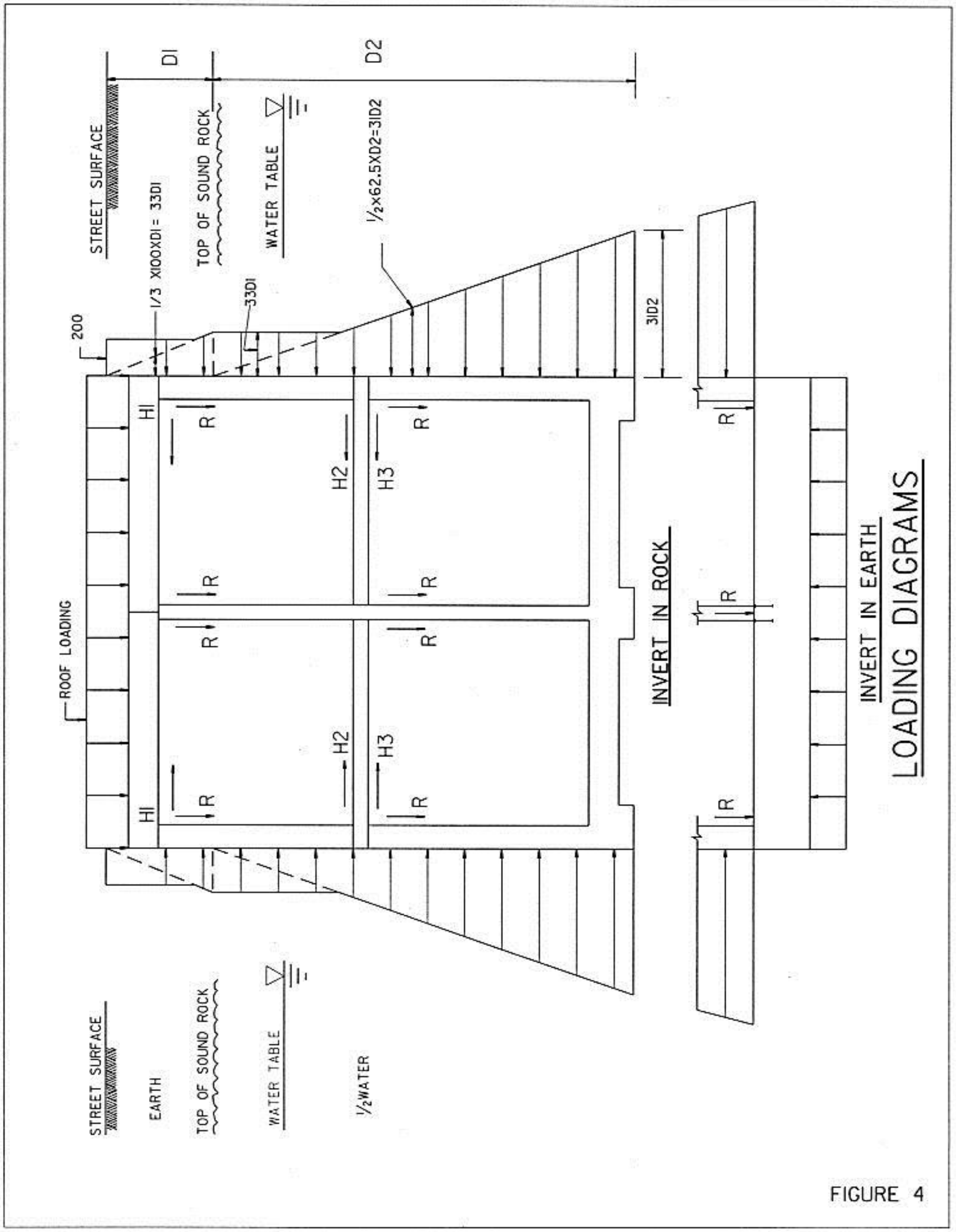


FIGURE 3

SD-3-27


STREET SURFACE
1/3 X100XD1= 33D1
TOP OF SOUND ROCK
WATER TABLE
½x62.5XD2=31D2
D1
D2
200
33D1
31D2
ROOF LOADING
H1
H2
H3
R
INVERT IN ROCK
INVERT IN EARTH
LOADING DIAGRAMS
STREET SURFACE
EARTH
TOP OF SOUND ROCK
WATER TABLE
½WATER
FIGURE 4

42. COMPRESSION DUE TO LATERAL PRESSURE SHALL BE NEGLECTED IN DESIGNING SUBWAY ROOF AND INVERT MEMBERS, BUT SHALL BE CONSIDERED FOR INTERMEDIATE FLOOR BEAMS WHICH SHALL BE DESIGNED AS FOLLOWS:

    WHEN THE REQUIREMENTS OF COMPOSITE ACTION, DESCRIBED IN CHAPTER 4, ARE FULFILLED, ASSUME ONE-HALF THE DIRECT LATERAL PRESSURE AS BEING TAKEN BY THE BEAMS. IN ALL OTHER CASES, ASSUME BEAMS TO TAKE FULL LATERAL PRESSURE.

43. IN EARTH, WHERE THE ACTIVE LATERAL PRESSURES AGAINST OPPOSITE SIDES OF A STRUCTURE ARE UNEQUAL, THE GREATER PRESSURE SHALL BE CONSIDERED FOR BOTH SIDES.

44. IN EARTH, WHERE ADJACENT TRACKS ARE AT DIFFERENT LEVELS, THE FULL THRUST DUE TO LATERAL PRESSURE SHALL BE CONSIDERED TRANSMITTED THROUGH THE ROOF AND INVERT, EXCEPT THAT WHERE AN INVERT INCLUDES ONE OR MORE FOOTINGS OF INTERVENING COLUMNS, THE THRUST AT THE INVERT IS ASSUMED TO BE TAKEN UP BY THE SOIL IN FRICTION.

45. IN ROCK, WHERE AN UPPER TRACK FLOOR PROJECTS BEYOND A LOWER WALL, THE LATERAL PRESSURE SHALL BE COMPUTED IN ACCORDANCE WITH THE RULES GIVEN IN PARAGRAPH 41, WHEN TRACK FLOOR IS SEALED. WHEN WEEP HOLES ARE PROVIDED IN THE TRACK FLOOR, THE WATER LEVEL SHALL BE TAKEN AT THE UPPER BASE OF RAIL.

46. A METHOD FOR EVALUATING THE LATERAL LOADS DUE TO EARTH ABUTTING AGAINST THE SUBWAY STRUCTURE WITH THE ADDITION OF LOADS RESTING ON THE EARTH IS PRESENTED IN PARAGRAPHS 47 THROUGH 53. LATERAL PRESSURE DUE TO WATER, IF ANY, SHALL BE ADDED TO THAT PRODUCED BY EARTH AND SUPERIMPOSED LOADS.

47. THE FOLLOWING DEFINITIONS OF TERMS ARE PROVIDED FOR USE IN THE FORMULAS AND FIGURES OF PARAGRAPHS 48 THROUGH 53.

    - H = HEIGHT OF PRISM - IN FT.
    - D = WIDTH AT TOP OF PRISM - IN FT.
    - w = ANY LOADING INTENSITY (wl, w2,….) APPLIED TO TOP OF PRISM - IN PSF.
    - a = DISTANCE (a_1, a_2,….) FROM WALL PLANE TO CORRESPONDING LOADING INTENSITY W- IN FT.
    - P = SURCHARGE ON PRISM (OVERLYING EARTH, FOOTINGS AND OTHER LOADS) – IN LBS.
    - Q = SURCHARGE WITHIN DISTANCE a - IN LB.
    - e = UNIT SIDE PRESSURE DUE TO WEIGHT OF PRISM (ZERO AT TOP, MAXIMUM AT BOTTOM
      OF PRISM) IN PSF.
    - p = UNIT SIDE PRESSURE DUE TO P - IN PSF.

48. THE SIDE PRESSURE FROM THE ABUTTING EARTH PER LINEAR FOOT OF WALL IS TAKEN AS THAT DUE TO THE EARTH CONTAINED WITHIN THE TRIANGULAR PRISM ABC (FIG. 5) ONE FOOT LONG AND BORDERED BY THE WALL PLANE AB, A HORIZONTAL PLANE OF LOADING BC, AND A PLANE OF RUPTURE AC PASSING THROUGH THE BOTTOM OF THE WALL SPAN.

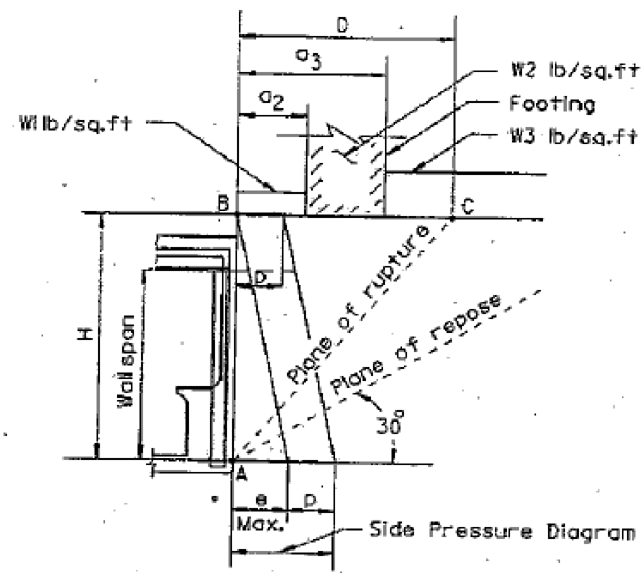


FIGURE 5

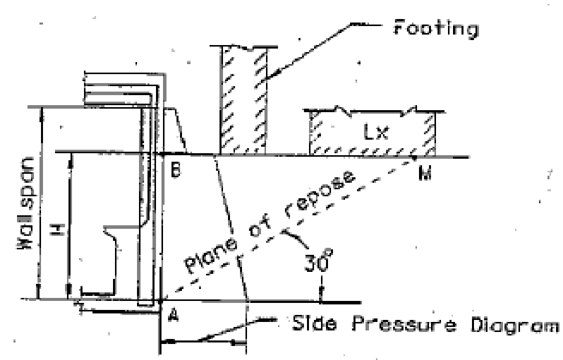


FIGURE 6

49. THE WIDTH D IS A FUNCTION OF THE PROPERTIES OF THE SOIL AND THE AMOUNT AND DISTRIBUTION OF THE SURCHARGE. FOR THE USUAL ANGLE OF REPOSE OF 30 DEGREES WITH THE HORIZONTAL:

$$D = 0.58H + \frac{wa - \sum Q}{w + 50H} \quad \text{.........................}(3)$$

IN DOUBTFUL CASES, D SHALL BE DETERMINED FOR EACH LOADING INTENSITY WITHIN THE PLANE OF REPOSE, THE CRITICAL VALUE OF D BEING THE ONE WHICH GIVES THE GREATEST LATERAL PRESSURE AS DETERMINED FROM FORMULAS (4) AND (5). WHERE FORMULA (3) GIVES A VALUE LESS THAN a, THE CORRESPONDING D SHALL BE TAKEN AS EQUAL TO a, WHERE IT GIVES A VALUE EXTENDING BEYOND THE OUTER LIMIT OF THE LOADING INTENSITY W USED, SUCH VALUE SHALL BE DISREGARDED.

$$e_{MAX} = 100D * \frac{H - 0.58D}{D + 0.58H} = 100DK \quad \text{.........................}(4)$$

$$p = \frac{P}{H} * \frac{H - 0.58D}{D + 0.58H} = \frac{\sum P}{H} * K \quad \text{.........................}(5)$$

<u>TABLE OF VALUES OF K</u>

D/H	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3
K	1.13	0.94	0.78	0.66	0.55	0.46	0.39	0.32	0.27	0.22	0.17	0.13

50. THE PLANE OF LOADING BC (FIG. 5) SHALL BE TAKEN AT THE TOP OF THE WALL WHEN THE SURCHARGE IS UNIFORM AT THIS LEVEL, OR WHERE A FOOTING IS LESS THAN 1 FOOT ABOVE OR BELOW IT, THE BOTTOM OF SUCH FOOTING BEING CONSIDERED LEVEL WITH THE TOP OF THE WALL. IN THIS CASE, THE VALUES OBTAINED FROM FORMULAS (4) AND (5) REPRESENT THE PRESSURES AGAINST THE WALL.

IN THE CASES WHERE FOOTINGS ARE HIGHER OR LOWER, THE PRESSURES AGAINST THE WALL ARE DETERMINED FROM THE TRAPEZOIDAL LOADING DIAGRAMS OBTAINED FOR THE PLANE OF LOADING USED. WHERE SUCH PLANE IS BELOW THE TOP OF THE WALL (FIG. 6) THE PRESSURE AGAINST THE UPPER PORTION OF THE WALL SHALL BE DETERMINED FROM THE LOADING CONDITIONS EXISTING BETWEEN THE WALL AND THE FOOTING ONLY.

WHERE A LOAD Lx (FIG. 6) IS INTERSECTED BY THE PLANE OF REPOSE AM, ONLY THE PORTION WITHIN THE PLANE NEED BE CONSIDERED.

51. SOME SIMPLE RULES FOR THE MORE FREQUENT CASES FOLLOW:

a. FOR A UNIFORM SURCHARGE, D SHALL BE TAKEN AS 0.58H, GIVING FOR THE LATERAL PRESSURE THE USUAL VALUE OF ONE-THIRD THE VERTICAL PRESSURE.

b. WHERE AN UPPER WALL OR COLUMN IS RESTING ON A CANTILEVERED ROOF BEAM (FIG. 7), FORMULA (3) IS SUFFICIENTLY APPROXIMATED BY:

$D = a + 0.58H$ ………………………………………………………………..(6)

c. WHERE THE FAR FACE OF AN UPPER SIDEWALL FOOTING RESTING ON SOIL IS WITHIN A DISTANCE OF 0.58H OF THE LOWER WALL (FIG. 8), D SHALL BE DETERMINED FROM FORMULA (3).

d. WHERE THE NEAR FACE OF AN UPPER SIDEWALL FOOTING IS MORE THAN 0.58H FROM THE LOWER WALL (FIG. 9), THE LATERAL PRESSURE SHALL BE COMPUTED FOR THE TWO CONDITIONS, D = a AND D = 0.58H, AND THE LARGER PRESSURE USED.

e. WHERE AN UPPER INTERIOR WALL OR COLUMN FOOTING IS RESTING ON SOIL (FIG. 10), THE LATERAL PRESSURE SHALL GENERALLY BE COMPUTED FOR D = a ONLY. HOWEVER, WHEN a EXCEEDS H, THE LATERAL PRESSURE CORRESPONDING TO D = 0.58H SHALL ALSO BE DETERMINED AND THE LARGER PRESSURE USED.

52. OVERLYING LOADS, WHEN CONSIDERED FOR SIDE PRESSURE, SHALL BE TREATED AS FOLLOWS:

a. IN COMPUTING LOADS FROM OR EARTH, STREET LIVE LOAD SHALL NOT BE INCLUDED.

b. LOAD FROM FLOORS SHALL INCLUDE LIVE LOAD. IN THE CASE OF TRACK FLOORS, THE LIVE LOAD SHALL BE TAKEN AT 450 PSF.

c. WHERE AN OVERLYING FOUNDATION IS NOT CONTINUOUS LONGITUDINALLY, THE LOAD SHALL BE CONSIDERED DISTRIBUTED OVER A LENGTH OF EARTH PRISM EQUAL TO THE LENGTH OF THE FOUNDATION PLUS TWICE THE TRANSVERSE DISTANCE FROM NET LINE OF LOWER WALL TO THE NEAR EDGE OF THE FOUNDATION AND TREATED AS ABOVE. IN NO CASE, HOWEVER, SHALL THE INCREASED LENGTH EXCEED THE LONGITUDINAL DISTANCE BETWEEN THE CENTERS OF ADJACENT FOUNDATIONS.

53. FOR SUBWAY WALLS UNDER OR ADJACENT TO BUILDINGS, THE LATERAL PRESSURE SHALL BE COMPUTED FROM THE GROUND OR STREET SURFACE, UNLESS LATERAL PRESSURE DUE TO THE BUILDINGS HAS TO BE PROVIDED FOR AND IS MORE SEVERE.

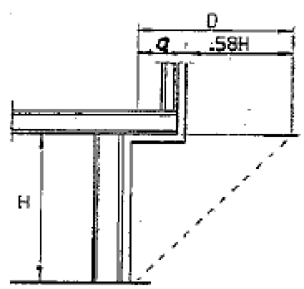


FIGURE 7

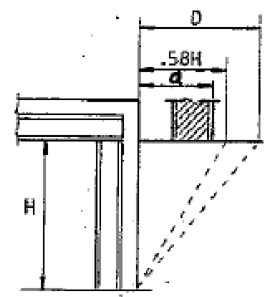


FIGURE 8

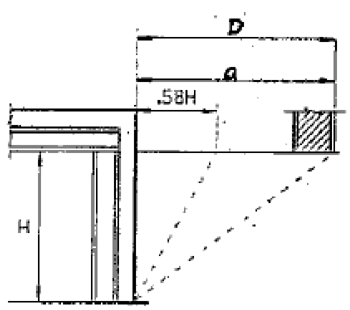


FIGURE 9

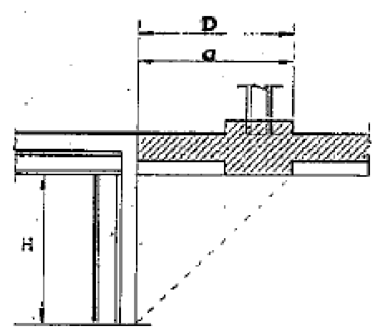


FIGURE 10

CHAPTER 3B

STRESSES

TABLE OF CONTENTS

I - INTRODUCTION

1. THE NEW YORK CITY TRANSIT (NYCT) STRUCTURAL DESIGN GUIDELINES SHALL BE USED IN THE ANALYSIS AND DESIGN FOR BOTH THE REHABILITATION OF EXISTING ELEVATED AND SUBWAY STRUCTURES AS WELL AS THE DESIGN OF NEW ELEVATED AND SUBWAY STRUCTURES.

2. THE FOLLOWING DOCUMENTS SHALL BE USED TO COMPLIMENT THE REQUIREMENTS OF THE NYCT STRUCTURAL DESIGN GUIDELINES TO THE EXTENT APPLICABLE:

   a. AMERICAN RAILWAY ENGINEERING AND MAINTENANCE OF WAY ASSOCIATION (AREMA) MANUAL FOR RAILWAY ENGINEERING.

   b. AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO) STANDARD SPECIFICATION FOR HIGHWAY BRIDGES.

   c. AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) MANUAL OF STEEL CONSTRUCTION – ALLOWABLE STRESS DESIGN.

   d. AMERICAN CONCRETE INSTITUTE (ACI) ACI 318, STANDARD BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE.

3. IN THE STRUCTURAL REHABILITATION OF EXISTING NEW YORK CITY TRANSIT ELEVATED AND SUBWAY STRUCTURES, THE FOLLOWING CRITERIA SHALL GENERALLY APPLY:

   a. ANY STRUCTURAL MEMBER THAT IS TO BE REPLACED SHALL, AS A MINIMUM, BE REPLACED IN KIND OR BY AN EQUIVALENT STRENGTH MEMBER.

   b. IF REQUIRED BY ANALYSIS, AN INCREASED STRENGTH MEMBER SHALL BE PROVIDED.

4. ALLOWABLE STRESSES TO BE USED IN THE EVALUATION OF STRUCTURAL ELEMENTS ARE GIVEN AS A FUNCTION OF YIELD STRESS, FY, IN TABLE 3B-1, “ALLOWABLE STRESSES FOR ELEVATED STRUCTURES”, AND TABLE 3B-2, “ALLOWABLE STRESSES FOR SUBWAY STRUCTURES”. WHERE EXISTING MATERIAL STRENGTH IS NOT KNOWN AND IT IS NOT CONSIDERED NECESSARY TO CONDUCT MATERIAL TESTS, THE STRENGTH CAN BE APPROXIMATED BY UTILIZING THE BRIDGE CODE OF THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS (AASHTO). THE MATERIAL YIELD STRENGTH, FY, PROVIDED IN THE AASHTO TABLES IS BASED UPON THE YEAR OF CONSTRUCTION OF THE STRUCTURE.

