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PAPERS AND DISCUSSIONS

OF GENERAL INTEREST

CONTENTS AND INDEX — VOLUME 41, 1954

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JOURNAL OF THE

BOSTON SOCIETY OF CIVIL ENGINEERS

Volume 41 | OCTOBER, 1954 | Number 4

BORING DATA FROM GREATER BOSTON

Section 5: West Roxbury and Brookline

PREPARED BY COMMITTEE ON SUBSOILS OF BOSTON

PREVIOUS sections of this series appeared in the JOURNALS of October 1949, October 1950, October 1951 and January 1953.

The tabulations on the following pages cover 214 borings in West Roxbury and 325 borings in Brookline. Locations for 149 of the borings in West Roxbury are shown on Map No. 9E, representing the easterly portion of the area, and the remainder on Map No. 9W, representing the westerly portion. Locations in Brookline are shown on Map No. 13.

Keys to boring notations and notes covering symbols are given on the maps. Column 1 of the tabulation contains two items, the boring number and the coordinates defining the location on the map. Columns 2 and 3 give elevations or depths; elevations always are given with plus or minus signs and refer to Boston City Base whereas depths are used when elevations are not known and it is to be noted that figures without sign always represent depths. In the final column soil types are given.

Since the data represented are from many sources, it is inevitable that the terminology used will not be entirely consistent. It is believed, however, that the various terms used for describing soil types are in reasonable agreement with those most commonly used and with those given in the Building Code of the City of Boston, 1944 edition, page 234. Data are generally presented in these articles in essentially the original form of the records.

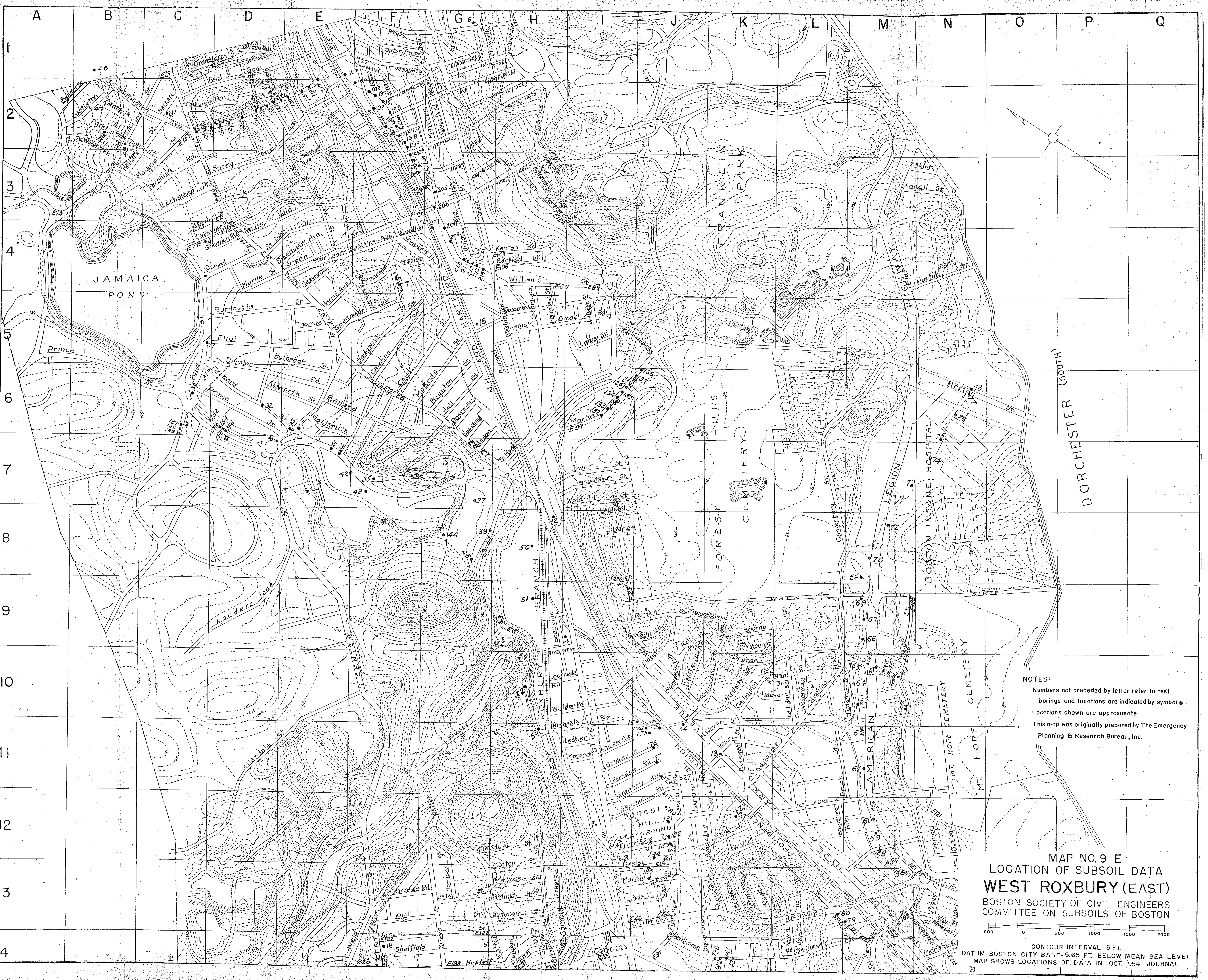
MILES N. CLAIR
IRVING B. CROSBY
CHESTER J. GINDER
LAWRENCE G. ROPES
DONALD W. TAYLOR, *Chairman*

September 3, 1954

Committee on Subsoils of Boston

BORING DATA—WEST ROXBURY (EAST)

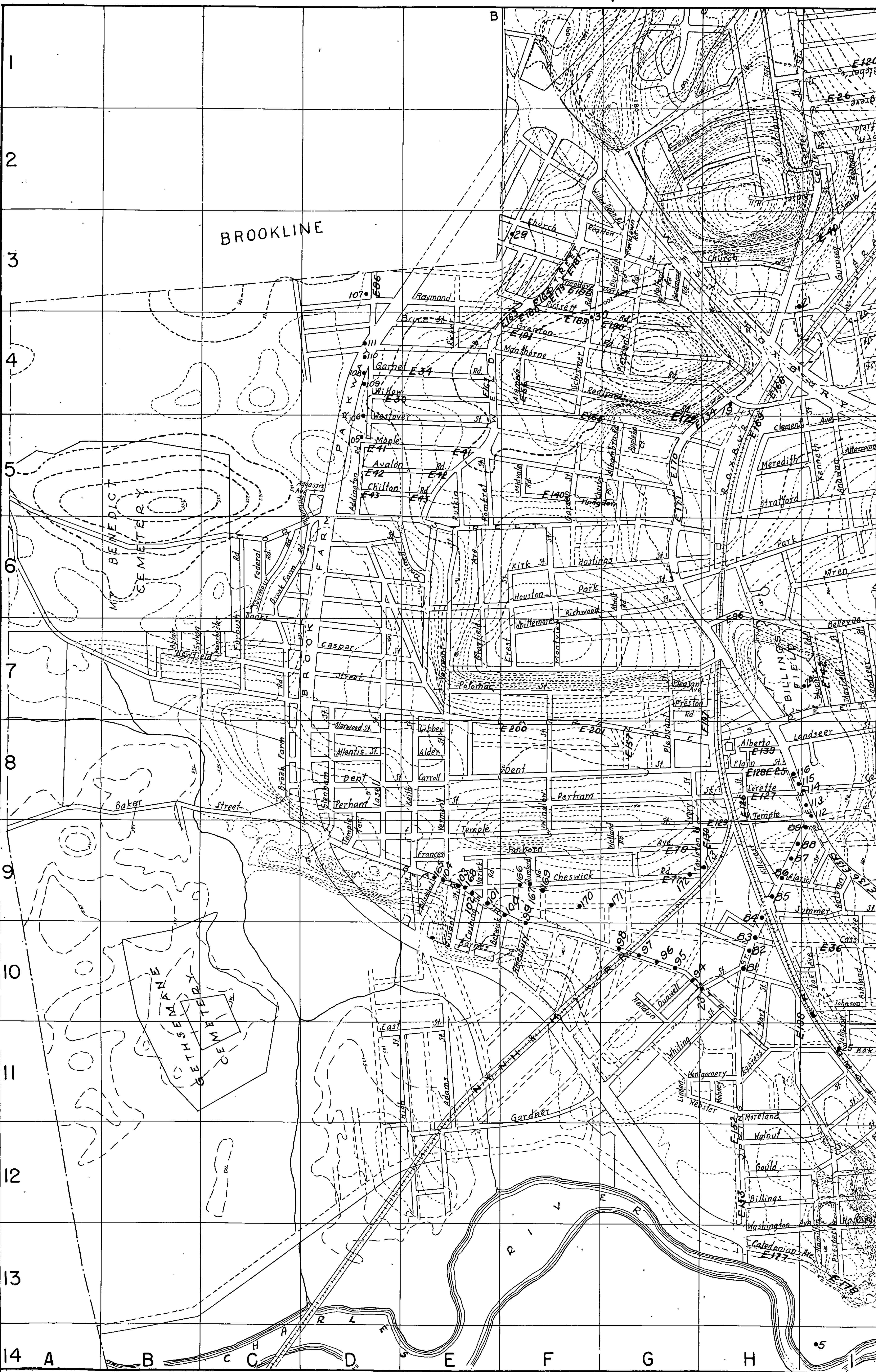
No. and location	Elevations or depths From	To	Formation
1 E6	Surface	8	Fill, gravel, stones
	8	17	Unknown
	17	25	Mud
3 I12	Surface	6	Stones, little clay
	6	18	Clay, stones. Hardpan
4 I9	Surface	6.0	Soft fill
	6.0	8.0	Sand and gravel fill
	8.0	18.5	Mud
	18.5	26.0	Sand, gravel; loose
6 G1	Surface	2	Loam and clay fill
	2	24	Hard clay and stones
7 F4	Surface	2	Loam
	2	19	Sand, gravel, little clay. Struck rock
8 C2	+96	+87	Coarse sand, gravel, to ledge
9 C4	+88	+63	Sand, gravel
	+63	+33	Fine sand
10 C4	+79	+50	Coarse sand, gravel
	+50	+26	Sand
	+26	+23	Sand, gravel
11 C5	+82	+35	Coarse sand, gravel
	+35	+27	Sand
12 H8	+58	+24	Sand, gravel
13 K11	+45	+21	Brown clay, sand, gravel
14 J11	+48	+28	Hard brown clay, sand, gravel
15 J10	+33	+29	Peat
	+29	+26	Coarse sand, gravel
	+26	+ 8	Fine sand
	+ 8	+ 4	Coarse sand
16 G5	Surface	9.5	Sand and gravel fill
	9.5	13.0	Sand fill
	13.0	16.5	Fine sand
	16.5	22.5	Firm sand
	22.5	27.7	Fine sand, clay; firm
	27.7	36.0	Hard compact fine sand, little clay
17 I12	Surface	1.7	Soft loam
	1.7	4.6	Coarse sand, gravel; stiff
	4.6	18.0	Stiff blue clay and fine sand
	18.0	20.0	Coarse sand, gravel; loose
18 F14	Surface	5.2	Cinders fill
	5.2	7.5	Soft peat
	7.5	9.7	Soft fine sand, little clay
	9.7	12.2	Firm fine sand, little clay
	12.2	16.5	Hard sand, gravel, little clay
	16.5	23.0	Hard compact fine sand
22 K12	Surface	6.5	Sand and gravel fill
	6.5	8.5	Sand, gravel, little clay
	8.5	18.0	Hard clay, fine sand
	18.0	20.5	Coarse sand
	20.5	25.5	Hard sand and gravel
	25.5	28.5	Fine loose sand
	28.5	32.5	Medium sand and clay
	32.5	36.2	Hard sand and gravel, little clay
	36.2	39.5	Hardpan—ledge or boulder
27 J11	Surface	6.5	Cinder fill
	6.5	11.3	Soft peat
	11.3	15.2	Coarse sand, gravel, little clay
	15.2	25.0	Fine sand, very little clay
31 C6	+81	+27	Sand, gravel, alternating
32 D6	+75	+ 8	Sand, gravel, alternating
33 E6	+70	+28	Sand, gravel, alternating
34 E7	+58	+20	Sand
	+20	+16	Stiff blue clay, sand
35 F7	+57	+47	Sand, gravel
	+47	+40	Yellow clay, fine sand
	+40	+32	Till (stiff blue clay, sand and gravel)
36 F7	+87	+66	Sand, gravel, clay
	+66	+47	Coarse sand, gravel, clay
	+47	+36	Yellow clay, sand
	+36	+25	Stiff yellow clay, fine sand, gravel
37 G7	+88	+51	Sand, gravel
	+51	+30	Coarse sand, gravel, clay
	+30	+23	Sand, gravel, clay
38 G8	+82	+23	Sand, gravel, alternating
39 C6	+84	+38	Sand, gravel
40 D7	+68	+29	Sand, gravel
41 E7	+58	+20	Sand, gravel
	+20	+16	Clay
42 E7	+55	+45	Sand, gravel
	+45	+22	Mud, hard bottom