5. IN TABLE 3B-1, FORMULAS ARE REFERENCED TO THE 1990 EDITION OF AREA (NOW AREMA) MANUAL FOR RAILWAY ENGINEERING.

II – ELEVATED STRUCTURES

TABLE 3B-1

ALLOWABLE STRESSES
FOR
ELEVATED STRUCTURES

NATURE OF STRESS	ALLOWABLE STRESS	REMARKS
AXIAL TENSION OR FLEXURAL TENSION (NET SECTION)	$0.55F_Y$	FOR ALL FORMULAS, F_Y IS EXPRESSED IN PSI.
AXIAL COMPRESSION (GROSS SECTION)	$0.55F_Y$ WHEN $kL/r \leq 3388/\sqrt{F_Y}$	k VALUES FOR COLUMNS TO BE CALCULATED AS STIPULATED IN AISC ASD 9TH EDITION, TABLE C-C2.1
	$0.60F_Y - (F_Y/1662)^{1.5}(kL/r)$ WHEN $3388/\sqrt{F_Y} < kL/r < 27111/\sqrt{F_Y}$	
FLEXURAL COMPRESSION, BUILT-UP I MEMBERS, WEAK AXIS BENDING	$0.55F_Y$	APPLIES TO COLUMNS BENDING ABOUT WEAK AXIS
COMPRESSION IN EXTREME FIBERS OF FLEXURAL MEMBERS:		
FORMULA A: $0.55F_Y\left[1-(0.55F_Y)\left\{(L/r_T)^2/1.8x10^9\right\}\right]$		
RIVETED OR BOLTED BUILT-UP MEMBERS SYMMETRICAL ABOUT THE PRINCIPAL AXIS IN THE PLANE OF THE WEB (OTHER THAN BOX MEMBERS)	USE FORMULA A	SEE AREA SECTION 2.4.1 FOR DEFINITIONS OF ALL TERMS r_T OBTAINED FROM AISC ASD 9TH EDITION IS EQUIVALENT TO r_Y FROM AREA
ROLLED OR WELDED BUILT-UP MEMBERS SYMMETRICAL ABOUT THE PRINCIPAL AXIS IN THE PLANE OF THE WEB (OTHER THAN BOX MEMBERS) AND CHANNELS.	THE LARGER OF: FORMULA A OR $10{,}500{,}000/(Ld/A_f)$ BUT NOT TO EXCEED $0.55F_Y$	L IS DISTANCE BETWEEN POINTS OF LATERAL SUPPORT FOR THE COMPRESSION FLANGE, A_f IS AREA OF SMALLER FLANGE EXCLUDING ANY PORTION OF THE WEB, d IS OVERALL MEMBER DEPTH

TABLE 3B-1 (CONTINUED)

ALLOWABLE STRESSES
FOR
ELEVATED STRUCTURES

NATURE OF STRESS	ALLOWABLE STRESS	REMARKS
COMPRESSION IN EXTREME FIBERS OF FLEXURAL MEMBERS (CONTINUED):		
BOX MEMBERS SYMMETRICAL ABOUT THE PRINCIPAL MEMBERS MIDWAY BETWEEN WEBS WHOSE PROPORTIONS MEET AREA ARTICLES 2.6.1 AND 2.6.2. $(L/r)_e$ IS DEFINED IN AREA ARTICLE 2.4.1	$0.55F_Y\left[1-(0.55F_Y)(L/R)_e^2/1.8x10^9\right]$	
SHEAR OF WEBS, GROSS SECTION	$0.35F_Y$	
DIAGONAL TENSION IN WEBS	$F_t = 0.55F_Y$ $f_t = f/2t\sqrt{f_V^2 + (f)^2}$	TO BE CHECKED AT ENDS OF RESTRAINED GIRDERS f= FLEXURAL STRESS $f_V =$ SHEAR STRESS
BEARING, STIFFENERS	$0.85F_Y$	
BEARING, HALF-PINS	$0.75F_Y$ OF WEAKER MATERIAL	
RIVETS IN SHEAR*	GRADE 1: $F_t = 13.5KSI$ GRADE 2, 3: $F_t = 20KSI$	VALUES RECOMMENDED BY AREA
RIVETS IN TENSION*	GRADE 1: $F_t = 17KSI$ GRADE 2, 3: $F_t = 26KSI$	NO PROVISION IN AREA. VALUES BASED ON AISC ALLOWABLE STRESSES REDUCED IN PROPORTION TO CORRESPONDING AREA VALUES FOR RIVETS IN SHEAR
RIVETS IN COMBINED SHEAR AND TENSION	GRADE 1: $F_t = 22.5 - 1.3f_V < 17KSI$ GRADE 2, 3: $34.5 - 1.3f_V < 26KSI$	$f_V =$ ACTUAL SHEAR STRESS IN RIVET
HIGH STRENGTH BOLTS	IF HIGH STRENGTH BOLTS ARE USED AS SUBSTITUTES FOR MISSING OR DEFECTIVE RIVETS, ALLOWABLE STRESSES SHALL NOT EXCEED THE VALUES FOR RIVETS	
*ALLOWABLE VALUES ARE GIVEN FOR HOT DRIVEN A502 RIVETS, GRADE 1 OR GRADES 2 AND 3. GRADE 1 SHOULD BE ASSUMED UNLESS TEXT OR DATA INDICATE A STRONGER GRADE		

III – SUBWAY STRUCTURES

TABLE 3B-2

ALLOWABLE STRESSES
FOR
SUBWAY STRUCTURES

NATURE OF STRESS		***ALLOWABLE STRESS***
TENSION, NET SECTION:................................		$0.55F_Y$
COMPRESSION:		
	MEMBERS IN WHICH L/R<3	$0.55F_Y$
	COLUMNS	$17 - 0.000485L^2 / R^2$
	SECONDARY MEMBERS	$\left[18000 / \left\{1 + (L^2 / 18000R^2)\right\}\right]$
	WEBS OF ROLLED SECTIONS AT TOE FILLET .... (BEAMS NOT INCASED IN CONCRETE SHALL BE INVESTIGATED FOR CRIPPLING)	$0.66F_Y$
BENDING, EXTREME FIBER:		
	COMPRESSION OR TENSION, GROSS AREA............	$0.55F_Y$
	TENSION, NET AREA	$0.55F_Y$
	WHERE COMPRESSION FLANGE IS SUFFICIENTLY EMBEDDED IN CONCRETE, SEE SECTION 10...........	$0.65F_Y$
	COPED AND BLOCKED SECTIONS	$0.42F_Y$
	PINS AND TURNED BOLTS	$0.83F_Y$
SHEARING:		
	WEBS, GROSS SECTION	$0.35F_Y$
	COPED AND BLOCKED SECTIONS	$0.28F_Y$
	RIVETS (BY DIRECT-ACTING RIVETING)...................	15
	RIVETS (BY POWER HAMMER RIVETING)	12.5
	PINS AND TURNED BOLTS	15
	UNFINISHED BOLTS	10
BEARING:		
	CONTACT AREA OF ROLLED SURFACES..................	$0.66F_Y$
	CONTACT AREA OF MILLED SURFACES	$0.85F_Y$
	CONTACT AREA OF ROLLED AND MILLED SURFACES	$0.75F_Y$
	RIVETS (BY DIRECT-ACTING RIVETING) AND TURNED BOLTS: DOUBLE SHEAR .... SINGLE SHEAR...	 40 30
	RIVETS (BY POWER HAMMER RIVETING)	27
	UNFINISHED BOLTS: DOUBLE SHEAR .... SINGLE SHEAR...	 25 20

NOTE: IN SUBWAY STRUCTURES, THE VALUE OF L/R SHALL NOT EXCEED 120 FOR MAIN MEMBERS AND 150 FOR SECONDARY MEMBERS.

TABLE 3B-3

ALLOWABLE UNIT STRESSES
FOR
COMPRESSION MEMBERS (KSI)

L/R	$17 - 0.000485L^2/R^2$
3 OR LESS	20.00
4	16.99
6	16.98
8	16.97
10	16.95
12	16.93
14	16.91
16	16.88
18	16.84
20	16.81
22	16.77
24	16.72
26	16.67
28	16.62
30	16.56
32	16.50
34	16.44
36	16.37
38	16.30
40	16.22
42	16.14
44	16.06
46	15.97
48	15.88
50	15.79
52	15.69
54	15.59
56	15.48
58	15.37
60	15.25
62	15.14
64	15.01
66	14.99

L/R	$17 - 0.000485L^2/R^2$
68	14.76
70	14.62
72	14.49
74	14.34
76	14.20
78	14.05
80	13.90
82	13.74
84	13.58
86	13.41
88	13.24
90	13.07
92	12.90
94	12.72
96	12.53
98	12.34
100	12.15
102	11.95
104	11.75
106	11.55
108	11.34
110	11.13
112	10.92
114	10.70
116	10.47
118	10.25
120	10.02
125	
130	
135	
140	
145	
150	

TABLE 3B-4

ALLOWABLE UNIT BENDING STRESSES (ASTM A36)
IN COMPRESSION FLANGES FOR BEAMS AND GIRDERS
(KSI)

L/W	$F'_C = 25 - 0.31L/W$	L/W	$F'_C = 25 - 0.31L/W$
16	20.00	30	15.70
18	19.42	32	15.08
20	18.80	34	14.46
22	18.18	36	13.84
24	17.56	38	13.22
26	16.94	40	12.60
28	16.32	42	11.98

6. FOR TEMPORARY WORK, USE STRESSES PROVIDED IN NEW YORK CITY TRANSIT FIELD DESIGN STANDARDS.

7. HOLES FILLED WITH RIVETS AS WELL AS ISOLATED HOLES UP TO 1 INCH IN DIAMETER FOR TIE RODS AND THE LIKE, ARE NOT CONSIDERED AS REDUCING THE WEB SECTION FOR RESISTING SHEAR.

8. DETAILS SUCH AS COPINGS AND RE-ENTRANT CUTS SHALL NOT BE RESORTED TO UNLESS UNAVOIDABLE.

9. WHERE THE SUBWAY STRUCTURE TAKES BUILDING LOAD, IT SHALL BE DESIGNED IN ACCORDANCE WITH BUILDING CODE STRESSES, UNLESS THOSE OF THE NEW YORK CITY TRANSIT ARE LOWER.

10. WHERE CONCRETE MAY BE COUNTED ON TO ACT INTEGRALLY WITH THE STEEL, AN EQUIVALENT DESIGN BENDING STRESS FOR STRUCTURAL STEEL MAY BE TAKEN AS 25 KSI (FOR ASTM A36 STEEL), IN LIEU OF AN ANALYSIS OF THE COMPOSITE SECTION. EXCEPT AS NOTED IN PARAGRAPH 11, THIS APPLIES TO:

    a. BEAMS AND GIRDERS HAVING THEIR COMPRESSION FLANGE EMBEDDED IN A FLAT SLAB WHICH EXTENDS IN DEPTH AT LEAST 1/3 THE DISTANCE TO THE EXTREME TENSION FIBER, AND IN WIDTH AT LEAST ½ OF THE DEPTH OF THE MEMBER BEYOND BOTH FLANGE EDGES.

    b. BEAMS AND GIRDERS ENCASED BETWEEN SUBWAY ROOF ARCHES.

    c. LONGITUDINAL MEZZANINE AND PLATFORM ROOF MEMBERS WITH OTHER MEMBERS FRAMED INTO THEM ON BOTH SIDES AND HAVING THEIR COMPRESSION FLANGE EMBEDDED IN CONCRETE FOR A WIDTH AS SPECIFIED IN SUBPARAGRAPH a ABOVE.

WHEN SPECIAL STEELS ARE SUBSTITUTED FOR STRUCTURAL STEEL, AN ANALYSIS OF THE COMPOSITE SECTION SHALL BE MADE.

11. NO INCREASE IN BENDING STRESS BEYOND 20 KSI (FOR ASTM A36 STEEL) SHALL BE ALLOWED FOR:

a. ROOF MEMBERS LOCATED BETWEEN BUILDING LINES OF INTERSECTING STREETS WHERE THE TOP OF THE SUBWAY ROOF IS LESS THAN 7 FEET BELOW THE STREET SURFACE, SO AS TO PROVIDE FOR PRESENT OR FUTURE DEPRESSED BAY CONSTRUCTION.

b. MEMBERS PLACED CLOSER CENTER-TO-CENTER THAN THE SUM OF DEPTH AND FLANGE WIDTH.

c. MEMBERS HAVING A NOMINAL DEPTH EXCEEDING 36 INCHES.

d. RECTANGULAR SECTIONS STRESSED IN SHEAR AND BENDING, SUCH AS WEB PLATES EXTENDING LONGITUDINALLY BEYOND FLANGES OF GIRDERS.

12. WHERE THE UNBRACED LENGTH OF THE COMPRESSION FLANGE OF BEAMS AND GIRDERS STRESSED IN BENDING EXCEEDS 16 TIMES ITS WIDTH, THE ALLOWABLE COMRESSIVE STRESS SHALL BE REDUCED IN ACCORDANCE WITH THE FORMULA:

$$F'_C = 25 - 0.31L/W$$

WHERE: $F'_C =$ ALLOWABLE UNIT STRESS IN KSI (SEE TABLE 3B-4)
$L =$ UNBRACED LENGTH OF FLANGE IN INCHES
$W =$ WIDTH OF FLANGE IN INCHES.

13. FOR BEAMS AND GIRDERS, WHERE THE RATIO OF SPAN TO DEPTH EXCEEDS THE VALUE K, AS GIVEN BELOW, THE ALLOWABLE BENDING STRESS SHALL BE REDUCED IN ACCORDANCE WITH THE FOLLOWING FORMULA:

$$F' = FK(D/L)$$

WHERE: $F' =$ REDUCED BENDING STRESS
F = BENDING STRESS
K = CONSTANT, HAVING THE VALUE 12 FOR MEMBERS SUPPORTING TRACKS, AND THE VALUE 20 FOR OTHER MEMBERS
D = DEPTH OF MEMBER IN INCHES
L = LENGTH OF SPAN IN INCHES

Chapter 4

Chapter 4

DETAILS OF DESIGN FOR STRUCTURAL STEEL

TABLE OF CONTENTS

CHAPTER 4

DETAILS OF DESIGN FOR STRUCTURAL STEEL

I DETERMINATE DESIGN

1. The structure shall preferably be so designed that stresses may be calculated directly.

II MINIMUM THICKNESS OF METAL

2. In general, no material of less than 3/8 in. thickness shall be used in main members, except as fillers. This limitation does not apply to rolled beams and channels, or to canopies of elevated stations, for which the minimum thickness of material shall be 1/4 in.

III OPEN DETAILS

3. Details shall be designed so as to give free access for inspection and painting. Water pockets shall be avoided.

IV STRENGTH OF DETAILS

4. The connections and other details of a member shall be of at least sufficient strength to resist the forces acting thereon.

V SPANS FOR CALCULATION

5. The spans assumed for calculation shall be as follows:

Subway Structures:

For beams and girders with framed end connections, clear distance between supports.

For beams and girders with bearing ends, clear distance between supports plus 3 in.

For beams on bed plates, distance between centers of plates.

Elevated Structures:

For cross girders, distance between centers of columns.

For stringers, distance between centers of cross girders.

For floor beams, distance between centers of stringers.

VI DEPTH RATIOS FOR BEAMS AND GIRDERS

6. The depth of beams and girders shall generally be not less than 1/12 of the span for members supporting tracks, and not less than 1/20 of the span for other members. See “Loads and Stresses”, Sect. 49.

VII PLATE GIRDERS

7. Make-up of Flange Cover plates shall not be used unless angles of more than 7/8 in. thickness would be required. When cover plates are used, the thickness of flange angles shall preferably not exceed 5/8 in., except when the gross section of the cover plates would exceed one-half the gross section of the flange. The gross section of the compression flange shall be not less than that of the tension flange.

8. SECTION MODULUS OF GIRDERS WITH OR WITHOUT COVER PLATES

Notation:
- A = Area of two angles, with one or two rivet holes per angle deducted (See Sect. 21) - in sq. in.
- a = Area of top or bottom cover plate, with two rivet holes per plate deducted - in sq. in.
- h = Depth of web - Nominal depth of girder - in inches.
- d = Distance between centers of gravity of angles - in inches.

d_1 = Distance from back to back of angles - nominal depth of girder plus 1/2 in. - in inches.
S_a = Section Modulus of four angles - in $in.^3$
S_c = Section Modulus of cover plates (top and bottom) - in $in.^3$
S_w = Equivalent Section Modulus of web - in $in.^3$
t = Thickness of web in inches.

Values:

$$S_a = \frac{Ad^2}{d_1} \qquad (1)$$

$$S_c = ad_1 \qquad (2)$$

$$S_w = \frac{th}{8}(h-3) \qquad \text{(See Sect. 10)} \qquad (3)$$

9. Splicing of Flange Flange members, when spliced, shall be covered by extra material at least equal in section to the member spliced. There shall be enough rivets on each side of the splice to develop the strength of the member.

Flange angles shall generally be spliced with angles.

Splices shall preferably be located at points where there is an excess of material. Where practicable, no two component parts of a girder shall be spliced at the same cross section.

10. Web for Flange Area In computing the bending strength of plate girders, one-eighth of the gross web section shall be counted as additional flange area, as per Sect. When the web is in one piece or is spliced for bending. No such allowance shall be made when the web is spliced for shear only.

11. Splicing of Web Web splices shall consist of one or more plates riveted on each side of the girder. The strength of plates and rivets to be sufficient to resist the shear and generally also the moment acting at the splice.

12. Length of Cover Plates Cover plates shall be extended beyond their theoretical ends to permit placing of additional rivets in accordance with Sect. 16.

Where cover plates are used, at least one upper cover plate shall extend from end to end of girder except when encased in concrete or protected against moisture.

In open track floors of elevated structures where cover plates are required on track stringers, the upper cover plates shall all extend the full length of the stringer, and the uppermost cover plate shall be set the required distance below the Base of Rail. The thickness of this cover plate shall be not less than 5/8 in., and the rivets passing through it shall be countersunk at the top.

13. <u>Thickness of Cover Plates</u> Cover plates shall be of equal thickness or shall diminish in thickness from the flange angles outward, but no one plate shall have a thickness greater than that of the flange angles, except as noted in Sect. 12.

14. <u>The Cross Section of the Cover Plates</u> in each flange shall preferably form not more than one-half, and in no case more than two-thirds, of the gross section of the flange.

15. <u>Flange Rivets through Web</u> The flanges shall be connected to the web with enough rivets to transfer the stress due to longitudinal shear as well as that due to loads supported by the flanges. When ties rest directly on the upper flange of elevated railway stringers, each wheel load, properly increased for impact, shall be considered distributed over a track length of 5 ft.

Load transmitted through arch action is not considered as producing vertical stress in the flange rivets of plate girders.

16. <u>Spacing of Flange Rivets</u> (See also Sect. 27.)

<u>Notation:</u>
- n = Number of rivets required between theoretical and actual end of cover plate (not to be less than four).
- A = Net area of one entire flange at section considered, not including web - in sq. in.
- A_1 = Net area of one flange (generally including 1/8 web) inside of the cover plate for which <u>n</u> is desired - in sq. in.
- a = Net area of all the cover plates of one flange at section considered - in sq. in.

α = Net area of the cover plate for which n is desired, - in sq. in.
Δ = 1/8 of web area if counted on to resist bending, - in sq. in.
d = Distance between centers of gravity of angles, - in inches.
f = Allowable unit bending stress, - in kip per sq. in.
r = Bearing or shearing value of one rivet (single shear in case of cover plates), - in kip.
V = Total transverse shear at section considered, - in kip.
w = Load applied directly to flange, - in kip per lin. ft.
p = Distance between rivets, assuming single gage lines in legs of angles, - in inches.