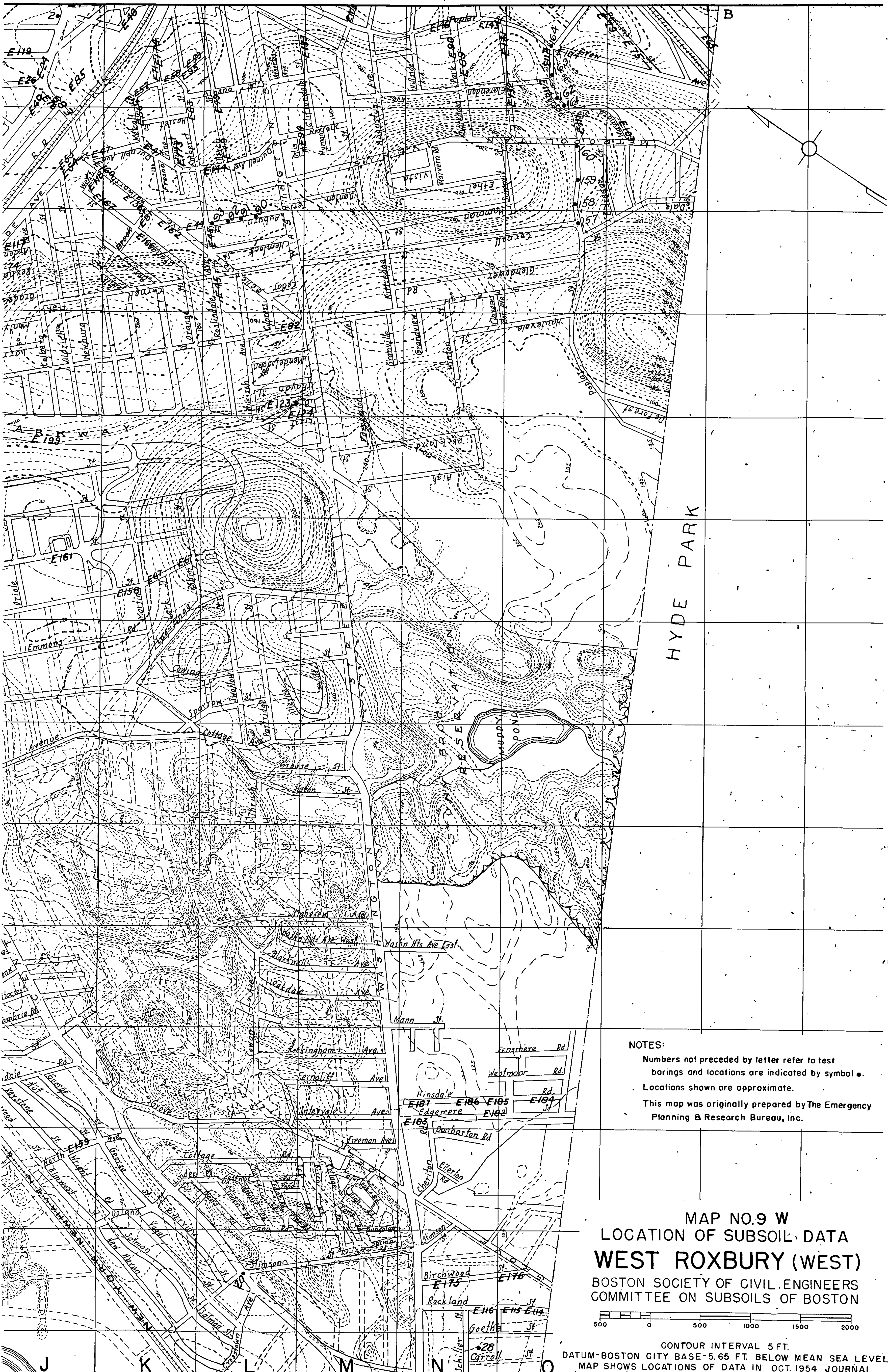
MAP NO. 9 E
LOCATION OF SUBSOIL DATA
WEST ROXBURY (EAST)
BOSTON SOCIETY OF CIVIL ENGINEERS
COMMITTEE ON SUBSOILS OF BOSTON
500 0 500 1000 1500 2000
CONTOUR INTERVAL 5 FT.
DATUM-BOSTON CITY BASE-5.65 FT. BELOW MEAN SEA LEVEL
MAP SHOWS LOCATIONS OF DATA IN OCT. 1954 JOURNAL
NOTES:
Numbers not preceded by letter refer to test borings and locations are indicated by symbol ●
Locations shown are approximate
This map was originally prepared by The Emergency Planning & Research Bureau, Inc.
DORCHESTER (SOUTH)
JAMAICA POND
FRANKLIN PARK
FOREST HILLS CEMETERY
MT. HOPE CEMETERY
BOSTON INSANE HOSPITAL
FOREST HILL PLAYGROUND
AMERICAN LEGION HIGHWAY
HARTFORD AND N.Y.N.H.
WEST ROXBURY BRANCH
WEST ROXBURY PARKWAY
CENTER ST.
HYDE PARK AVE.
WASHINGTON ST.
WALK HILL STREET