Values:

Rivets Connecting Flange Angles to Web

For longitudinal shear only:

$$p = \frac{rd}{V}\left(\frac{A+\Delta}{A}\right) \qquad (4)$$

For longitudinal shear and load w applied directly to flange (See Sect. 15):

$$p = \frac{r}{\sqrt{\left(\frac{V}{d}\cdot\frac{A}{A+\Delta}\right)^2+\left(\frac{w}{12}\right)^2}} \qquad (5)$$

Rivets Connecting Cover Plates to Flange Angles

Within theoretical length of cover plates

$$p = \frac{rd}{V}\left(\frac{A+\Delta}{a}\right) \qquad (6)$$

Between theoretical and actual length of cover plates:

$$n = \frac{fA_1\alpha}{r(A_1+\alpha)} \qquad (7)$$

17. Stiffeners – Rules for stiffeners on Plate Girders are given in sections 42 to 47.

VIII BUILT-UP SECTIONS OTHER THAN PLATE GIRDERS

18. The Section Modulus of beams with cover plates and of other built-up sections, not including plate girders, shall be computed from the Moments of Inertia of the component parts. Cover plates for beams shall be designed in accordance with Sects.VI and VII.

IX LATTICING AND STAY PLATES

19. Lattice-bars of compression members shall be so spaced that the L/r of the portion of the flange included between the lattice-bar connections will not be greater than two-thirds of the L/r of the member, and in no case more than 40.

Single lattice-bars shall generally be inclined at an angle of sixty degrees to the axis of the member, and double lattice-bars at an angle of forty-five degrees, with a rivet at their intersection.

The latticing of compression members shall be proportioned to resist a shearing force normal to the member equal to 2 percent of the direct load.

Single lattice-bars shall have a thickness of not less than one-fortieth, and double lattice-bars a thickness of not less than one-sixtieth, of the distance between the rivets connecting them to the compression member, and their minimum width and end connections shall be as follows:

For 8" C's	rolled or built-up, use 2" bars with 1-5/8" rivet at each end
For 9" C's	rolled or built-up, use 2-1/2" bars with 1-3/4" rivet at each end
For 10" & 12" C's	rolled or built-up, use 3" bars with 1-3/4" rivet at each end
For 15" & 18" C's	rolled or built-up, use 3-1/2" bars with 1-7/8" rivet at each end

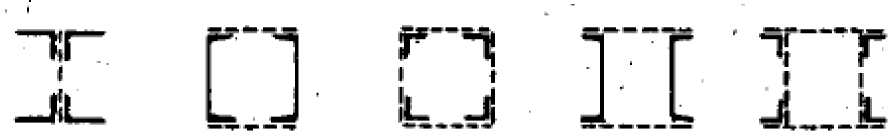


Figure 1

SD-4-6

For types of latticed sections, see Fig. 1.

20. The latticed sides of main compression members shall be provided with stay-plates at the ends and at any intermediate point where the latticing is interrupted.

The length of end stay-plates shall be not less than 1-1/4 times the distance between the lines of rivets connecting them to the outer flanges, and the length of intermediate stay-plates shall be not less than 3/4 times that distance.

In tension members composed of shapes, the latter shall be connected by stay-plates, or by stay-plates and lattice-bars.

Stay-plates are generally not required in secondary members.

X NET SECTIONS

21. Tension members shall be designed for their net section, except as noted in Sect. 22, rivet holes being taken as 1/8 in. larger than the nominal diameter of the rivet.

Where rivets are staggered, the net section of tension members shall be the smallest area which can be obtained in a straight or zig-zag line across the member, the net area of diagonal sections between rivet holes being counted at 3/4 their value.

In no case shall a deduction be made for more than one hole in each leg of plate girder flange angles, tests having shown that plate girders usually fail by buckling of the compression flange.

XI EFFECTIVE SECTIONS OF ANGLES IN TENSION

22. In tension members, the effective section of an angle when connected at its end by one leg only, shall be assumed as the net section of the connected leg plus one-half that of the unconnected leg.

When the angle is connected by both legs, the effective section shall be assumed as the next section of the angle.

XII ECCENTRIC CONNECTIONS

23. Where eccentric connections are used, provision shall be made for the additional stresses produced in the member and its details.

Exception is made for standard stiffener and connection angles on beams and girders, where eccentricity shall not be considered.

XIII ABUTTING COMPRESSION MEMBERS

24. Abutting members subject to compression only, when faced for bearing and supported against lateral deflection within 18 in. of abutting ends, shall be connected with four or more rivets to hold the abutting parts securely in place. When faced for bearing and not thus supported against lateral deflection, such members shall be spliced on four sides for 50 percent of full strength. Abutting members not faced for bearing shall be spliced for full strength.

XIV DIMENSIONS AND PLACING OF RIVETS

25. <u>Rivet Diameter</u> In proportioning rivets, the nominal diameter of the rivet shall be used. Rivets shall generally be 7/8 in. in diameter.

26. <u>Rivet Grip</u> If the grip of rivets carrying calculated stress exceeds 5 times the diameter of the rivet, the number of rivets shall be increased at least one percent for each additional 1/16 in. of grip. Where the grip would exceed 6 times the diameter, rivets of larger size shall be used.

27. <u>Rivet Spacing</u> The distance center to center of rivets shall be not less than three diameters when holes are sub-punched and reamed or drilled from the solid, and not less than three diameters plus 1/4 in. when holes are punched full size, except that when 7/8 in. rivets are used with 6 in. stiffener or connection angles, the gage may be made 2-1/4 in. from back of angle and 2-1/2 in. between rivet lines and the rivets placed 2-1/2 in. center to center both between and in the flange angles. (See General Contract Provisions.)

In the direction of stress, rivets shall be spaced not farther apart than 16 times the thickness of the thinnest plate connected, nor more than 30 times that thickness at right angles to the direction of stress. However, in no case shall the spacing of rivets in the direction of stress exceed 6 in.

When two or more plates are in contact and riveted for compactness, the stitch rivets shall be not farther apart in any direction than 24 times the thickness of the thinnest plate connected.

At the ends of compression members, the spacing of rivets in the direction of stress shall not exceed 4 times the diameter of rivet for a distance equal to the width of the member.

28. Edge Distance of Rivets The minimum distance from center of rivet to a rolled or finished edge shall be 1-1/2 times the diameter of the rivet, and to a sheared edge 1/8 in. more, except that in flanges of beams and channels the gages given in the steel handbooks shall be used.

The distance from center of rivet to the edge of a plate shall not exceed 8 times the thickness of the plate.

29. Rivets through Fillers shall be considered only 75 percent effective in shear, unless each filler is extended for direct riveting by at least one-third the number of such rivets. (See Sect. 56.)

30. Use of Field Rivets The number of field rivets shall be reduced to a minimum.

31. Use of Unfinished Bolts Unfinished bolts may be used for connecting base plates to grillages where difficulty in riveting is encountered, provided the bolts will not be subject to calculated stresses.

32. Bearing Area of Pins, Bolts and Rivets The effective bearing area of pins, turned bolts and rivets shall be taken as their diameter multiplied by the length in bearing. For countersunk rivets, the length in bearing shall be considered red reduced by one-fourth the rivet diameter if countersunk on one side, and by one-half the rivet diameter if countersunk on both sides.

XV WELDING

33. (a) Shop welding is permitted except for:

(1) Members or connections for elevated structures.

(2) Members or connections subjected to impact or fatigue loading, e.g., subway track floor.

(b) Field welding of buildings is permitted, e.g., bus garages and above ground substations. (Design to be based on the structural specifics and economics of each situation.)

(c) Field welding of subway or below ground structures is not permitted unless approved by the Civil Engineering and Architectural Division, and is prohibited where shop welding is not permitted.

34. The welding portions of the following Specifications are to be used:

(a) <u>Subway and Below Ground Structures</u>

American Railway Engineering Association (AREA) Specifications, 1970, for subway and below ground structures.

(b) <u>Buildings</u>

American Institute of Steel Construction Specifications (AISC), 1963, for buildings.

XVI BEARING AT SUPPORTS

35. Only such part of bearing area shall be considered effective as is located within lines drawn at forty-five degrees through the edges of the supports (see pp. SD-4-11 and SD-4-12). At sidewall columns supporting beams, the inner 6 in. of the column depth shall be considered in contact with the beam; at interior columns, the full depth under the beam shall be thus considered.

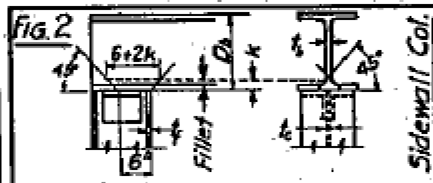
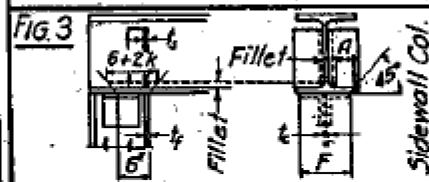
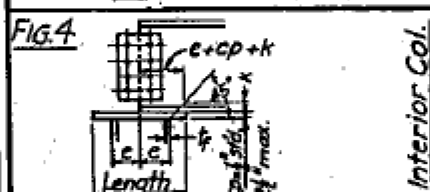
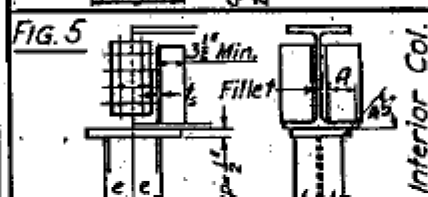
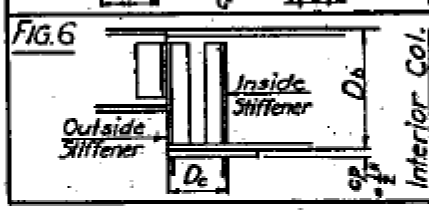
36. At sidewall columns supporting beams, if the bearing area as determined from Sect. 35 is deficient, stiffeners shall be provided on the beam. Where sidewall columns support plate girders, cap plates 3/4 in. in thickness shall be used.

37. At interior columns, where a welded cap plate is used for connecting column to roof members, the plate shall have a thickness of 3/4 in. when supporting a plate girder and at least 1/2 in. when supporting a beam. In the latter case, the thickness may be varied up to 2-1/2 in. as required for bearing. If sufficient bearing area cannot be thus obtained, stiffeners shall be used on the beam, with cap plate 1/2 in. thick.

Where the reaction from one span is more than twice that from the other span, the beam or plate girder producing the heavier reaction shall extend over the full depth of the column section.

DISTRIBUTION OF LOADING AT SUBWAY ROOF SUPPORTS AND FORMULAS FOR BEARING AND SHEAR VALUES.

Dimensions in in. Bearing and Shear Values in kip.

FIG. 2 	Sidewall Col.	BEAM	Bearing = $24t_b(6+2k)$ Shear = $13.0t_bD_b$
		COL.	Bearing = $27[(6-t_f)t_c+(2+0.0145W_b)t_f]$ W_b = Weight of beam-in lb. per lin. ft.
FIG. 3 	Sidewall Col.	BEAM	Bearing = $24t_b(6+2k)$ + value of stiffs.(but not to exceed beam shear). Shear = $13.0t_bD_b$
		COL.	Bearing = $27[Ft_f+(6-t_f)t_c]$
		2-STIFF.	Bearing = $2\times27At_s$ or value of rivets, whichever is less.
FIG. 4 	Interior Col.	BEAM	Bearing = $24t_b(e+cp+k)$ Shear = $13.0t_bD_b$
		COL.	Bearing = $27[(e-t_f)t_c+(2cp+2+0.0145W_b)t_f]$
FIG. 5 	Interior Col.	BEAM	Bearing = $24t_b(e+\frac{1}{2}+k)$ + value of stiffs.(but not to exceed beam shear). Shear = $13.0t_bD_b$
		COL.	Bearing = $3.97W_c = 27(\frac{1}{2}$ Area of column) W_c = Weight of column-in lb. per lin. ft.
		2-STIFF.	Bearing = $2\times27At_s$ or value of rivets, whichever is less.
FIG. 6 	Interior Col.	BEAM	Bearing = $24t_b(D_c+\frac{1}{2}+k)+1\frac{1}{2}\times$ Value of Inside Stiffs. Shear = $13.0t_bD_b$
		COL.	Bearing = $4.85W_c$ (for 16.5 x area of column) but not to exceed column formula value.
		2-STIFF.	Bearing = $2\times27At_s$ or value of rivets, whichever is less.

ROOF GIRDER ON SIDEWALL COLUMN.

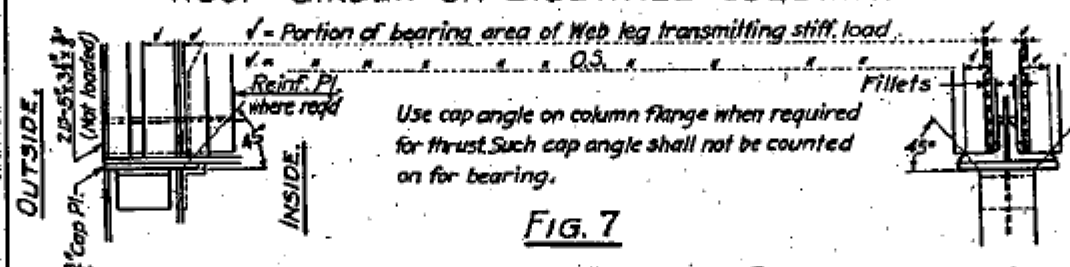


FIG. 7

GIRDER EXTENDING OVER INTERIOR COLUMN.

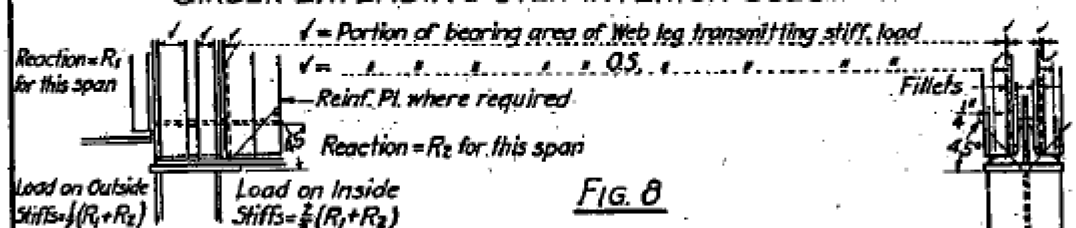


FIG. 8

DISTRIBUTION OF LOADING AT INTERMEDIATE FLOOR AND INVERT SUPPORTS, AND FORMULAS FOR BEARING VALUES OF BEAMS AT TRANSVERSE BEAM SUPPORTS.

Dimensions in in. Loads and Bearing Values in kip.

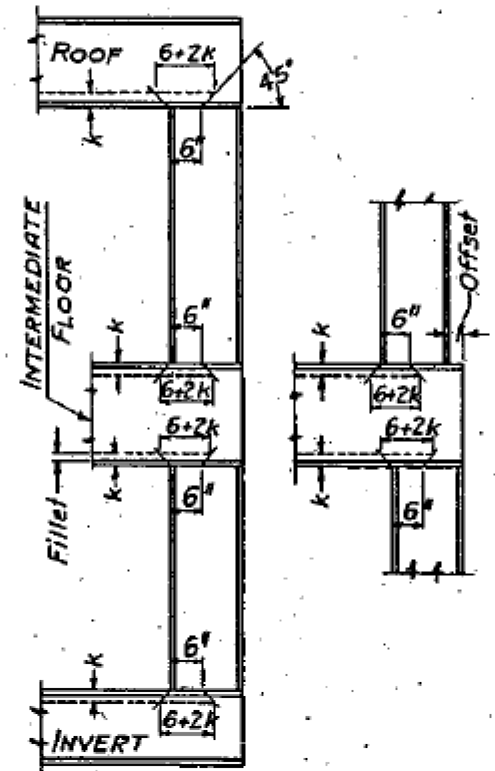


FIG. 9

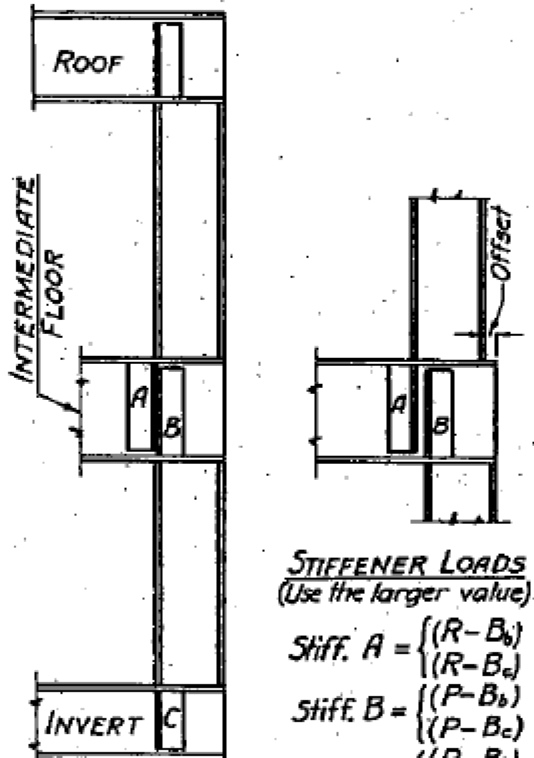


FIG. 10

STIFFENER LOADS
(Use the larger value)

$$\text{Stiff. } A = \begin{cases} (R - B_b) \\ (R - B_c) \end{cases}$$

$$\text{Stiff. } B = \begin{cases} (P - B_b) \\ (P - B_c) \end{cases}$$

$$\text{Stiff. } C = \begin{cases} (P - B_b) \\ (P - B_c) \end{cases}$$

NOTES.

R = Load on upper column = Reaction of roof beam, considering load to back of column.
P = Load on lower column = R + Reaction from intermediate floor.
B_b = Bearing value of beam.
B_c = Bearing value of column (No stiffeners on beam).
Formulas for B_b and B_c are given on page DD-8.

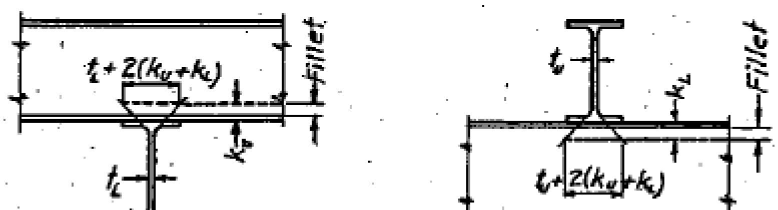


FIG. 11

Bearing Value of upper beam $= 20\,t_u\,[t_L + 2(k_u + k_L)]$
Bearing Value of lower beam $= 20\,t_L\,[t_u + 2(k_u + k_L)]$

38. In considering bearing of columns on invert beams or plate girders, rules similar to those for columns supporting beams or plate girders shall apply.

39. For intermediate floor beams, the distribution of load to be assumed at sidewall columns is shown on page SD-4-12.

XVII STIFFENERS ON BEAMS

40. Stiffeners on beams shall be designed for the difference between the load transmitted (see "Loads and Stresses," Sect. 14) and the bearing value of the beam web or column, whichever is less.

Where a beam is extended over the full depth of an interior column to support a beam from an adjacent span (Fig. 6, pg. SD-4-11), the inside and outside stiffeners, if such are required on the supporting beam, shall be proportioned, respectively, for two-thirds and one-half of the column load after deducting bearing value of beam web or column, whichever is less.

41. Stiffeners on beams shall be finished at bearing end. The non-bearing end may be cut back 1 in. from inside of flange to allow space for drift pin. On account of minor variations in the distance between flanges, it is impracticable to fabricate stiffeners on beams so that they will bear at both ends. To meet this condition, two stiffeners may be used as shown in Fig. 10, page SD-4-12.

The outstanding leg of stiffener angles, when exposed, shall extend as nearly as practicable to the edge of the flange. When encased in concrete, it shall be at least 1/2 in. inside of the flange edge. The web leg shall generally be not less than 3-1/2 in.

The thickness of stiffener angles shall be in multiples of 1/8 in., with a minimum of 3/8 in.

Rivets in web leg of stiffener angles shall be computed for vertical shear only (See Sec. 23). For rivet spacing in 6 in. web leg, see Sect. 27.

Where interior columns are less than 8 in. deep or where splice plates are used on roof beams over a column, the web legs of stiffener angles shall point toward the center of span.

XVIII STIFFENERS ON PLATE GIRDERS

42. Stiffeners shall be provided at points of concentrated loading, including reaction points, at web splices and at free end or ends of girders.

When the web thickness of a girder is less than 1/50 of the unsupported distance between flange angles, intermediate stiffeners, usually 5”x3-1/2”x3/8” angles, shall be riveted on both sides of the web. The distance center to center of such stiffeners shall generally not exceed the full depth of the web but shall in no case be less than 4 ft. nor more than 6 ft.

43. At sidewall columns the inside stiffeners on the girder shall be designed for the full girder reaction. End stiffeners shall be 5”x3-1/2”x3/8” angles.

44. At interior columns, where each girder is extended over one-half the column depth, stiffeners shall be provided on each girder to transmit its full end reaction.

Where a girder is extended over the full depth of an interior column to support a beam or girder from the adjacent span, the inside and outside stiffeners shall be proportioned; respectively, for two-thirds and one-half of the total column load.

45. At intermediate floor girders, the stiffeners over the lower columns shall be designed in accordance with Sects. 43 and 44.

Where an upper column rests on the top flange of the girder and is vertically aligned with the lower column, the stiffeners shall be designed for the combined column and intermediate floor load in bearing, web rivets, however, being provided only for the intermediate floor load.

The load of an upper sidewall column resting on a girder is assumed to be taken by the inside stiffeners (see page SD-4-12), that of an upper interior column is assumed to be equally divided between the supporting stiffeners.

46. Bearing stiffeners on girders shall be finished at bearing ends and connected to the web by enough rivets to transmit the calculated load and shall not be crimped.

The outstanding leg of stiffener angles, when exposed, shall extend as nearly as practicable to the edge of the flange. When encased in concrete, it shall be at least 1/2 inch inside of the flange edge. The web leg shall generally be not less than 3-1/2 inches.

The thickness of stiffener angles shall be in multiples of 1/8 inch, with a minimum of 3/8 in., and shall generally not exceed 5/8 in. Where two stiffeners of 5/8 in. thickness are insufficient, four angles may be used or, if this is impracticable, two angles with plates riveted to the outstanding legs. The thickness of stiffeners may be greater than 5/8 in. when the thickness of flange angles or web of a girder is such as to require sub-punching and reaming.

47. Stiffener rivets through fillers shall be considered only 75 percent effective in shear, except where fillers are extended for direct riveting (see Sec. 29), or where stiffeners are placed on top of web reinforcing plates, in which cases the full shearing value may be allowed. When the web is less than 9/16 in. thick, the bearing value of the rivets may govern, 9/16 in. being the bearing thickness which is equivalent to 75 percent of double rivet shear.

Stiffener rivets through flange angles shall not be counted on as transmitting load to web, except where stiffeners are placed on top of web reinforcing plates, in which case the full shearing value of the rivets may be allowed. Short web reinforcing plates may be added under stiffeners where space available for stiffener rivets would otherwise be insufficient. (See Fig. 8, pg. SD-4-11).

XIX SUBWAY COLUMN DETAILS

48. For built-up columns, the distance back to back of angles shall be 1/4 in. greater than the nominal depth (width of web plate), except for 6 in. columns when it shall be 1/2 in. greater.

49. In double deck structures, where an upper and lower interior column are vertically aligned, the lower column shall preferably be run through to the roof, or else be spliced to the upper column at the intermediate floor framing as shown in Fig. 12 (a, b, & c).

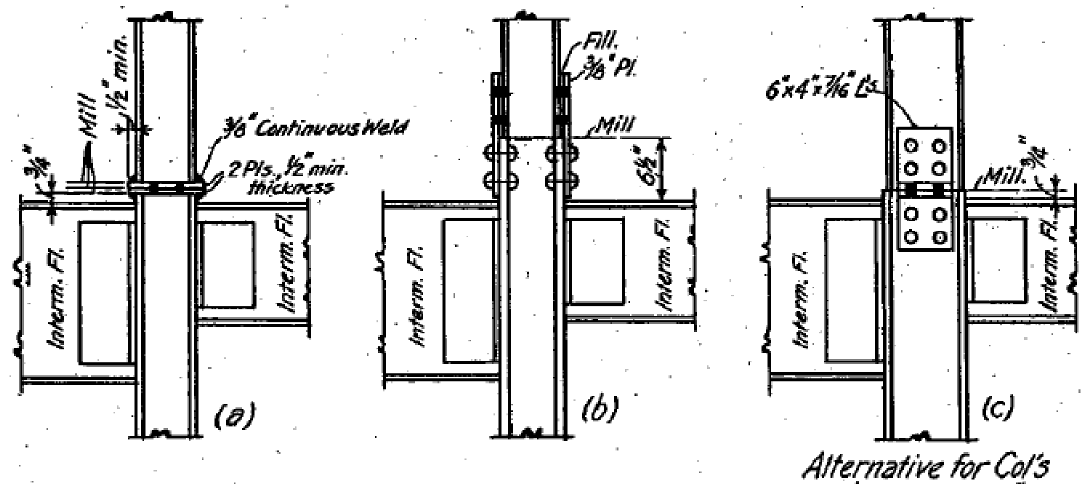


Fig. 12

In the case of sidewall columns vertically aligned, where it is impracticable to transmit the load of the upper column through the intermediate floor beam, the columns may be spliced above the intermediate floor, the beam being connected to the lower column as shown in Fig. 13.

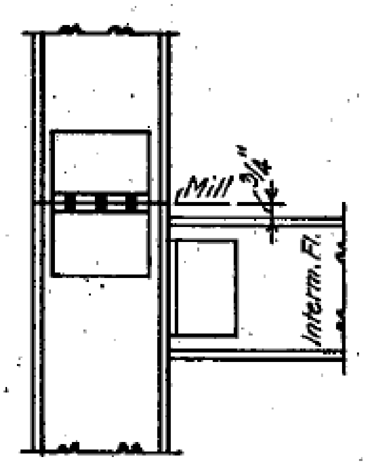


Figure 13

50. Where the track grade is 1/2 percent or less, longitudinal members shall be made level and stepped up at columns, except at mezzanine floors where the members shall follow the track grade.

Where the track grade exceeds 1/2 percent, longitudinal members shall always follow the track grade. Beveled cap plates may be used where it would otherwise be necessary to bevel the top of columns, or the columns may be run through and the longitudinal members shear-connected to them.

51. Cap and base angles, if used on columns shall generally be 7/16 in. thick.

Cap angles shall generally not be used on the web of an interior column 8 in. or less in depth where the superimposed beams are spliced at the center of the column, nor on the flange of such column under a beam supported by a member extending across the column from an adjacent span.

Cap angles on web of sidewall columns shall connect roof member by at least four rivets for 8 in. and 10 in. columns, and by at least six rivets for 12 in. columns and over. Such angles shall not be counted on for bearing.

Cap angles on inside flange of sidewall columns shall be used only where required to provide additional rivets for resisting side pressure. Where a sufficient number of 7/8 in. rivets cannot be provided in the angles connecting a column web to a beam, 1 in. rivets shall be tried before resorting to a cap angle on the column flange.

The length of cap angles on column flanges shall, if possible, not exceed the width of beam or column flange, whichever is less. Base angles on column flanges may extend beyond the flange if embedded in concrete.

52. Bearing plates shall not be riveted to the flanges of interior columns supporting beams or girders for the purpose of increasing the bearing area at the support, such plates being either uneconomical or ineffective.

XX MISCELLANEOUS

53. <u>Universal Mill Plates</u> shall be used for cover plates and for cap and base plates where feasible, to eliminate shop planing of edges and to insure full bearing where required.

54. <u>Copings and Re-entrant Cuts</u> shall not be resorted to unless unavoidable (See “Loads & Stresses,” Sect. 45). In any case they shall be rounded.

55. <u>Web Reinforcing Plates on Beams</u> are not permitted.

56. In Plate Girders, the total number of rivets through fillers or web reinforcing plates shall be at least equal to the reaction divided by the web bearing value of one rivet (Sections 29 & 47).

57. Specifications for Drilling, Subpunching & Reaming If the thickness of the material is not greater than the nominal diameter of the rivet or bolts plus 1/8 in., the holes may be punched. If the thickness of the material is greater than the nominal diameter of the rivet or bolt plus 1/8 in., the holes shall either be drilled from the solid or subpunched and reamed.

58. Specification for Remodeling Existing Steel When necessary to use flame cutting, it shall be done by approved method as directed by the Engineer. When permitted, the edge of the cut member shall be straight and the distance from the center of rivets to the cut edge shall not be less than 1-1/2 times the diameter of the rivet plus 3/8 inch.

Chapter 5

Chapter 5

DESIGN OF SIDEWALL COLUMNS SUBJECTED TO BENDING FROM EXTERNAL SIDE PRESSURE

TABLE OF CONTENTS

CHAPTER 5

DESIGN OF SIDEWALL COLUMNS SUBJECTED TO BENDING FROM EXTERNAL SIDE PRESSURE

I - NOTATION:

A = Area of column section in sq. in.
L = Unbraced length of column in inches, generally 168 for 13'-2" headroom
M = Bending moment in in. kip
P = Total direct load in kip
r = Radius of gyration of column in inches
S = Section modulus of column in in.^{3}

f'_D = Maximum allowable direct stress in kip per sq. in.

f'_B = Maximum allowable bending stress in kip per sq. in. (For steel embedded in concrete and carrying no direct load.)
f_{B1} = Allowable bending stress in kip per sq. in. for P = 150 kip
f_{B2} = Allowable bending stress in kip per sq. in. for $P = f'_D A$
f_B = Allowable bending stress in kip per sq. in. for any other load

Water Level = Ground water level, or T.A. datum (El. 100.0), whichever is higher.

<u>RULES FOR DESIGN:</u> (See also "Loads & Stresses," Sects. 39 to 41)

2. Under ordinary conditions, only dead load is to be considered in determining side pressure, which shall be computed as follows:

a) In earth above water level, as one-third the vertical pressure, assuming weight of earth as 100 lb. per cu. ft.

b) In earth below water level, same as above, and, in addition, full water pressure from water level.

c) In sound rock, where concrete can be rammed into it and produce a seal for at least one-half the area exposed, as one-half water pressure from water level, top of rock or top of subway roof, whichever is highest. In unsound rock, where no such seal can be obtained, full water pressure shall be assumed instead.

d) With rock line within the span of a sidewall column, as full water pressure from water level or top of subway roof, whichever is higher, and in addition earth pressure on portion above rock, except that where rock line is less than 3 ft. below top of column, the water pressure may be reduced to one-half below rock surface if in sound rock.

The above rules are for general guidance only, as drainage conditions depend on position of subway relative to rock surface over a considerable area; also, field conditions may call for subsequent changes in design.

3. f'_D and f_B may be determined from the following formulas, which apply to any column length:

<u>For T.A. Stresses:</u>

$$f'_D = 17-0.000485\,\frac{L^2}{r^2} \qquad (1)$$

$$\text{For } P \leq 150: \quad f_B = 25 - \frac{P}{2A} \qquad (2)$$

$$\text{For } P > 150: \quad f_B = (20 - f'_D) + \left(5 + f'_D - \frac{150}{2A}\right)\frac{f'_D A - P}{f'_D A - 150} \qquad (3)$$

<u>For Bldg. Code Stresses:</u>

$$f'_D = 17-0.000485\,\frac{L^2}{r^2} \qquad (4)$$

$$\text{For } P \leq 150: \quad f_B = 22 - \frac{P}{2A} \qquad (5)$$

$$\text{For } P > 150: \quad f_B = (20 - f'_D) + \left(2 + f'_D - \frac{150}{2A}\right)\frac{f'_D A - P}{f'_D A - 150} \qquad (6)$$

4. In general, sidewall columns shall extend 6 in. below the bottom of track trough or rough floor line, unless riveted to a transverse grillage or invert beam; at side platforms, mezzanines and station entrances, this distance shall be 4 in. min.

II - INTERIOR COLUMNS

5. Formula to be used for the design of interior columns:

$$U = 17 - 0.000485 \frac{L^2}{r^2}$$

where: U = Allowable Unit Stress in kips per square in.
L = Unbraced length of column in in.
r = Governing Radius of Gyration in in.

Formulas for combined stresses due to direct load and bending are given in "Loads & Stresses," Sect. 53.

III-GRILLAGES, BASE SLABS AND FOOTINGS

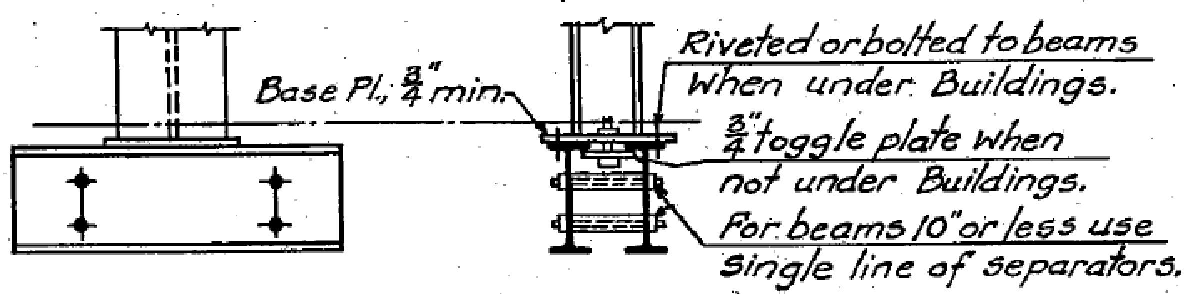
6. Grillage beams shall have sufficient flange bearing area, considering both flanges, to transmit safely the superimposed load to the concrete at the allowable unit bearing stress. The total width of bearing on concrete shall be taken as A+2B, where "A" is the distance center to center of outer grillage beams, not to exceed the number of beams less one multiplied by twice the flange width of one beam, and "B" is the distance from center of outer beam to adjacent face of concrete footing, not to exceed the flange width of one beam. The clear distance between flanges of grillage beams shall be not less than 2-1/2 inches.

7. For grillages made up of standard beam sections, the load shall be considered uniformly distributed on the concrete at a bearing value not exceeding 750 lb. per sq. in. (500 lb. for Bldg. Code.)

8. For grillages made up of wide flange beam sections and for column base slabs, the load shall be considered uniformly distributed on the concrete at a bearing value not exceeding 500 lb. per sq. in. (Same for Bldg. Code.)

The lower bearing value for the concrete under wide flanges and slabs in T.A. design is due to their greater flexibility causing a substantial variation in pressure, from a maximum at the center of beam flange or slab to a minimum at the edges.

The legs of wide flange beam sections, considered as cantilevers from end of fillet, shall not be stressed in bending beyond 20 kip per sq. in. (18 kip for Bldg. Code.)



TYPICAL GRILLAGE BEAMS.

FIG. 1

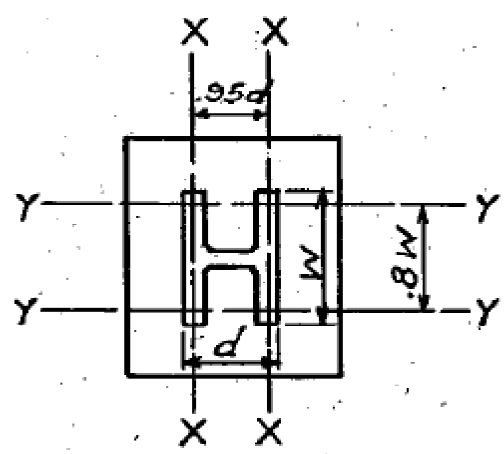
9. Grillages shall be designed to resist moment, shear and full bearing under the column. Stiffeners on grillage beams, where required, shall be designed for the difference between the reaction and the bearing value of the beam webs or column, whichever is less.

The allowable bearing value for steel shall be taken at 24 kip per sq. in. (18 kip for Bldg. Code).

Web buckling shall not be considered when the beams are encased in concrete and the load is applied after the concrete has set.

10. Base slabs shall be designed for bending about axes X-X and Y-Y, Fig. 2, whichever governs, considering the total load spread uniformly over the entire slab area.

11. For grillages consisting of two or more beams, the column load shall be considered equally distributed among the several beams.



TYPICAL BASE SLAB.

FIG. 2

12. Grillage beams shall have a depth of not less than 6 in. and a length of not less than 1'-6".

13. Beams or base slabs, supporting columns spaced about 5 ft. on centers, shall be considered as distributing the load to the soil longitudinally for a distance equal to the column spacing, owing to the arching effect of the concrete. For grillages spaced further apart, such as those under platform columns, the load shall be considered uniformly spread to the soil longitudinally on 30^o planes passing through the upper flange ends as indicated in Fig. 3, the arching effect of the concrete being in this case disregarded. The transverse reinforcement, if any, shall be designed for the load carried over the full length L and longitudinal bars 12 in. center to center shall be provided, the same size bars being used for both sets of reinforcement, with an upper limit of No. 6 for the longitudinal bars. See also "Design of Concrete and Reinforced Concrete Structures."

The grillage shall be designed for the total load uniformly distributed over the length of beam. Double tier grillages may be used if economical.

14. The weight of the concrete footing shall be disregarded in the design and in determining bearing on soil.

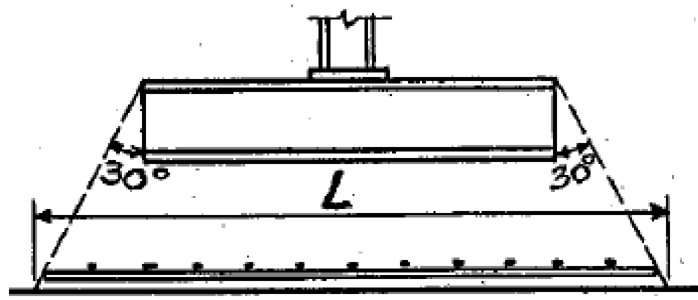


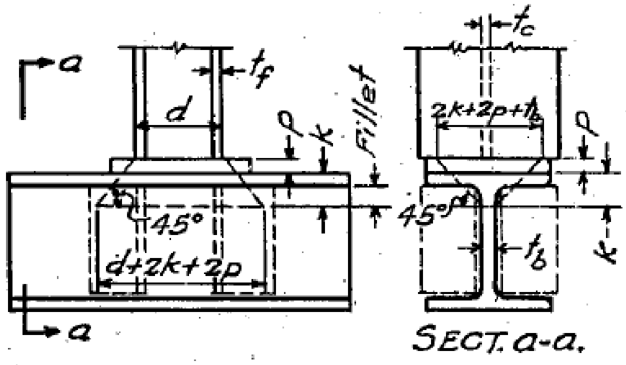
FIG. 3

15. Figs. 4 to 6 show details of columns supported on one or two grillage beams, together with corresponding formulas for determining the effective steel bearing area of columns and beams, and the design of base plates. The following notation is used:

A_b = Effective steel bearing area of beam or beams - in sq. in.
A_c = Effective steel bearing area of column - in sq. in.
P = Total column load - in kip.
d, e, k, p, t_b, t_f and w = Dimensions as indicated in Figs. 4 to 6 in inches.

ONE-BEAM GRILLAGES

16. In computing bearing area, Formula 7 or 8 shall be used, whichever gives the smaller value. The thickness p of base plate for one-beam grillages may be varied as required for bearing from 1/2 in. to 2-1/2 in. If the required bearing area cannot be thus obtained, stiffeners shall be used as shown dotted in Fig. 4, with base plate generally 1/2 in. thick.



$$A_b = (d+2k+2p)t_b \quad (7)$$

$$A_c = dt_c + (4k+4p+2t_b-2t_c)t_f \quad (8)$$

FIG. 4

Bending shall not be considered in designing base plates for one-beam grillages.

One-beam grillages shall generally be avoided in cases where column web and beam web would be normal to each other.

TWO-BEAM GRILLAGES

17. In computing bearing area of grillage beam web, Formulas 9 & 10 shall be used. The thickness p of base plate for two-beam grillages may be varied as required for bearing from 3/4 in. to 2-1/2 in. If sufficient bearing length cannot be thus obtained, stiffeners shall be used on beams as shown dotted in Figs. 5 & 6, with base plate of the required thickness.

The base plate is considered to have sufficient bending strength when its thickness p is equal to e (Figs. 5 & 6), except that when the latter exceeds 2-1/2 in., the plate shall be designed for a bending moment of Pe/2 and its thickness may be increased beyond 2-1/2 in., if necessary.