No. and location	Elevations or depths From	To	Formation
43	+53	+43	Sand, gravel
F7	+43	+31	Mud, hard bottom
44	+70	+60	Sand, gravel
G8	+60	+48	Sand, gravel, clay
	+48	+38	Sand, gravel
45	+55	+44	Sand, gravel
G8	+44	+35	Sand, gravel, clay
			Rock
46	+75	+68	Sand, gravel
B1			Rock
47	+45	+31	Sand, gravel
B2	+31	+29	Clay, sand, gravel
	+29	+27	Yellow clay
48	+101	+75	Sand, gravel, clay
B2	+75	+51	Sand
	+51	+38	Yellow clay
	+38	+27	Clay, sand, gravel
49	+76	+62	Coarse sand, gravel
B2	+62	+44	Sand
50	+31	−11	Sand
H8			
51	+32	− 8	Mud
H9	− 8	−13	Sand
	−13	−17	Clay
52	+55	+33	Sand, gravel
H10			
53	+44	+30	Coarse gravel
H11	+30	+24	Clay, coarse gravel
54	Surface	3	Rubbish fill
J10	3	5	Soft peat
	5	10	Coarse sand, gravel, little clay
	10	16	Firm fine sand, very little clay
55	Surface	4	Sand and gravel fill
J11	4	10	Firm sand, gravel, little clay
	10	18	Fine sand, very little clay
	18	25	Coarse sand, gravel little clay
56	+39.4	+38.6	Loam
M13	+38.6	+31.9	Coarse sand, gravel
	+31.9	+26.0	Fine sand, gravel
			Boulder
57	+37.6	+35.8	Loam
M12	+35.8	+30.1	Coarse sand, gravel,
	+30.1	+25.6	Sand, gravel
	+25.6	+21.6	Hard sand and gravel
	+21.6	+13.9	Fine sand
	+13.9	+10.6	Sand, gravel
58	+41.7	+41.4	Loam
M12	+41.4	+32.7	Hard sand and gravel
	+32.7	+18.7	Coarse sand, gravel
59	+36.4	+34.0	Peat
M12	+34.0	+29.4	Sand, gravel
	+29.4	+20.9	Fine sand
	+20.9	+18.4	Coarse sand, gravel
60	+40.0	+39.4	Loam
M12	+39.4	+36.4	Very hard sand and gravel
	+36.4	+33.9	Fine sand
	+33.9	+25.0	Coarse sand, gravel
61	+38.2	+35.7	Gravel, stones
M11	+35.7	+23.2	Coarse gravel
62	+37.2	+35.7	Peat
M11	+35.7	+33.2	Fine sand
	+33.2	+21.8	Sand, gravel, stones
63	+37.2	+35.9	Peat
M10	+35.9	+28.7	Sand, gravel
	+28.7	+21.2	Coarse sand, gravel
64	+37.6	+36.4	Peat
M10	+36.4	+34.0	Sand
	+34.0	+30.0	Gravel
	+30.0	+27.0	Fine sand
	+27.0	+23.3	Sand, gravel, stones
65	+38.8	+36.3	Sandy peat
M10	+36.3	+27.7	Sand, gravel, stones
	+27.7	+23.3	Sand, stones
66	+38.1	+35.6	Sandy peat
M9	+35.6	+26.4	Sand, gravel, stones
	+26.4	+23.3	Sand, stones
67	+39.6	+37.2	Sandy peat
M9	+37.2	+27.2	Clay, fine sand, stones
	+27.2	+23.3	Clay, fine sand
68	+40.7	+38.0	Loam
M9	+38.0	+34.0	Blue clay, gravel
	+34.0	+24.7	Clay, sand, stones
69	+41.4	+40.6	Stiff blue clay
M8	+40.6	+38.4	Coarse sand, gravel
	+38.4	+31.0	Sand
	+31.0	+26.5	Fine sand, stones
	+26.5	+21.2	Stiff blue clay, gravel
70	+43.8	+42.4	Loam
M8	+42.4	+37.9	Fine sand
	+37.9	+36.7	Stiff blue clay
	+36.7	+34.8	Yellow clay, sand
	+34.8	+29.3	Clay, gravel, sand
71	+49.2	+47.3	Loam
M8	+47.3	+46.1	Fine sand
	+46.1	+31.1	Sand, gravel
72	+48.2	+45.5	Loam
M8	+45.5	+43.8	Fine sand
	+43.8	+30.0	Clay, small stones
73	+41.6	+39.4	Peat
M7	+39.4	+37.7	Fine sand
	+37.7	+34.2	Sand, gravel
	+34.2	+30.4	Clay
	+30.4	+28.5	Clay, sand, gravel
74	+41.2	+39.2	Peat
N7	+39.2	+34.0	Fine sand
	+34.0	+25.1	Clay, sand, gravel
75	+41.7	+39.0	Peat
N6	+39.0	+37.3	Sand
	+37.3	+27.6	Stiff blue clay