The base plate, being much more flexible than the grillage, shall not be designed for bending about an axis normal to the length of the beams, but shall be assumed to spread the column load to the beam webs at an angle of 45°.

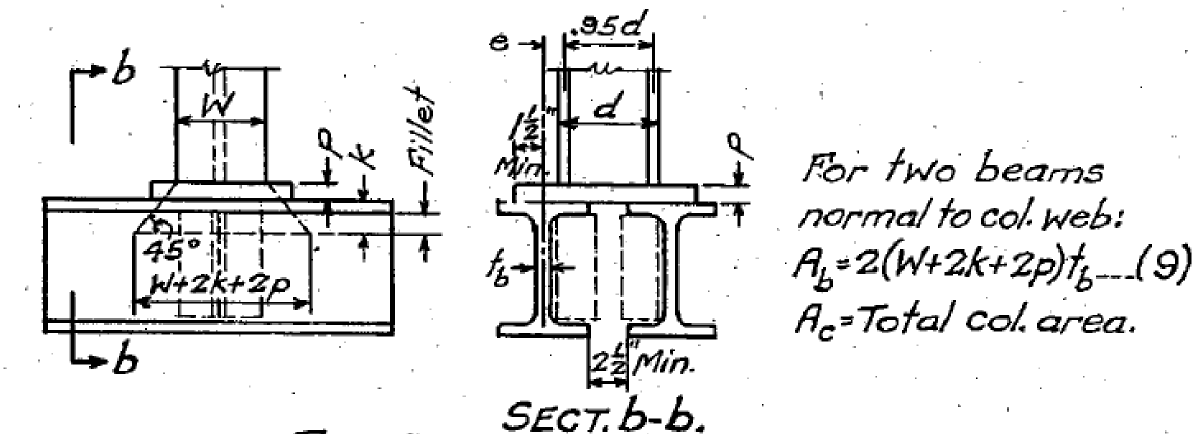


FIG. 5

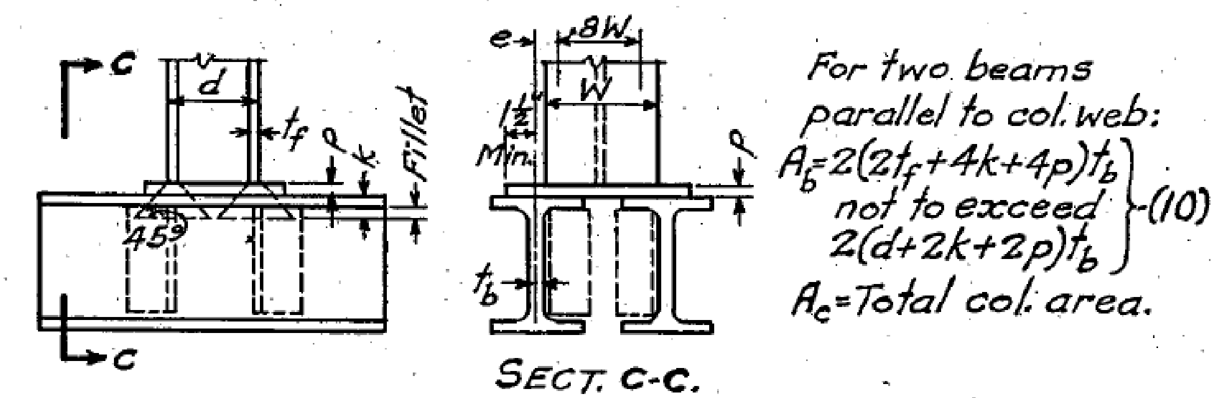


FIG. 6

PLATE GIRDER GRILLAGES

18. Where heavy column loads or other conditions necessitate the use of plate girder grillages, the following rules shall be observed:

Where flanges of plate girders transmit the load directly to the concrete, stiffeners shall not be used for the purpose of strengthening the lower flange, and the concrete and outstanding legs of flange angles shall not be stressed beyond the values given for rolled beam grillages in Sect. 8.

Both upper and lower flanges may be counted on for bearing, but as the load from the upper flange spreads through the concrete to the level of and beyond the lower flange, the amount of this load will depend on the available width of concrete to spread it.

Where the flanges are insufficient for spreading the load to the concrete, or where enough flange rivets to transfer the stress due to longitudinal shear as well as that due to vertical pressure cannot be provided, a tier of grillage beams (Fig. 7) having sufficient bearing area to transmit the load to the concrete as per Sect. 6 shall be placed under such girder, and the bearing of the girder flanges on the concrete shall be disregarded.

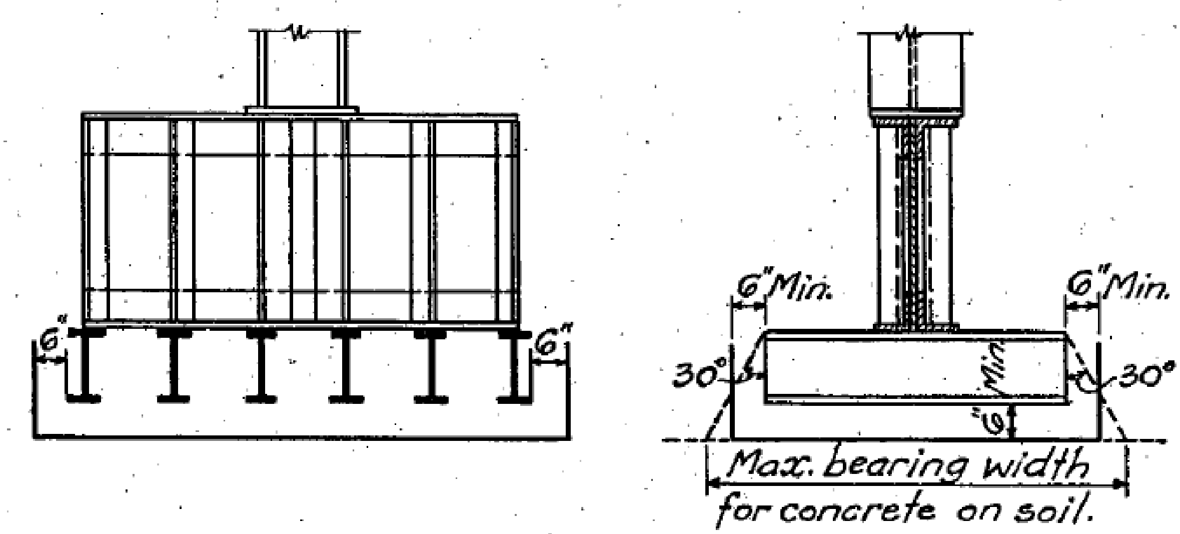


FIG. 7

Stiffeners shall be provided above grillage beams where latter are used, and otherwise as indicated in “Details of Design for Structural Steel,” Sect. 42.

Base plates, when resting on a grillage girder, shall have a thickness of not less than 3/4 inch.

SIDEWALL COLUMN FOOTINGS IN EARTH

19. In earth above water level, for the usual soil bearing value of 8 kip per sq. ft., the type of sidewall footing to be used shall be determined as follows:

When the load carried by column and wall, including their own weight, is 90 kip or less, the concrete at subgrade shall be considered sufficient as a footing.

When the load exceeds 90 kip, one of the following designs may be used, depending on economy and adaptability:

a) Steel cantilever footings, as given on page SD-5-12. (Reinforced concrete cantilever footings shall not be used.)

b) Steel footings projecting beyond the net line of sidewall, as shown in Fig. 8. Double tier grillages (Fig. 9) may be used where single beam footings are insufficient for the load. The weight of the backfill above the projection shall be disregarded in designing these footings because of the confinement of the soil at the projecting end.

c) Invert beams spanning between interior and sidewall columns.

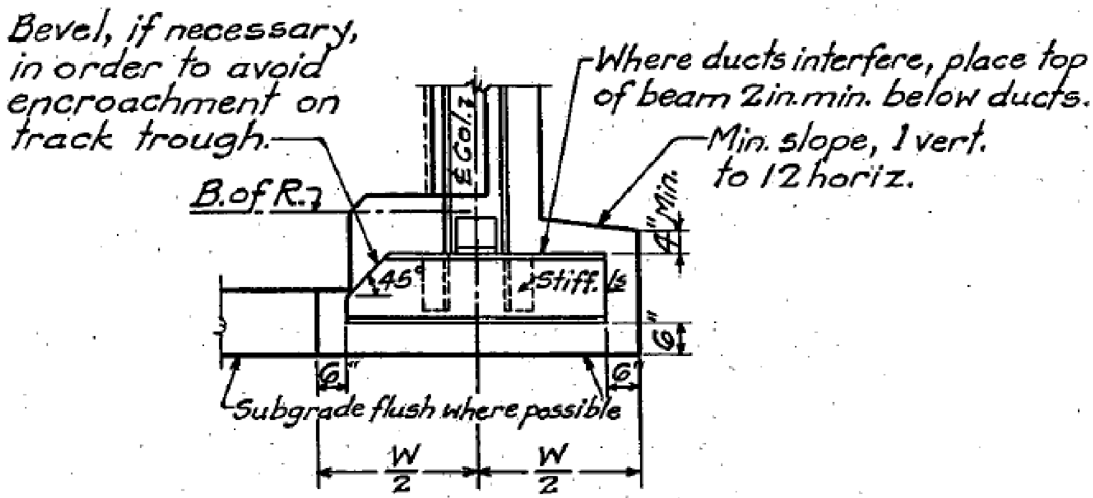


FIG. 8

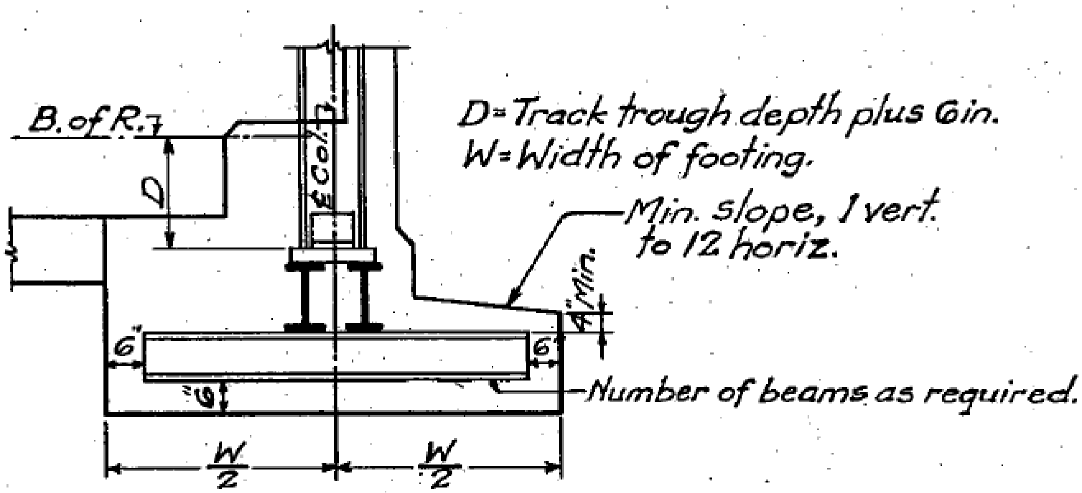


FIG. 9

20. In earth below water level, a steel beam invert for distributing the load over the entire subgrade shall generally be provided. Where the need for such an invert is doubtful, the design shall be made for a reinforced concrete invert so as to avoid waste if its omission is warranted by the conditions at subgrade, as revealed in the excavation. See also “Loads and Stresses,” Sect. 18.

SIDEWALL COLUMN FOOTINGS IN ROCK

21. In rock, when the load carried by column and wall, excluding their own weight, is 200 kip or less, and when the bond stress between column and wall does not exceed 30 lb. per sq. in., the concrete at subgrade shall be considered sufficient as a footing; otherwise, a steel slab or grillage shall be provided under the column to distribute the total column load (see “Loads & Stresses,” Sect. 17). The bond stress between column and wall shall be computed by the following formula (load in lb., dimensions in in.):

$$\text{Bond Stress} = \frac{\text{Total Column Load}}{\text{Col. length times (2 times width of flange plus 2 times depth of sect. minus 5”)}}$$

Footings shall, wherever possible, be designed so as not to project beyond the payment line.

STEEL CANTILEVER FOOTINGS FOR SIDEWALL COLUMNS 5FT. ON CENTERS IN EARTH ABOVE WATER LEVEL.

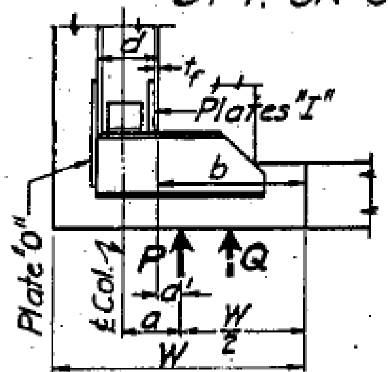


FIG. 51.

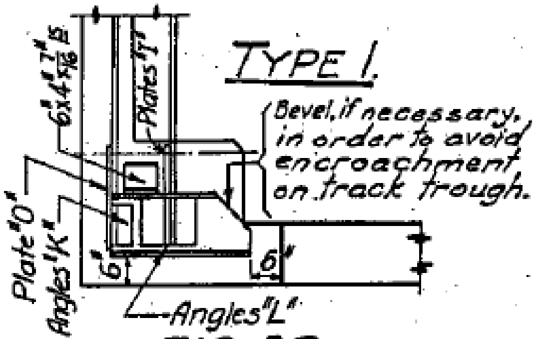


FIG. 52.

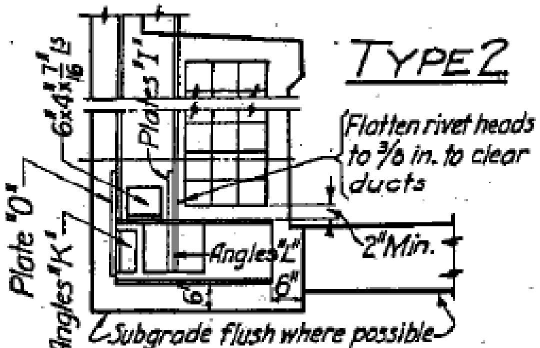


FIG. 53.

FORMULAS (Lengths in in., Loads in kip):

$$Q = \frac{Pb}{W}; \quad M_b = \frac{Qb}{2}; \quad M_c = Pa; \quad T = \frac{Pa'}{d}; \quad C = T + P; \quad f_s = \frac{P}{A} + \frac{M_c}{S}$$

W = Required footing width.

P = End reaction of roof beam, including load to back of column, plus reaction due to intermediate floors, if any, plus weight of wall taken at 0.75 kip per ft. of height.

Q = Upward pressure on portion $\underline{b}$ of footing (= max. shear on beam).

M_b = Maximum moment in cantilever beam.

M_c = Moment at bottom of column.

T = Tension in plate "O".

C = Compression in plates "I" and column flange.

a = Distance from center line of column to center of footing.

a' = Distance from face of column to center of footing.

b = Distance from face of column to toe of footing.

d = Depth of column section.

f_s = Maximum unit stress at bottom of column.

A = Area of column section.

S = Section modulus of column section.

Notes: Bearing on soil assumed uniform at 8 kip per sq. ft.

Value f_s for Type 1 shall not exceed 20 kip per sq. in..

Value f_s for Type 2 shall not exceed 25 kip per sq. in. (22 kip for Bldg. Code).

Bearing length of beam web shall be taken as thickness t_f (Fig. 51) of column flange plus spread through fillet.

Chapter 6

CHAPTER 6

DESIGN OF CONCRETE AND REINFORCED CONCRETE STRUCTURES

TABLE OF CONTENTS

CHAPTER 6

DESIGN OF CONCRETE AND REINFORCED CONCRETE STRUCTURES

The working stress design provisions specified in the “ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63)” as modified below shall be used for design.

However, provisions of other acceptable codes, e.g., N.Y.C. Building Code, may be used for a particular design project, if approved by CEAD.

Use loads as specified in “Loads and Stresses” and “Design of Sidewall Columns.”

No reduction (based on these design standards) shall be made to the Authority’s typical arch construction.

Modifications to ACI 318-63:

1. Use ACI 318-71 Chapter 3 - Materials, in lieu of ACI 318-63 Chapter 4 - Materials.

2. Use ACI 318-71 Chapter 7 - Details of Reinforcement, in lieu of ACI 318-63 Chapter 8 - Details of Reinforcement. (See also Sections 12 and 13, below.)

3. Use ACI 318-71 Chapter 12 - Development of Reinforcement, in lieu of ACI 318-63 Chapter 13 - Bond and Anchorage - Working Stress Design, Section 918 - Anchorage requirements - General and Section 919 - Anchorage of web reinforcement.

   NOTE: Use the corresponding provisions of Chapters 3, 7 and 12 in ACI 318-71 for cross references to Chapters 4, 8 and 13 in ACI 318-63.

4. DENSITY OF PLAIN AND REINFORCED CONCRETE

w = 150 lb./cu. ft.

5. PROPORTION OF MIX

   Use a 1:2:4 mix.

6.. QUALITY OF CONCRETE

   Compressive strength of concrete (f'c) at age of 28 days = 3000 psi.

7. REINFORCING BARS

   Deformed bars shall be rolled from first quality new billet steel, and shall conform to the "Specification of Deformed Billet-Steel Bars for Concrete Reinforcement," ASTM serial designation A615 for grade 60.

   Allowable tensile stress = 24,000 psi.

8. ALLOWABLE STRESSES IN NON-REINFORCED CONCRETE ARCHES

   a. Compression:

      fc = 0.25 f'c = 750 psi

   b. Tension:

      fc = $\sqrt{f'c}$ = 55 psi

   c. Shear:

      v (as a measure of diagonal tension)

      = $\sqrt{f'c}$ = 55 psi

9. ALLOWABLE BOND STRESS BETWEEN CONCRETE AND ROLLED SHAPES

   u = 30 psi

10. TRACK SLABS

    A. Minimum Thickness

       The minimum total thickness of reinforced concrete one-way slabs supporting tracks shall be 1/12 of the span.

B. Allowable Stresses

a. Concrete: Multiply all allowable stresses specified in these design standards by 0.55.

b. Reinforcing Steel: Multiply allowable stress specified in these design standards by 0.75.

c. Shear Reinforcement: Provide a minimum of shear reinforcement as specified in Section 11, below. Shear reinforcement shall consist of stirrups perpendicular to the longitudinal reinforcement.

11. SHEAR REINFORCEMENT

Provide a minimum of shear reinforcement in all primary reinforced concrete beams and girders. This reinforcement shall consist of at least #3 stirrups perpendicular to the longitudinal reinforcement and spaced not further apart than d/2.

12. COVERING OF BARS

The distance from the concrete surface to the outermost surface of the steel shall be as follows:

	Minimum	Standard
a) In wall footings and column footings	3”	3½”
b) At surface exposed to weather	2”	2½”
c) In beams and columns	2”	2½”
d) In slabs and walls	1”	1½”
e) In thin slabs (effective depth less than 4”)	3/4”	1”

13. SPACING OF BARS IN A LAYER

In beams and slabs, the minimum center to center distance of parallel bars shall be 4” for No. 6 or smaller bars. For larger bars, the minimum clear space between bars shall be 4”.

14. NEGATIVE REINFORCEMENT IN SIMPLE BEAMS AND SLABS

In simple beams and slabs supported on interior or side walls, negative reinforcement of at least 1/2 times the maximum area of the positive reinforcement shall be placed above the supports to prevent cracking. These bars shall have a length of at least 50 times their thickness and shall extend to the quarter points of adjoining spans.

15. BEARING FOR STEEL BEAMS

Where steel beams rest on concrete walls, provide bed plates set back from face of wall a distance of 1/4 times the wall thickness, not to exceed 3”. The bed plates shall be considered as transmitting uniformly the full end reaction.