No. and location	Elevations or depths From	To	Formation
76 N6	+42.0	+39.2	Peat
	+39.2	+33.7	Sand
	+33.7	+28.5	Stiff blue clay
77 N6	+42.2	+41.0	Peat
	+41.0	+16.8	Fine sand
	+16.8	+12.8	Fine sand, clay
78 N6	+44.7	+41.0	Fill; sand, gravel, clay
	+41.0	+34.1	Fine sand
	+34.1	+30.6	Blue clay
79 M13	+71.3	+65.5	Sand and gravel fill
	+65.5	+62.5	Fill; peat, sand, gravel
	+62.5	+59.3	Fine sand, gravel, clay
80 L13	+75.3	+70.5	Sand and gravel fill
	+70.5	+65.6	Fine sand, fine gravel
	+65.6	+63.3	Fine sand, coarse gravel
117 E2	+41.8	+41.0	Macadam concrete
	+41.0	+36.0	Fine sand, small stones
	+36.0	+30.0	Clay
118 E2	+49.2	+44.2	Sand, small stones
	+44.2	+38.2	Sand, gravel, small stones
119 E2	+50.5	+43.0	Sand, small stones Rock below
120 E2	+57.0	+50	Sand, gravel
	+50	+46	Fine sand, gravel
121 D2	+59.6	+52.6	Coarse sand, gravel Rock below
122 D2	+64	+59	Fine sand, stones
	+59	+53	Coarse sand, gravel, stones
123 D2	+67.8	+61.3	Coarse sand, gravel Rock below
124 D2	+71	+61	Medium sand, gravel
125 D2	+74.2	+64.2	Medium sand, gravel
126 D2	+81.8	+71.8	Medium sand, gravel
127 D2	+84.2	+73.2	Medium sand, gravel
128 D2	+86.7	+75.7	Medium sand, gravel
129 C2	+89	+78	Medium sand, gravel
130 C2	+90.8	+79.8	Medium sand, gravel
131 C2	+93.3	+86.3	Medium sand, gravel
132 I6	+55	+53	Road bed
	+53	+48	Hardpan
	+48	+45	Coarse sand, gravel
	+45	+39	Fine sand, little clay
133 I6	+54	+52	Road bed
	+52	+47	Coarse sand, gravel
	+47	+40	Sand, gravel, stones (hardpan)
134 I6	+53.0	+51.0	Road bed
	+51.0	+47.0	Coarse sand, gravel
	+47.0	+41.4	Sand, gravel, stone (hardpan) Boulder or ledge below
135 I6	+54.2	+52.2	Road bed
	+52.2	+46.2	Coarse sand, gravel
	+46.2	+40.7	Sand, little clay
136 I6	+55.4	+53.4	Road bed
	+53.4	+47.7	Coarse sand, gravel Boulder or ledge below
137 J6	+57.4	+48.9	Coarse sand, gravel
	+48.9	+44.9	Sand
	+44.9	+41.7	Fine sand, little clay
138 J6	+58.0	+56.0	Road bed
	+56.0	+49.3	Coarse sand, gravel
	+49.3	+46.1	Fine sand
	+46.1	+42.1	Fine sand, little clay
139 K14	+62.5	+56.5	Sand, gravel, loam
	+56.5	+54.5	Peat
	+54.5	+49.5	Sand, gravel
140 K14	+60.8	+56.3	Sand, gravel, loam
	+56.3	+51.3	Peat
	+51.3	+47.3	Sand, gravel, some clay
141 K14	+60	+57	Cinder fill
	+57	+51	Peat
	+51	+46	Fine sand, little clay
	+46	+43	Sand, gravel
142 K14	+61.3	+58.3	Cinder fill
	+58.3	+51.8	Peat
	+51.8	+48.8	Fine sand, little clay
	+48.8	+43.8	Sand, gravel
143 M10	+47.5	+46.9	Road bed
	+46.9	+37.5	Sand
	+37.5	+32.5	Fine sand
144 M10	+46.8	+46.1	Road bed
	+46.1	+35.8	Sand, gravel
	+35.8	+31.8	Fine sand
145 M10	+46.4	+45.8	Road bed
	+45.8	+35.4	Sand, gravel
	+35.4	+31.4	Fine sand
146 M10	+46.2	+45.1	Sand, gravel
	+45.1	+41.1	Fine sand