Chapter 7

CHAPTER 7

WATERPROOFING OF SUBWAYS WITH CONSTRUCTION FOR SUBWAY FLOOR AND DETAILS OUTSIDE OF STEEL IN ROOF AND SIDEWALKS

TABLE OF CONTENTS

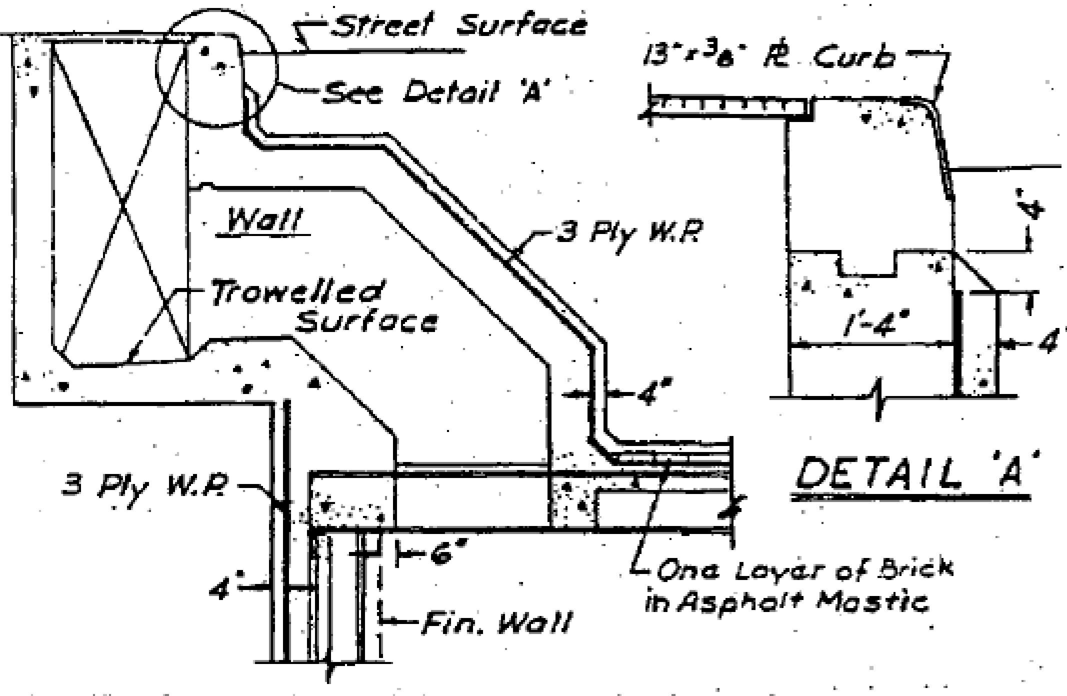
FLOORS OF SIDE PLATFORMS, PASSAGEWAYS & ENTRANCES

Top of Finished Floor Higher than 5' Above El. 100.0 or Ground Water Level	Earth	No W.P.-8" Conc. No. 5 Transverse Rods - 12" Centers
	Rock	No Waterproofing - 4" Concrete
Top of Finished Floor Between El. 100.0 or Ground Water Level & 5' Above	Earth	8" Concrete - 3 Ply Waterproofing - 4" Concrete
	Rock	4" Concrete - 3 Ply Waterproofing - 4" Concrete
Floor Below El. 100.0 or Ground Water Level	Earth	Steel Invert - 4" Conc.-1 Layer of Brick in Asph. Mastic - 1 Ply W. P. 6" Conc.
	Rock	Reinforced Concrete Invert-1 Layer of Brick in Asph. Mastic - 1 Ply W.P. - 6" Conc.
	Rock	Platform Concrete - 4 Ply Waterproofing - 4" Concrete

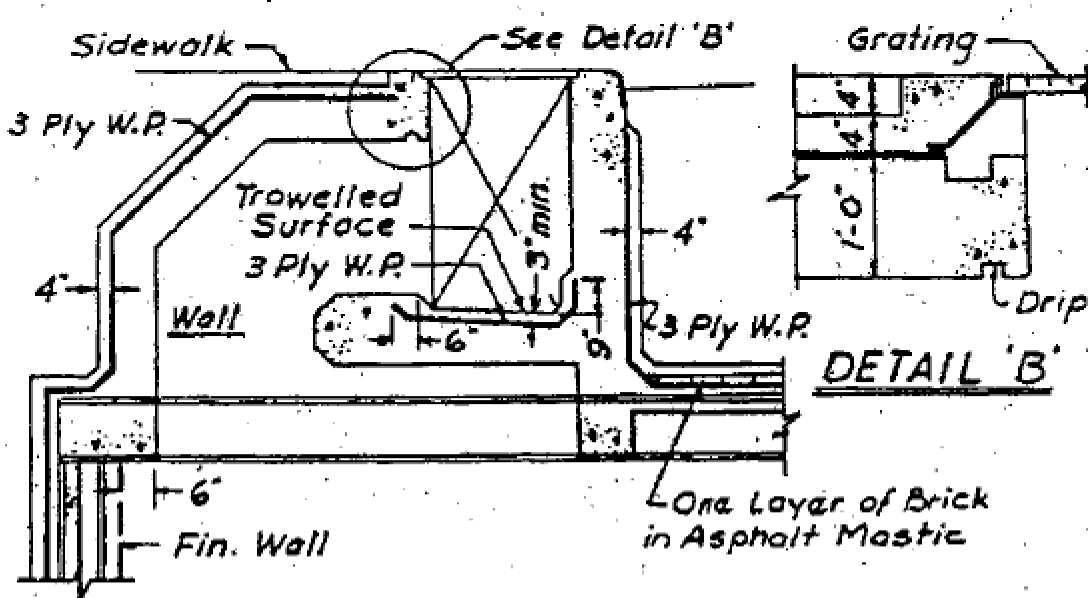
NOTES FOR WATERPROOFING OF SUBWAYS:

1. Chiller Rooms, Climate Changer Rooms, Under Platforms, Fan Rooms, Fan Chambers and Control Rooms, Elevator and Escalator Machinery Rooms and Pits, Ejector Rooms, Sump and Pump Chambers, Circuit Breaker Houses, Electrical Distribution Rooms, Electrical Panel Rooms, Substations, Relay Rooms, Signal Tower Control Rooms, Central Instrument Rooms, Communications Rooms, Compressor Rooms, Telephone Compartment Rooms & Cable Shafts to be completely protected by waterproofing.

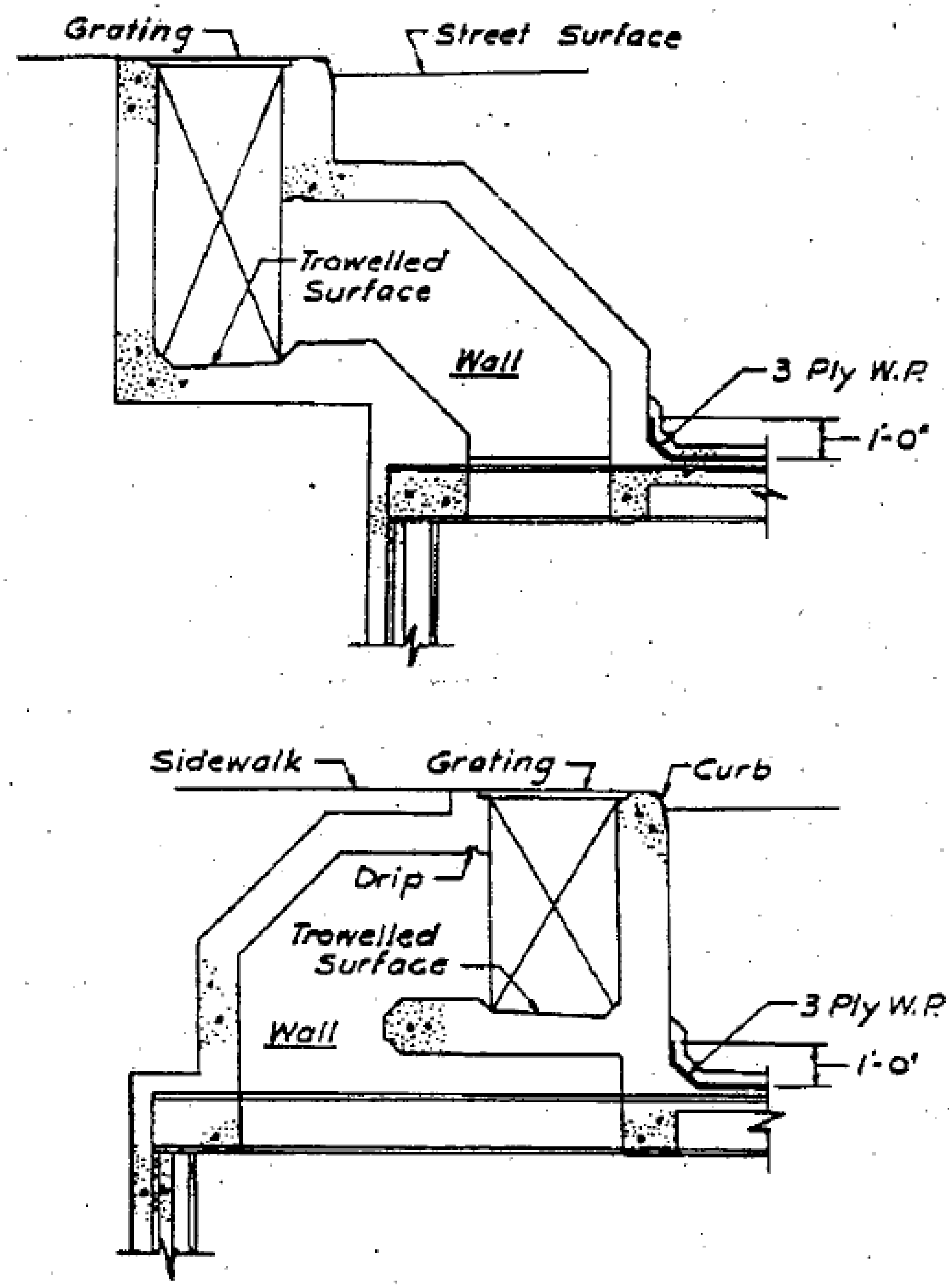
2. At the start of each contract, a recommendation for the use of either (a) ASPHALTIC membrane waterproofing or (b) ELASTOMERIC membrane waterproofing shall be submitted to the Civil Engineering and Architectural Division for APPROVAL.



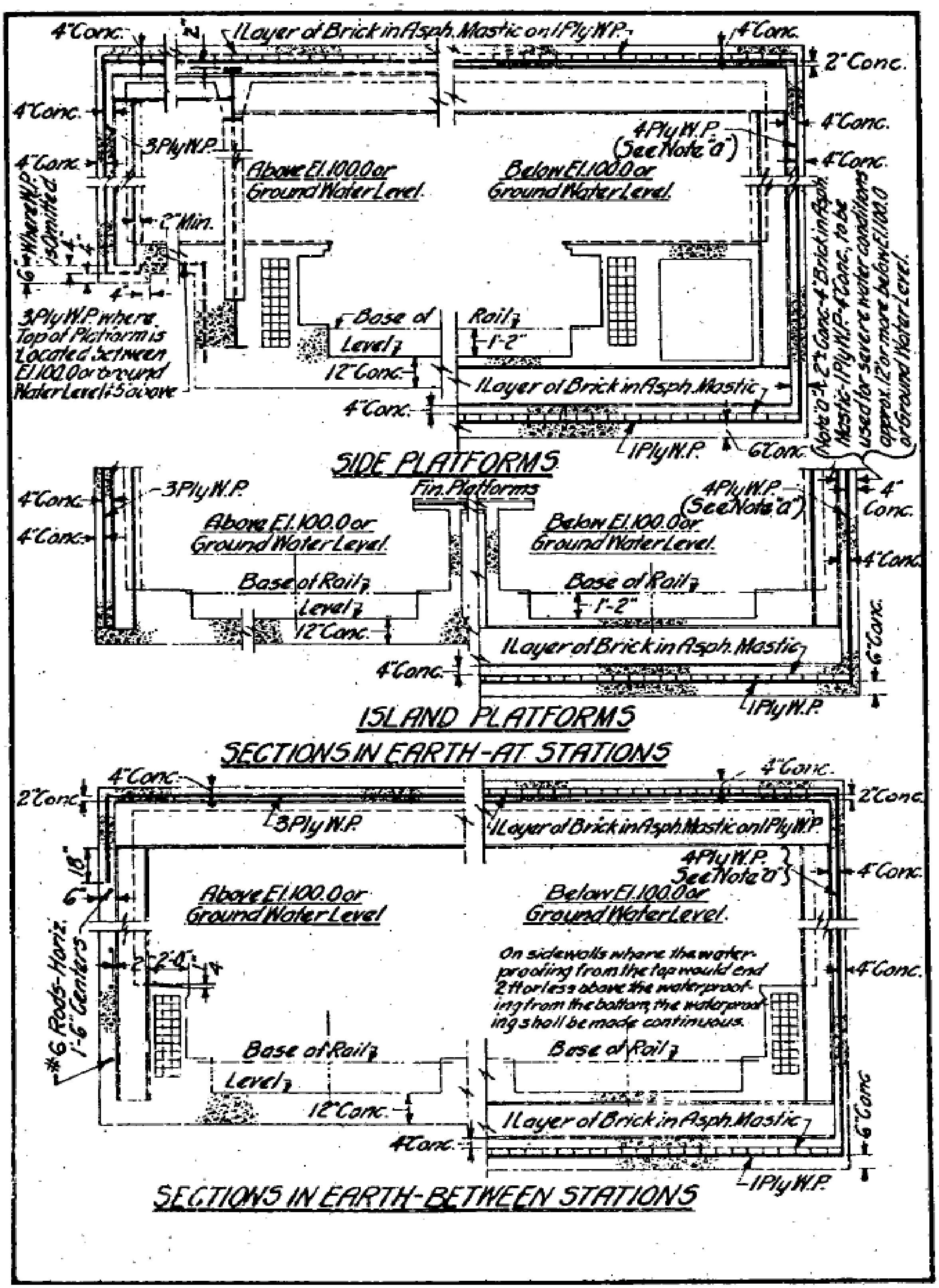
WATERPROOFING AT VENTILATOR AT STATIONS (ABOVE EL. 100.0 OR GROUND WATER)



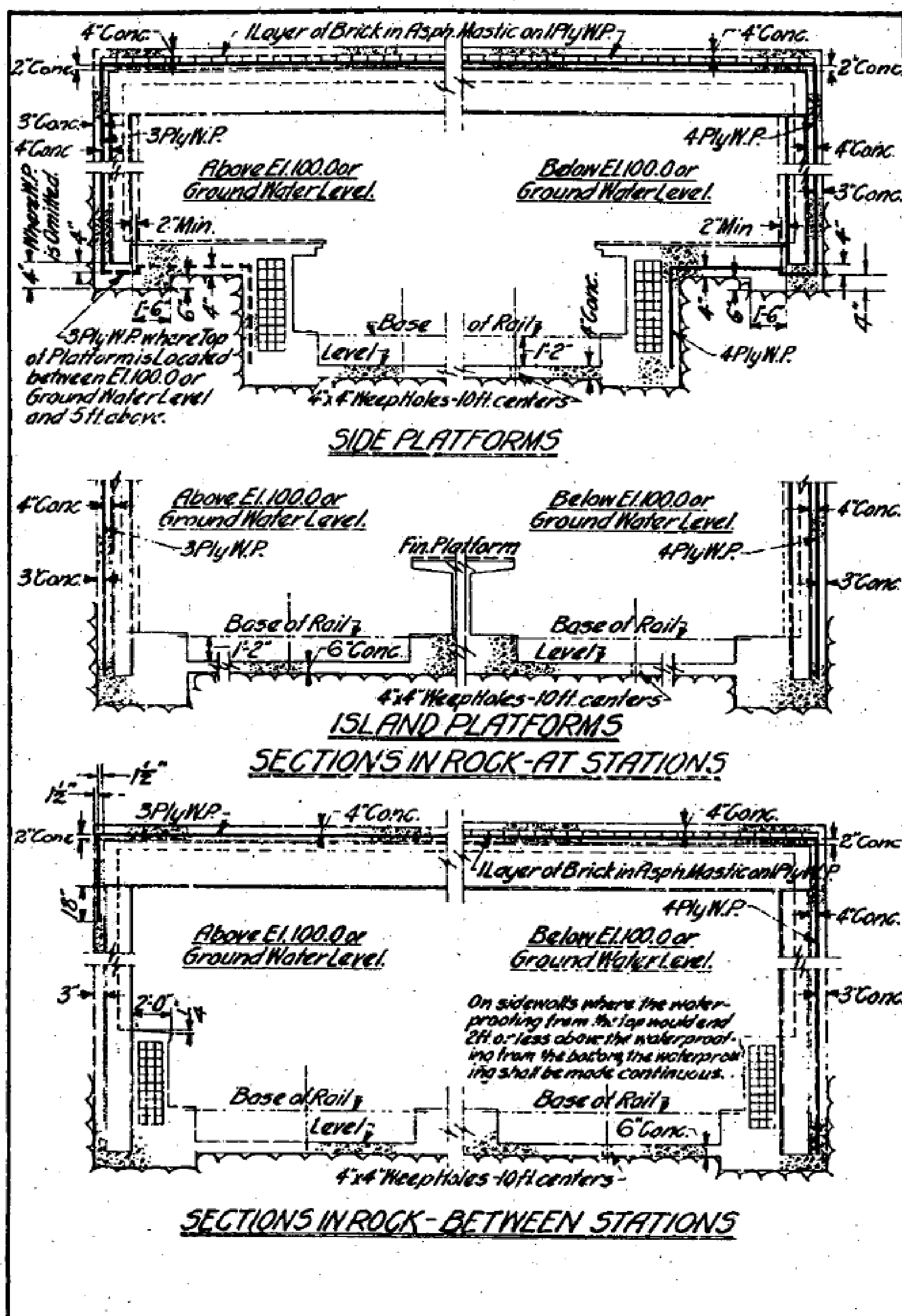
WATERPROOFING AT VENTILATOR AND DRIP PAN — AT STATIONS (ABOVE EL. 100.0 OR GROUND WATER)



WATERPROOFING AT VENTILATORS BETWEEN STATIONS (ABOVE EL. 100.0 OR GROUND WATER)



SIDE PLATFORMS
ISLAND PLATFORMS
SECTIONS IN EARTH-AT STATIONS
SECTIONS IN EARTH-BETWEEN STATIONS
1 Layer of Brick in Asph. Mastic on 1 Ply W.P.
Above El.100.0 or Ground Water Level.
Below El.100.0 or Ground Water Level.
4 Ply W.P. (See Note "a")
Base of Rail Level
12" Conc.
4" Conc.
2" Conc.
6" Conc.
1 Ply W.P.
3 Ply W.P.
1 Layer of Brick in Asph. Mastic
Fin. Platforms
3 Ply W.P. where Top of Platform is Located between El.100.0 or Ground Water Level & Above
On sidewalls where the waterproofing from the top would end 2 ft or less above the waterproofing from the bottom, the waterproofing shall be made continuous.
#6 Rods-Horiz. 1'-6" Centers


4"Conc.
1 Layer of Brick in Asph. Mastic on 1 Ply W.P.
4"Conc.
2"Conc.
2"Conc.
3"Conc.
3 Ply W.P.
4"Conc.
4 Ply W.P.
4"Conc.
Above El. 100.0 or Ground Water Level.
Below El. 100.0 or Ground Water Level.
3"Conc.
2"Min.
2"Min.
4"Conc.
Base of Rail Level
1'-2"
1'-6"
3 Ply W.P. where Top of Platform is Located between El. 100.0 or Ground Water Level and 5 ft. above.
4"x4" Weep Holes - 10 ft. centers
4 Ply W.P.
SIDE PLATFORMS
4"Conc.
Above El. 100.0 or Ground Water Level.
Below El. 100.0 or Ground Water Level.
4"Conc.
3 Ply W.P.
Fin. Platform
4 Ply W.P.
3"Conc.
3"Conc.
Base of Rail
Base of Rail Level
1'-2"
6"Conc.
4"x4" Weep Holes - 10 ft. centers
ISLAND PLATFORMS
SECTIONS IN ROCK-AT STATIONS
1½"
1½"
3 Ply W.P.
4"Conc.
4"Conc.
2"Conc.
2"Conc.
1 Layer of Brick in Asph. Mastic on 1 Ply W.P.
18"
4 Ply W.P.
4"Conc.
Above El. 100.0 or Ground Water Level.
Below El. 100.0 or Ground Water Level.
3"
2'-0"
3"Conc.
On sidewalls where the waterproofing from the top would end 2 ft. or less above the waterproofing from the bottom the waterproofing shall be made continuous.
Base of Rail Level
Base of Rail
6"Conc.
4"x4" Weep Holes - 10 ft. centers
SECTIONS IN ROCK - BETWEEN STATIONS

Chapter 8

CHAPTER 8

NATURAL VENTILATION DESIGN CRITERIA BETWEEN STATIONS

TABLE OF CONTENTS

CHAPTER 8

NATURAL VENTILATION DESIGN CRITERIA BETWEEN STATIONS

1. A blast shaft with 200 $\pm$ square feet net grating area shall be provided at the entrance portal of each station. (See layout on page SD-8-5).