BROOKLINE
MT BENEDICT CEMETERY
GETHSEMANE CEMETERY
BROOK FARM PARKWAY
CHARLES RIVER
ROXBURY
BILLINGS FIELD
N.Y. N.H. & H. R.R.
Raymond
Bryce St.
Garnet
Willow
Westover St.
Maple
Avalon Rd.
Chilton Rd.
Church
Russett
Grafton
Redlands
Kirk St.
Hastings
Houston
Park
Richwood
Whittemore
Caspar
Street
Potomac St.
Harwood St.
Libbey
Alder
Carroll
Dent
Perham
Temple
Sanborn
Cheswick
Frances
Baker Street
Brook Farm Rd.
East St.
Adams
High
Gardner
Montgomery
Webster
Moreland
Walnut
Gould
Billings
Washington Ave.
Caledonian Ave.
Landseer
Alberta
Elgin
Lorette
Summer
Clement Ave.
Meredith
Stratford
Wren
Bellevue
Preston
Pleasant Ave.
Whiting
Hart
Express
Dunwell
Hellaon
Hodgdon
Mdn Athorne
Ruskin
Pomfret
Federal
Seymour
Banks
Newfield
Sylvan
Manchester
Kenneth
Johnson
A B C D E F G H
1 2 3 4 5 6 7 8 9 10 11 12 13 14


NOTES:
Numbers not preceded by letter refer to test borings and locations are indicated by symbol •.
Locations shown are approximate.
This map was originally prepared by The Emergency Planning & Research Bureau, Inc.
MAP NO. 9 W
LOCATION OF SUBSOIL DATA
WEST ROXBURY (WEST)
BOSTON SOCIETY OF CIVIL ENGINEERS
COMMITTEE ON SUBSOILS OF BOSTON
500 0 500 1000 1500 2000
CONTOUR INTERVAL 5 FT.
DATUM-BOSTON CITY BASE-5.65 FT. BELOW MEAN SEA LEVEL
MAP SHOWS LOCATIONS OF DATA IN OCT. 1954 JOURNAL
HYDE PARK
MUDDY POND
B
J
K
L
M
N
O

No. and location	Elevations or depths From	To	Formation
147	+45.5	+44.8	Road bed
M10	+44.8	+38.5	Sand, gravel
	+38.5	+30.5	Fine sand
148	+43.8	+36.8	Sand, gravel
M10	+36.8	+28.8	Fine sand
149	+85.7	+83.7	Road bed
C6	+83.7	+79.2	Sand and gravel fill
	+79.2	+67.7	Coarse sand, gravel
150	+85.2	+83.7	Road bed
C6	+83.7	+77.7	Sand and gravel fill
	+77.7	+67.7	Coarse sand, gravel
151	+85.0	+83.5	Road bed
C6	+83.5	+77.0	Sand and gravel fill
	+77.0	+67.0	Coarse sand, gravel
152	+77	+74	Road bed
C6	+74	+67	Sand, gravel, mud; mixed
	+67	+57	Sand, gravel
153	+76.4	+75.1	Road bed
D6	+75.1	+71.1	Sand and gravel fill
	+71.1	+63.4	Sand, peat, loam
	+63.4	+56.4	Coarse sand, gravel
154	+76.0	+74.5	Road bed
D6	+74.5	+71.0	Sand and gravel fill
	+71.0	+61.3	Sand, loam, mixed
	+61.3	+56.0	Coarse sand, gravel
155	+75.7	+74.2	Broken stone
D6	+74.2	+64.9	Sand and gravel fill
	+64.9	+57.7	Soft peat
	+57.7	+50.7	Coarse sand, gravel
156	+75.7	+74.2	Broken stone
D6	+74.2	+63.2	Sand and gravel fill
	+63.2	+51.2	Soft peat
	+51.2	+47.2	Coarse sand, gravel
174	+38.5	+35.5	Peat
J11	+35.5	+23.0	Medium sand, fine gravel
175	+39.3	+35.1	Peat
J11	+35.1	+24.1	Medium sand, fine gravel
176	+39	+34	Peat
J11	+34	+24	Medium sand
177	+39.5	+34.5	Peat
J11	+34.5	+24.5	Medium sand
178	+31.4	+21.4	Coarse sand
J11	+21.4	+16.4	Medium sand, pea gravel
179	+32.5	+22.5	Coarse sand
J11	+22.5	+17.5	Coarse sand, pea gravel
180	+33.5	+23.2	Sand, fine gravel
J12	+23.2	+18.2	Medium sand
181	+35.5	+25.5	Sand, fine gravel
J12	+25.5	+22.5	Fine sand, clay
182	+39.5	+34.5	Peat
J12	+34.5	+24.5	Medium sand, gravel
183	+38	+23	Gravel, coarse sand
J12			
184	+40.4	+28.4	Gravel, coarse sand
J12			Rock below
185	+41.6	+35.6	Gravel, coarse sand
J13			Rock below
186	+38.5	+33.5	Gravel
J13	+33.5	+28.5	Sand, clay
			Rock below
187	+21	+20	Concrete
E1	+20	+17	Sand, gravel
	+17	+14	Clay
	+14	7	Blue clay
188	+21	+20	Concrete
F2	+20	+17	Clay, sewage
	+17	+13	Yellow clay
	+13	+ 6	Blue clay
	+ 6	+ 4	Clay, gravel
189	+21	+17	Sand, gravel
F2	+17	+13	Yellow clay, sand
	+13	+ 1	Clay, gravel, sand
190	+21	+16	Sand, gravel
F2	+16	+15	Yellow clay, sand
	+15	+ 5	Gravel, sand, clay
			Rock
191	+21	+13	Coarse sand, gravel
F2	+13	+12	Yellow clay, sand
	+12	+ 3	Gravel, sand, clay
			Rock
192	+34	+27	Gravel, sand, clay
F2			Rock
193	+21	+11	Gravel, sand, clay
F2	+11	+ 5	Yellow clay, sand
	+ 5	− 3	Gravel, sand, clay
194	+22	+18	Gravel
F2	+18	− 2	Gravel, sand, clay
195	+34	+28	Gravel, sand, clay
F2			Rock
196	+34	+26	Gravel, clay
F2			Rock
197	+22	+17	Gravel, sand, clay
F2	+17	+16	Fine sand
	+16	+14	Gravel, sand, clay
	+14	+13	Fine sand, gravel
	+13	+ 1	Gravel, sand, clay
			Rock
198	+22	+15	Gravel, sand, clay
F2	+15	+11	Fine sand, clay
	+11	− 1	Gravel, sand, clay
	− 1	− 2	Fine sand, gravel
	− 2	− 3	Gravel, sand, clay
199	+22	+18	Coarse gravel, sand
F2	+18	+11	Gravel, sand, clay
	+11	+ 5	Fine sand, clay
	+ 5	+ 3	Fine sand
	+ 3	− 3	Gravel, sand, clay
200	+21	+17	Coarse gravel, sand
F3	+17	+ 9	Gravel, sand, clay
	+ 9	+ 8	Sand, gravel
	+ 8	− 1	Gravel, sand, clay

No. and location	Elevations or depths From	To	Formation
201	+21	+17	Coarse sand, gravel
F3	+17	0	Gravel, sand, clay
			Rock
202	+21	+11	Gravel, sand, clay
G3	+11	+ 9	Fine sandy gravel
	+ 9	+ 4	Gravel, sand, clay
			Rock
203	+22	+16	Coarse gravel, clay, sand
F3	+16	+14	Fine sand, very little clay
	+14	+ 7	Gravel, clay
			Rock
204	+22	+16	Gravel, sand, clay
F3	+16	+14	Fine sand, clay
	+14	− 3	Gravel, sand, clay
205	+21	+17	Gravel, sand, clay
G3	+17	+14	Fine sand
	+14	+ 5	Gravel, sand, clay
	+ 5	+ 3	Coarse gravel, sand
			Rock
206	+35	+28	Fill
G3	+28	+20	Fine gravel
	+20	+16	Sand
	+16	+13	Gravel
	+13	− 3	Clay
207	+34	+29	Fill
G4	+29	+27	Peat
	+27	+23	Fine sand
	+23	+16	Coarse gravel
208	+22	+17	Sand, gravel
G4	+17	+ 2	Fine sand, very little clay
209	+22	+15	Gravel, sand, clay
G4	+15	+12	Coarse sand, gravel
	+12	+ 7	Fine sand, clay
	+ 7	− 2	Gravel, sand, clay
210	+21	+16	Gravel, sand, clay
G4	+16	+11	Fine sand, little clay
	+11	+ 9	Sand, gravel
	+ 9	+ 2	Fine sand, clay
	+ 2	− 9	Fine sand, gravel
211	+21	+16	Gravel, sand, clay
G4	+16	+11	Fine sand, little clay
	+11	+ 7	Fine sand, gravel
	+ 7	+ 4	Gravel
			Rock
212	+21	+16	Gravel, sand, clay
G4	+16	+12	Fine sand, little clay
	+12	+ 8	Fine sand, gravel
			Rock
213	+21	+16	Gravel, sand, clay
G4	+16	+ 7	Fine sand, little clay
	+ 7	+ 3	Coarse sand, gravel
			Rock
214	+21	+16	Gravel, sand, clay
G4	+16	+14	Coarse sand
	+14	+ 7	Fine sand, little clay
	+ 7	− 5	Coarse sand, gravel

Boring Data—West Roxbury (West)

No. and location	Elevations or depths From	To	Formation
2	Surface	2	Hard gravel, stones
J1	2	5	Sand, gravel
	5	20	Stones, gravel
5	Surface	40	Mud
I14	40	46	Soft sand and clay
	46	53	Gravel
19	Surface	8.6	Fill; sand, little gravel, rubbish
H4	8.6	12.3	Loamy sand
	12.3	21.0	Hard sand, very little fine gravel
20	Surface	6	Hard coarse gravel and sand fill
L13	6	7	Loamy sand
	7	11	Firm fine sand
21	Surface	7	Loam, sand
H3	7	20	Hard packed sand and gravel
	20	26.2	Fine compact sand and clay
			Struck rock
23	Surface	1.5	Cinder fill
H10	1.5	4.5	Medium sand, gravel
	4.5	22.7	Fine sand
	22.7	26.6	Loose sharp sand
	26.6	37.0	Fine sand, little clay
	37.0	41.5	Firm sand
	41.5	45.0	Firm sand, very little clay
24	Surface	5.0	Soft loamy sand fill
I7	5.0	12.5	Hard fine sand and gravel
	12.5	18.0	Stiff fine sand
25	Surface	5.5	Hard coarse gravel and sand fill
I11	5.5	12.5	Fine soft sand
	12.5	17.5	Loose soft sharp sand
26	Surface	2.0	Sand and gravel fill
I11	2.0	4.5	Sharp sand, little clay
	4.5	10.0	Hard coarse gravel, sand, little clay
	10.0	12.0	Firm fine sand, little clay
	12.0	17.0	Firm sharp sand