2. In addition to the blast shaft, vent shafts with a net grating area of 1.0 square foot for every 850 cubic feet of tunnel volume shall be spaced uniformly between the stations in banks of 6 to 8 bays per bank. The first bank of gratings shall be placed as close to the end of the previous station as practical and will function as relief vents for that station. Tunnel volume shall be measured from end of platform of previous station to beginning of platform of next station, not including volume of enclosures.

3. Vent shafts for emergency fan chambers between stations shall be utilized for natural ventilation net area requirements by providing an inlet opening to bypass fans where feasible.

4. Large single shafts shall be substituted for banks of vents when the subway is constructed under deep cover. The net area of the large single shaft shall be equal to the net grating area of the substituted banks of vent.

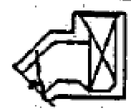
5. The net cross-sectional area of the vent shaft safety grill or other obstructions shall not be less than the net area of the grating. The net grating area is the cross-sectional area of the opening minus the area occupied by bearing and cross bars.

6. The standard vent shafts shown on pages SD-8-3 and SD-8-4 shall be used except where a sidewalk width is less than 13 feet or where utility restoration precludes a standard 4’-0” x 4’-2” vent shaft, in which case the modified vent shaft shown on pages SD-8-9, SD-8-10 and SD-8-11 shall be used.

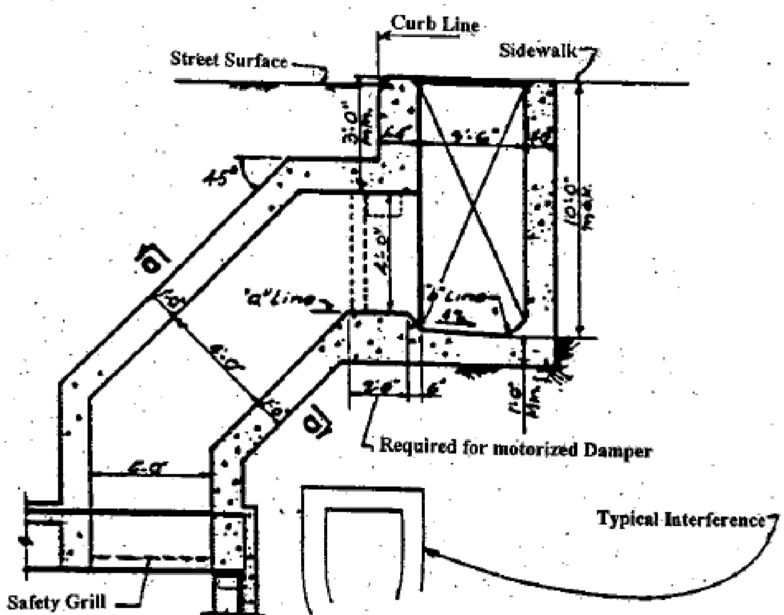
STANDARD VENT SHAFT

Assume Net Grating Area =13.75sq.ft. per 5 ft. Bay for Net Area Computation

MODIFIED VENT SHAFT



Assume Net Grating Area =11.75sq.ft. per 5 ft. Bay for Net Area Computation



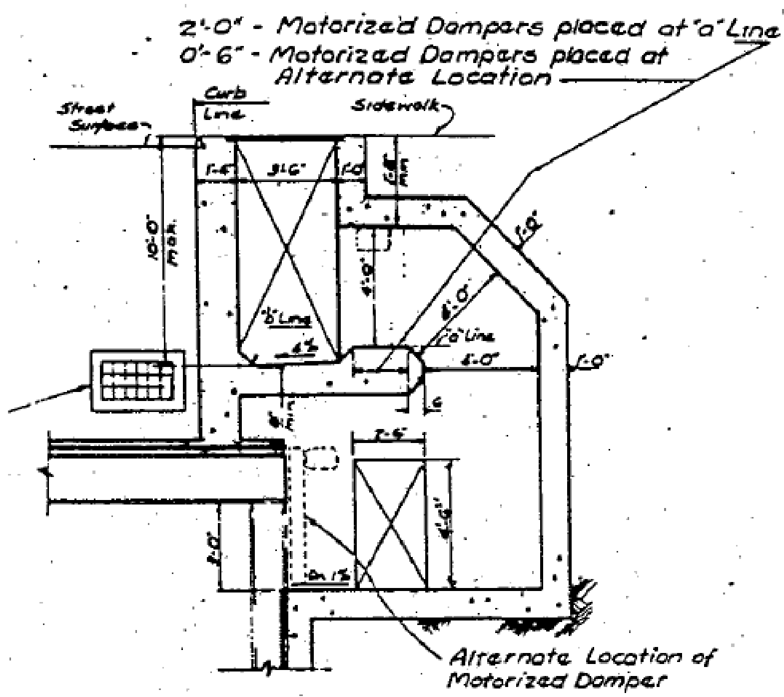
VENT SHAFT THRU ROOF

(FLUE UNDER ROADWAY)

NOTE: FLUE ACCESS TO TRACKWAY WILL BE DETERMINED BY LOCATION OF SUBSURFACE STRUCTURES AND UTILITIES.

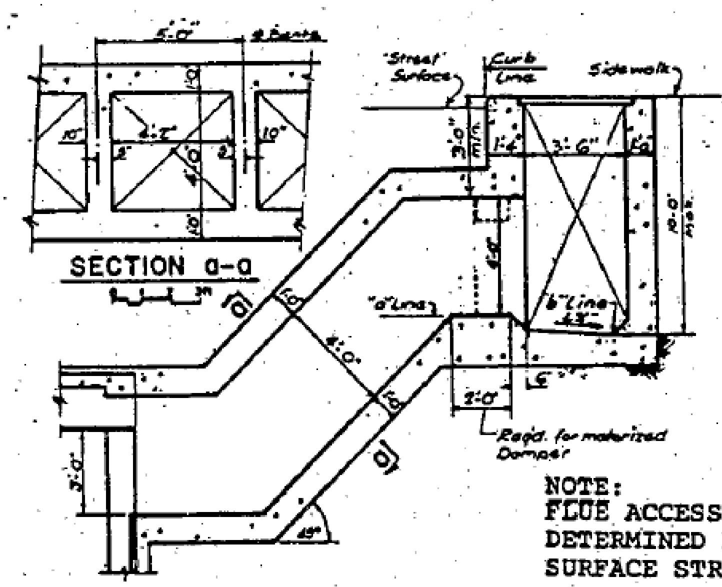
CROSS REFERENCES

For Section a-a ------ See Page SD-8-3



VENT SHAFT THRU SIDEWALL

(FLUE UNDER SIDEWALK)

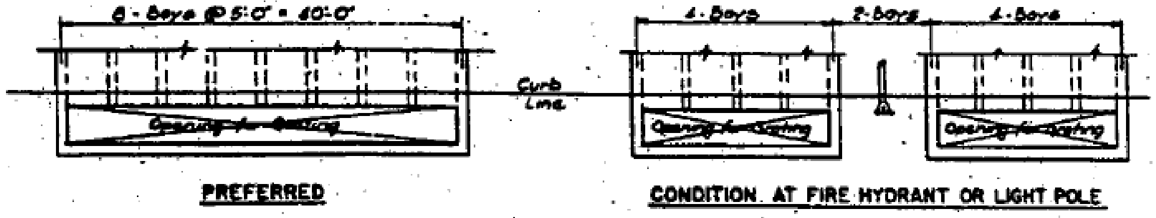


NOTE:
FLUE ACCESS TO TRACKWAY WILL BE DETERMINED BY LOCATION OF SUB-SURFACE STRUCTURES AND UTILITIES

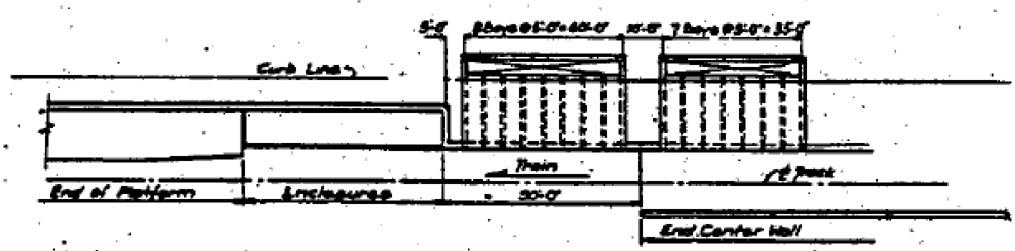
VENT SHAFT THRU SIDEWALL

(FLUE UNDER ROADWAY)

NATURAL VENTILATION DESIGN CRITERIA AND VENT SHAFT STANDARDS



LAYOUT OF STANDARD BANK OF 8 VENT SHAFTS



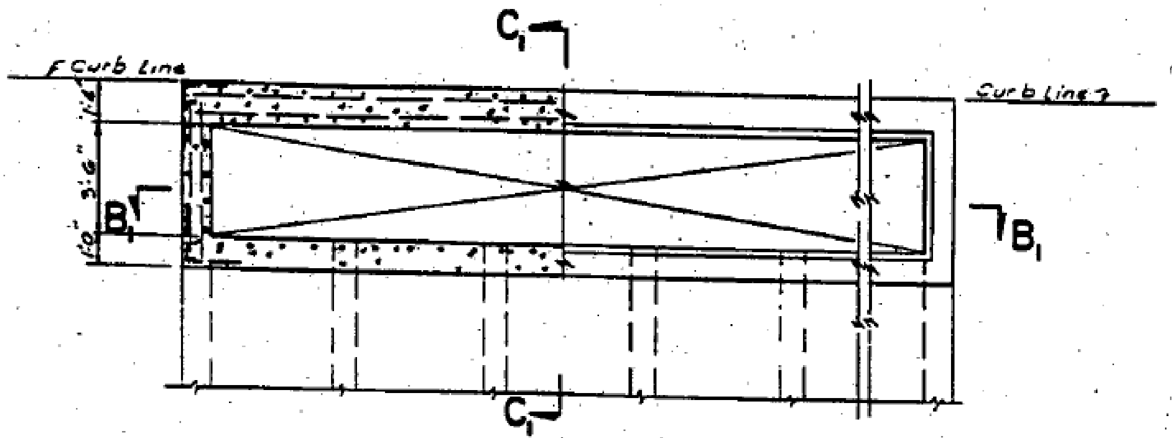
LOCATION OF BLAST SHAFT GRATINGS

CROSS REFERENCES

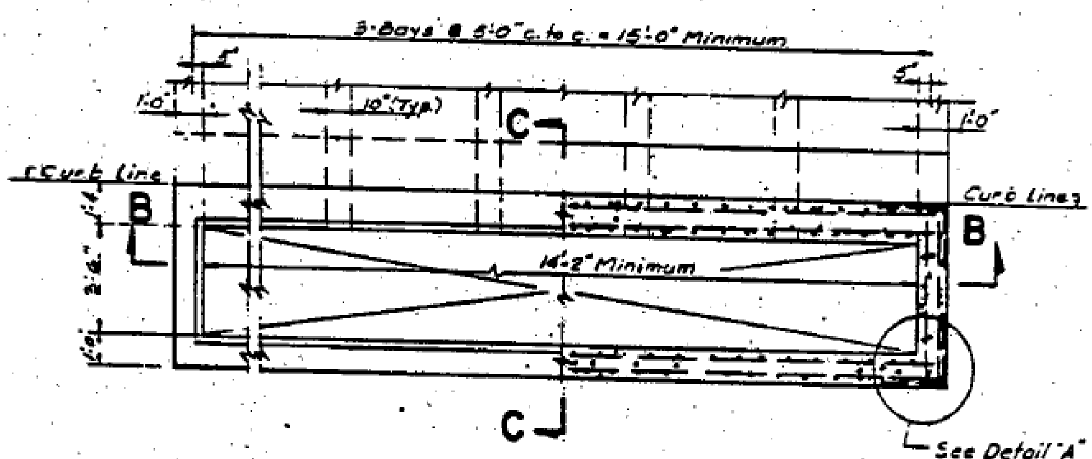
For Standard Vent Shaft
Construction Details - - - - See Pages SD-8-5, SD-8-6, SD-8-7

For Modified Vent Shaft
Standards - - - - - - - - - - - See Pages SD-8-8, SD-8-9. SD-8-10

For Modified Vent Shaft
Construction Details - - - - See Pages SD-8-11, SD-8-12, SD-8-13



PLAN A-A
(FLUE UNDER SIDEWALK)

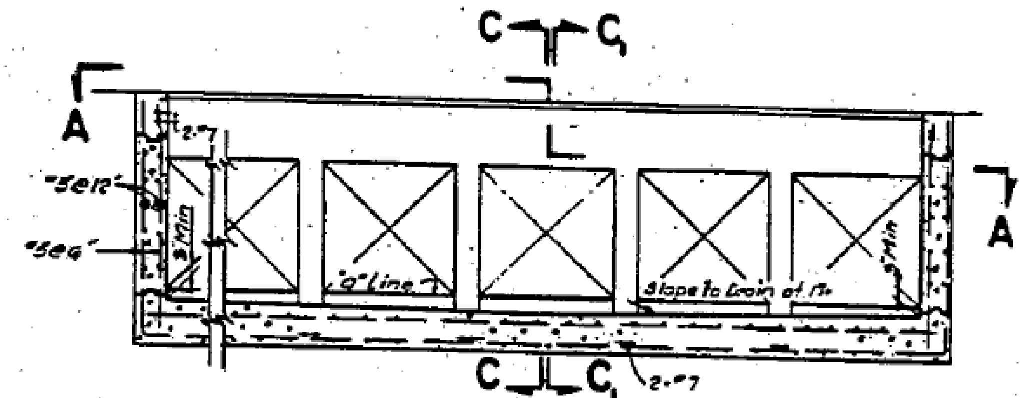


PLAN A-A
(FLUE UNDER STREET)

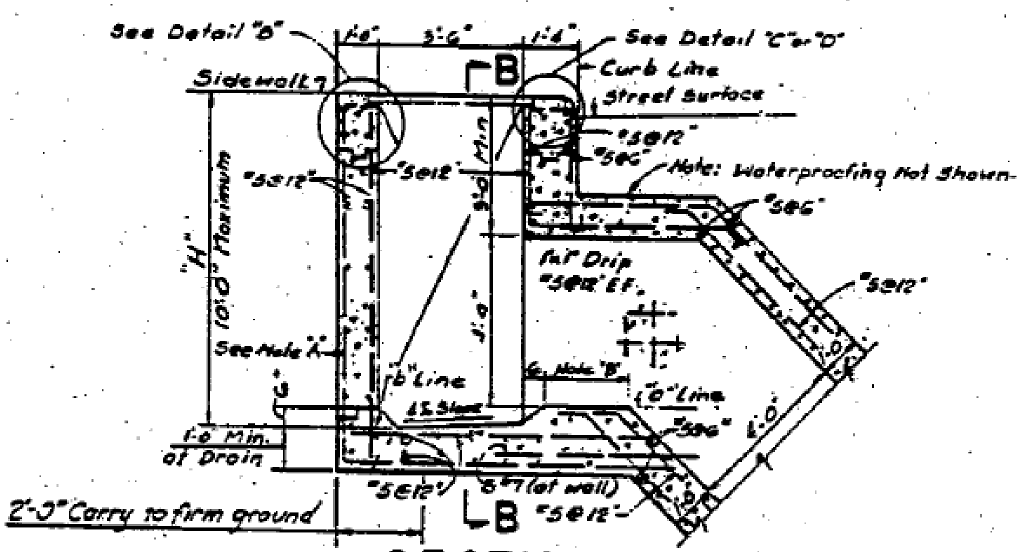
CROSS REFERENCES

For Sections B-B, B_1-B_1 & C-C ------ See Page SD-8-6
For Section C_1-C_1 ------------- See Page SD-8-7
For Detail "A" --------------- See Page SD-8-7

STANDARD VENT SHAFT CONSTRUCTION DETAILS



SECTION B–B
SECTION B1–B1 (Similar)
Scale 0 1 2 3 Ft.

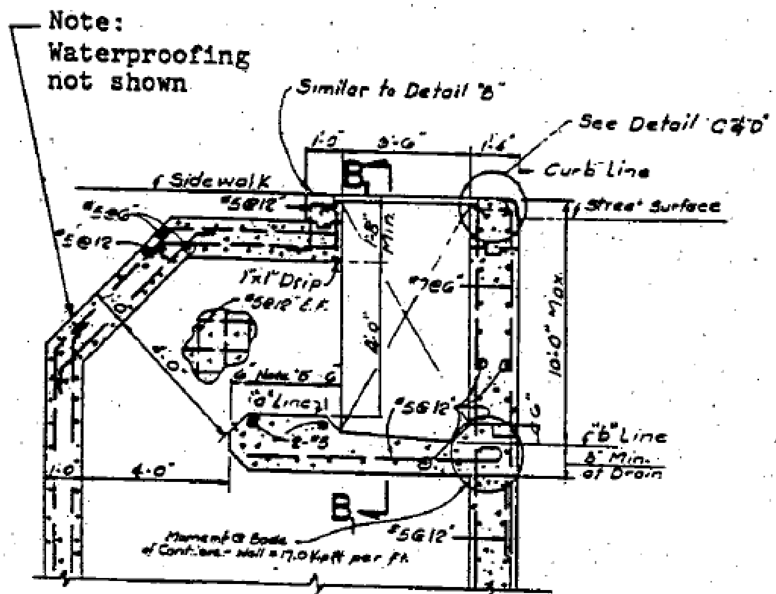


SECTION C–C

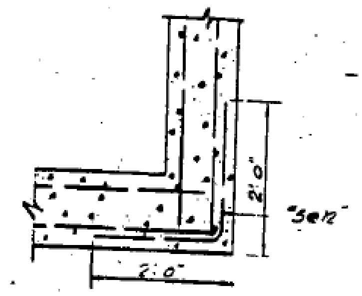
CROSS REFERENCES

For Plan A-A---------- See Page SD-8-5
For Details "B","C" &"D" See Page SD-8-7
For Drip Pan Drainage - - - - - See Design Criteria for Hydraulics .
For Waterproofing - - - - - - - - See Pages SD-7-2,SD-7-3

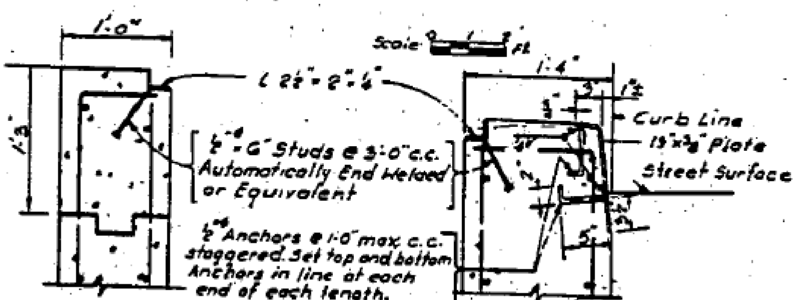
STANDARD VENT SHAFT CONSTRUCTION DETAILS



SECTION C₁–C₁



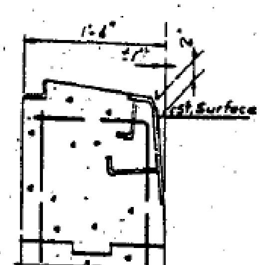
DETAIL "A"



DETAIL "B"

DETAIL "C"

TYPE "D" STEEL FACED CURB



DETAIL "D"

TYPE "D" STEEL FACED CURB AT DRIVEWAYS

CROSS REFERENCES

For Section B₁–B₁ ---- See Page SD-8-6

For Note "B" --------- See Page SD-8-6

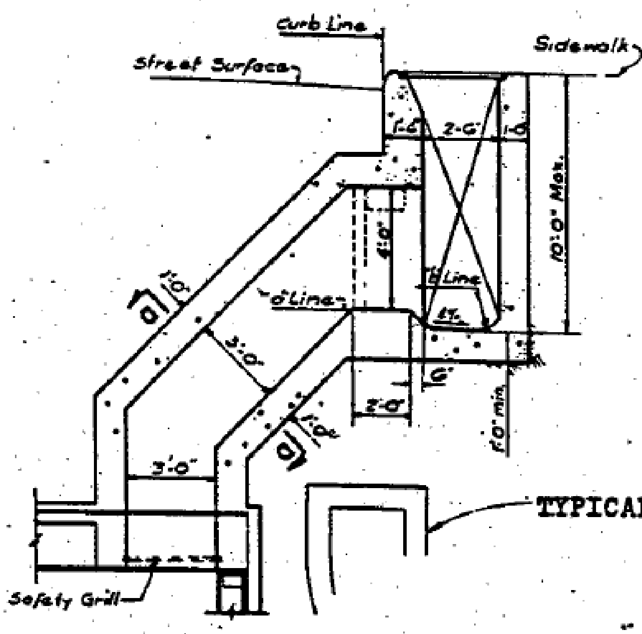
For Drip Pan Drainage - - - - - See Design Criteria for Hydraulics

For Waterproofing - - - - - - - See Pages SD-7-2, SD-7-3

STANDARD VENT SHAFT CONSTRUCTION DETAILS

SD-8-7

NOTE:
THE MODIFIED VENT SHAFT SHOWN ON PAGES SD-8-8, SD-8-9& SD-8-10 SHALL BE USED ONLY WHERE THE SIDEWALK WIDTH IS LESS THAN 13 FEET, OR WHERE UTILITY RESTORATION PRECLUDES THE USE OF THE STANDARD VENT SHAFT CONFIGURATION SHOWN ON PAGES SD-8-2 &SD-8-3



MODIFIED VENT SHAFT
THRU ROOF
(FLUE UNDER ROADWAY)

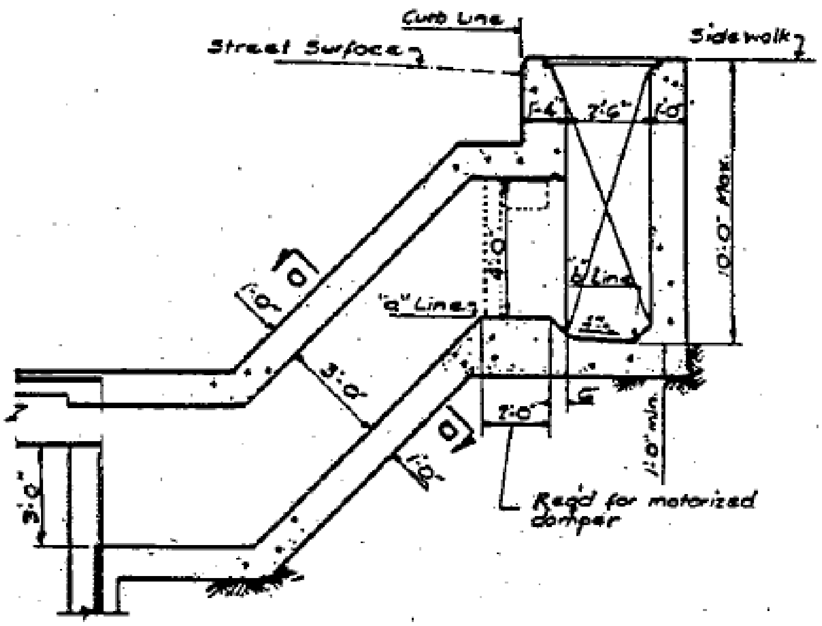
NOTE:
THE MODIFIED VENT SHAFT SHOWN ON THIS SHEET SHALL BE USED ONLY WHERE THE SIDEWALK WIDTH IS LESS THAN 13 FEET, OR WHERE UTILITY RESTORATION PRECLUDES THE USE OF THE STANDARD VENT SHAFT CONFIGURATION SHOWN ON PAGES SD-8-3, SD-8-4, SD-8-5

CROSS REFERENCES

FOR SECTION a-a - - - - - - - - - - SEE PAGE SD-8-9

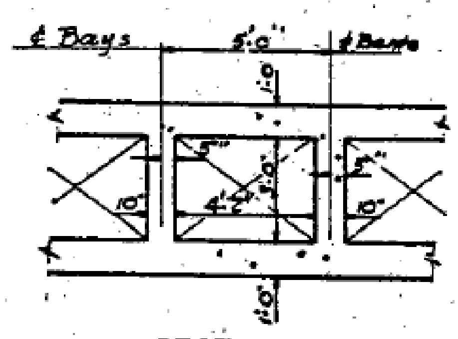
FOR MODIFIED VENT SHAFT CONSTRUCTION DETAILS - - - - SEE PAGES SD-8-11, SD-8-12, & SD-8-13

MODIFIED VENT SHAFT CONSTRUCTION STANDARDS



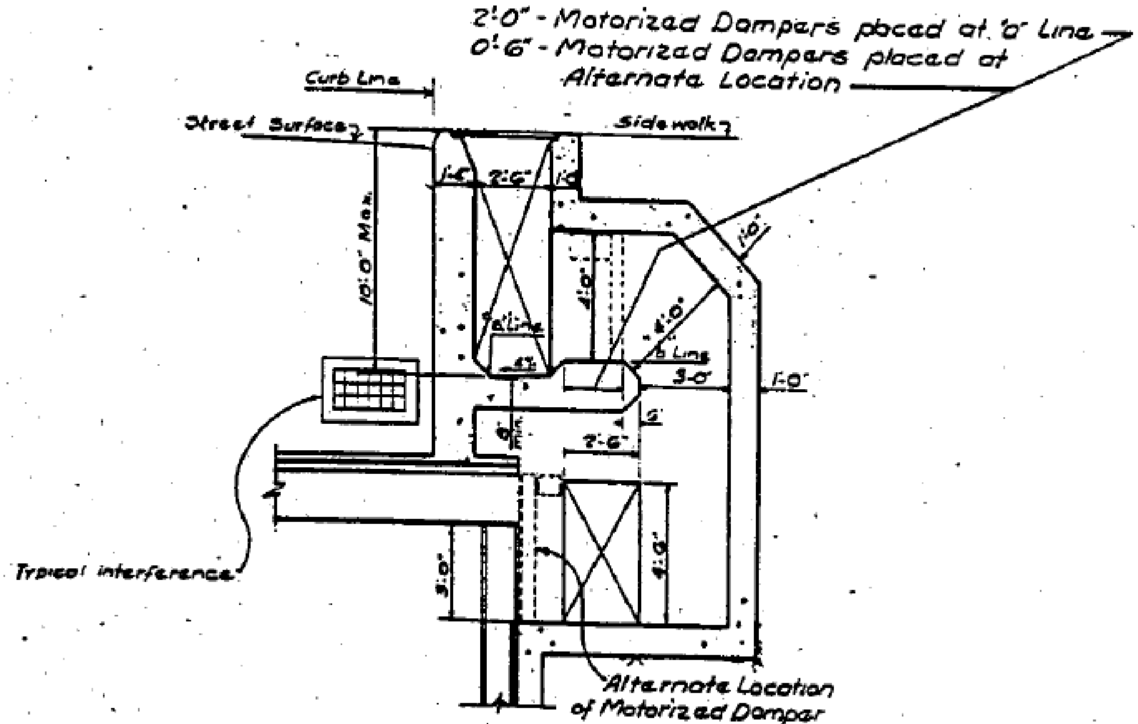
MODIFIED VENT SHAFT THRU SIDEWALL

(FLUE UNDER ROADWAY)



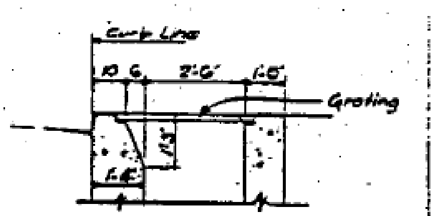
SECTION a-a

MODIFIED VENT SHAFT STANDARDS

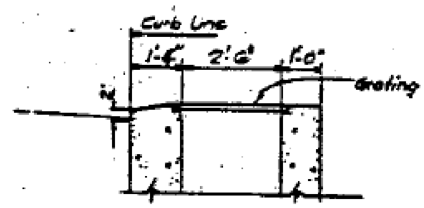


MODIFIED VENT SHAFT THRU SIDEWALL

(FLUE UNDER SIDEWALK)

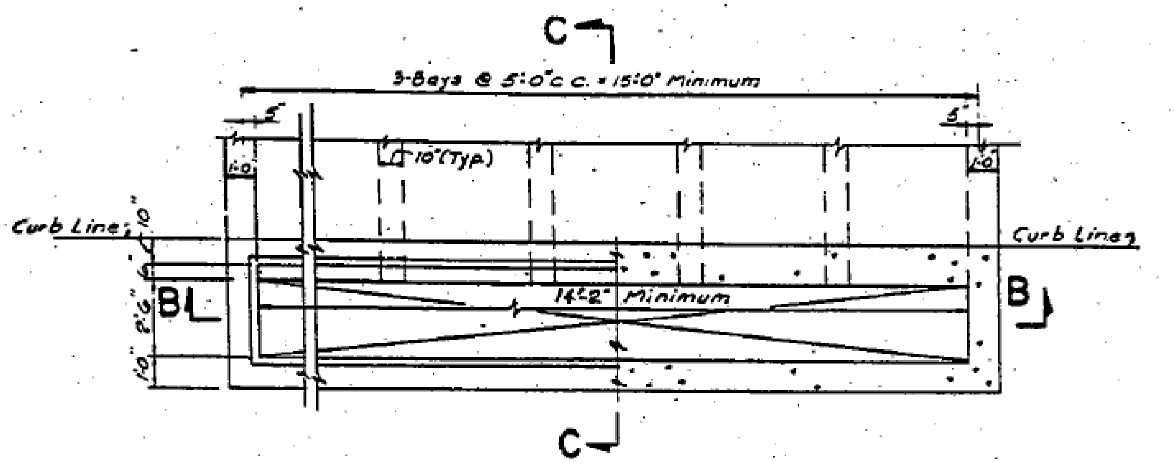


Modified Vent Top Detail

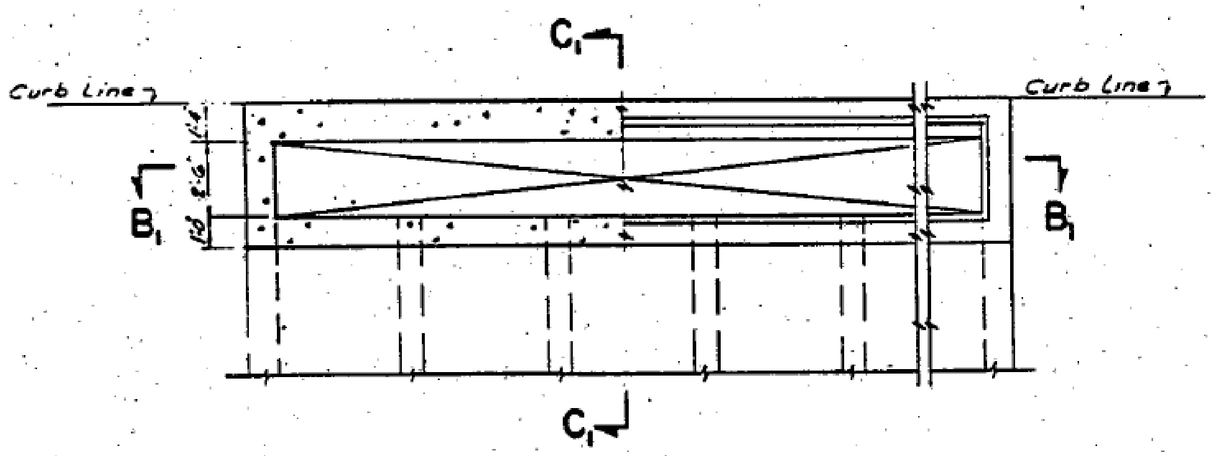


Modified Vent Top Detail at Driveway

MODIFIED VENT SHAFT STANDARDS



PLAN A-A
(FLUE UNDER STREET)
Scale: 0 1 2 3 Ft.

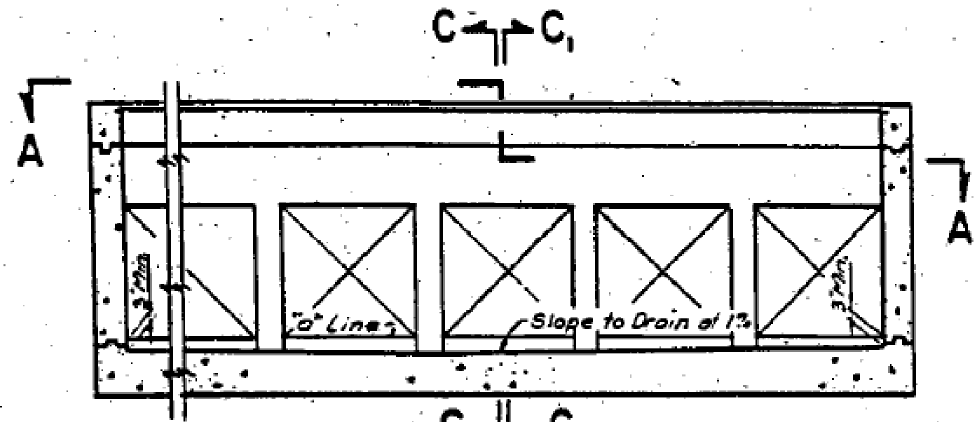


PLAN A-A
(FLUE UNDER SIDEWALK)
Scale 0 1 2 3 Ft.

CROSS REFERENCES

For Sections B-B, B_1-B_1 &C-C See Page SD-8-12
For Section C_1-C_1------------ See Page SD-8-13
For Reinforcing------------- See Note on Page SD-8-13

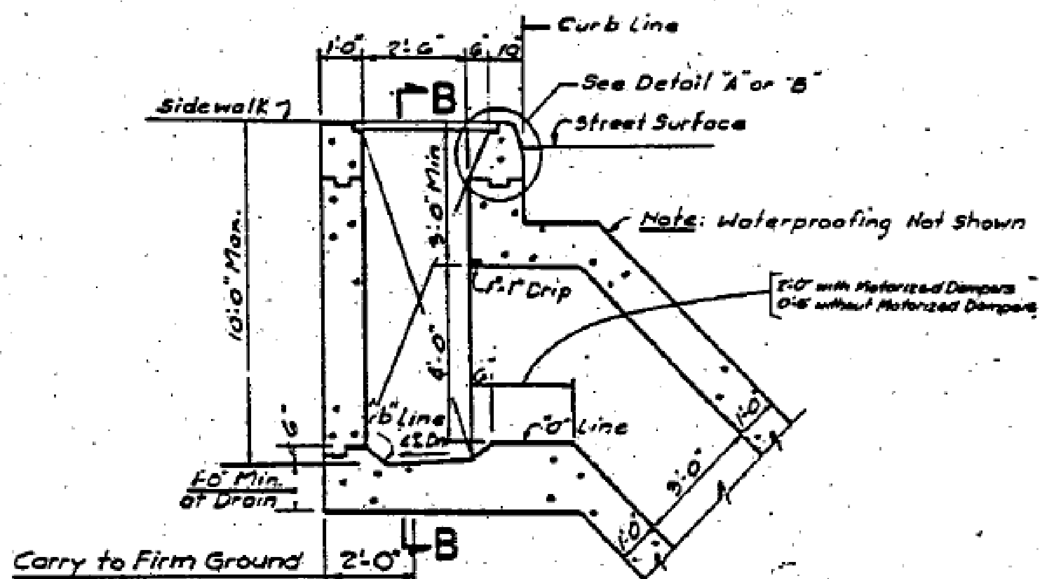
MODIFIED VENT SHAFT CONSTRUCTION DETAILS



SECTION B—B

SECTION B1—B1 (Similar)

Scale 0 1 2 Ft.



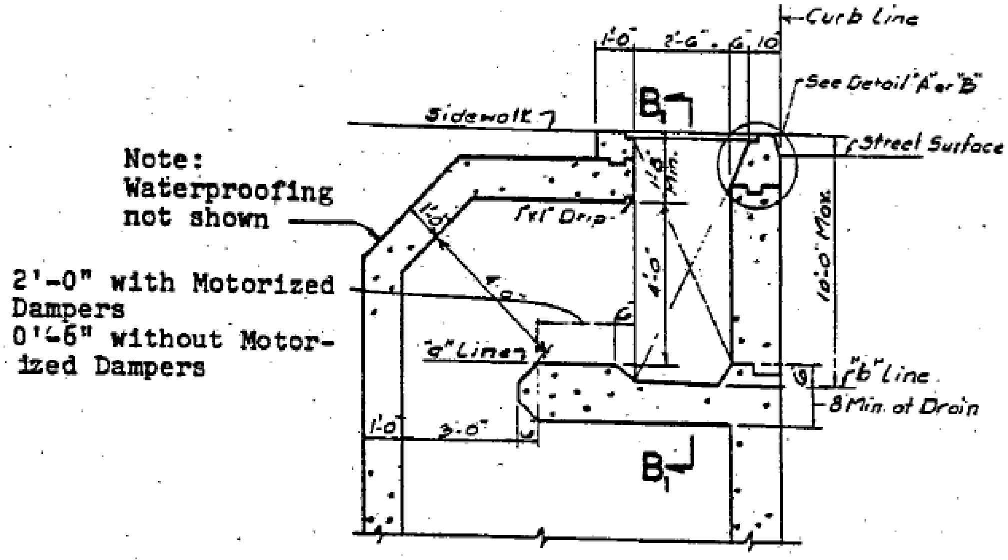
SECTION C—C

Scale 0 1 2 Ft.

CROSS REFERENCES

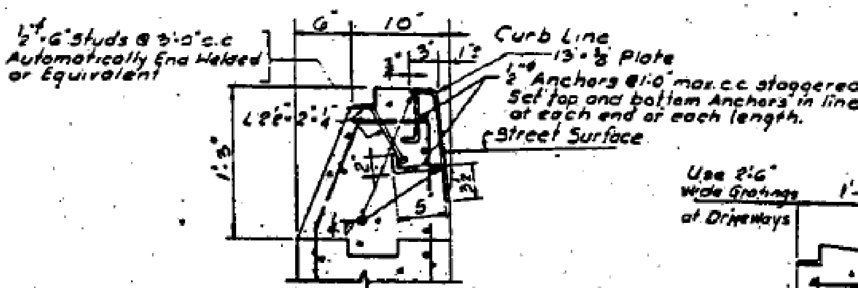
For Plan A-A	See Page SD-8-11
For Section C1-C1	See Page SD-8-13
For Details "A" & "A"	See Page SD-8-13
For Reinforcing	See Note on Page SD-8-13

MODIFIED VENT SHAFT CONSTRUCTION DETAILS



SECTION C_1-C_1

Scale 0 1 2 Ft.

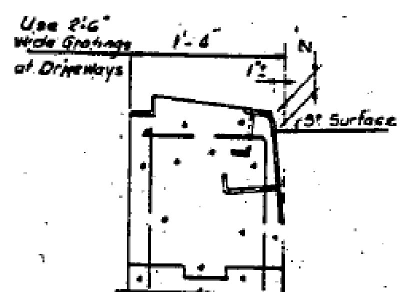


DETAIL "A"

TYPE "D" STEEL FACED CURB

Scale 0 4 8 In.

NOTE:
FOR LOCATION AND SIZE OF REINFORCING RODS SEE SIMILAR PLANS AND SECTIONS ON PAGES SD-8-5, SD-8-6 & SD-8-7.



DETAIL "B"

TYPE "D" STEEL FACED CURB AT DRIVEWAYS

Scale 0 4 8 In.

FOR SECTION B_1-B_1 ----- See Page SD-8-12

MODIFIED VENT SHAFT CONSTRUCTION DETAILS