MAP NO. 13
LOCATION OF SUBSOIL DATA
BROOKLINE
BOSTON SOCIETY OF CIVIL ENGINEERS
COMMITTEE ON SUBSOILS OF BOSTON
500 0 500 1000 1500 2000
CONTOUR INTERVAL 5 FT.
DATUM- BOSTON CITY BASE-5.65 FT. BELOW MEAN SEA LEVEL
MAP SHOWS LOCATION OF DATA IN OCT. 1954 JOURNAL
NOTES:
Numbers not preceded by letter refer to test borings and locations are indicated by symbol ●
Numbers preceded by letter W refer to borings drilled into bedrock and locations are indicated by symbol ○.
Locations shown are approximate.
This map was originally prepared by The Emergency Planning & Research Bureau, Inc.
A B C D E F G H I J K L M N O P Q R S T U V W X
JAMAICA POND
THE COUNTRY CLUB
BEACON ST.
COMMONWEALTH
BOYLSTON
BROOKLINE AV.
NEWTON STREET
WEST ROXBURY PARKWAY
INDEPENDENCE DRIVE
CLINTON ROAD
FAIRWAY
PLAYGROUND

No. and location	Elevations or depths From	To	Formation
28 N14	Surface	1.3	Loam
	1.3	14.0	Hard sand and gravel. Hardpan
29 F3	Surface	3.2	Sand, gravel
30 F4	Surface	3	Sand, gravel
	3	12	Hard sand, gravel and little clay
81 H10	+124.0	+122.4	Crushed stone, gravel, sand
	+122.4	+120.4	Sand, gravel
	+120.4	+109.4	Coarse sand
	+109.4	+104.0	Fine sand
82 H10	+125.8	+124.2	Crushed stone, gravel, sand
	+124.2	+122.2	Coarse sand, gravel
	+122.2	+107.8	Coarse sand
	+107.8	+102.8	Coarse sand, gravel
83 H10	+122.1	+120.8	Crushed stone, gravel, sand
	+120.8	+118.8	Coarse sand, gravel
	+118.8	+110.0	Coarse sand
	+110.0	+103.1	Fine sand
84 H9	+115.5	+113.6	Crushed stone, gravel, sand
	+113.6	+112.0	Coarse sand, gravel
	+112.0	+103.2	Fine sand
85 H9	+118.0	+116.0	Crushed stone, gravel, sand
	+116.0	+114.1	Coarse sand, gravel
	+114.1	+109.6	Sand, clay
	+109.6	+105.3	Sand, gravel
86 H9	+126.3	+125.3	Crushed stone, gravel, sand
	+125.3	+123.3	Fine sand, gravel
	+123.3	+117.3	Fine sand
	+117.3	+113.5	Coarse gravel, sand
	+113.5	+105.3	Sand
87 H9	+134.0	+132.5	Crushed stone, gravel, sand
	+132.5	+126.8	Fine sand
	+126.8	+124.5	Coarse gravel, sand
	+124.5	+108.0	Sand
88 H9	+135.2	+133.2	Crushed stone, gravel, sand
	+133.2	+125.9	Coarse sand, gravel
	+125.9	+114.7	Coarse sand
	+114.7	+107.4	Fine sand
89 I9	+134.8	+133.3	Crushed stone, gravel, sand
	+133.3	+128.6	Coarse sand, gravel
	+128.6	+116.4	Coarse sand
	+116.4	+109.1	Fine sand
90 L3	+110.7	+109.4	Fill, sand, gravel, peat
	+109.4	+103.5	Sand, clay
	+103.5	+ 96.7	Coarse sand, gravel, clay
91 L3	+108.8	+105.6	Fill; sand, gravel, peat, ashes
	+105.6	+ 96.8	Fine sand, gravel, clay

No. and location	Elevations or depths From	To	Formation
92 L3	+108.0	+106.3	Fill; sand, gravel, ashes
	+106.3	+104.5	Stiff black peat
	+104.5	+ 99.2	Fine sand, stones, clay
	+ 99.2	+ 95.0	Loose sand and gravel
93 L3	+108.0	+105.9	Sand and gravel fill
	+105.9	+105.1	Black peat, fine sand
	+105.1	+ 99.4	Fine sand, coarse gravel
	+ 99.4	+ 95.0	Coarse sand, gravel, soft blue clay
94 G10	+121.4	+120.0	Sand and gravel fill
	+120.0	+106.4	Very fine sand
95 G10	+122.8	+119.2	Fill; sand, gravel, loam
	+119.2	+117.0	Coarse sand, gravel
	+117.0	+107.8	Sharp sand, fine gravel
96 G10	+123.0	+121.6	Sand and gravel fill
	+121.6	+120.8	Sandy loam
	+120.8	+118.4	Coarse sand, gravel
	+118.4	+108.0	Very fine sand
97 G10	+124.5	+120.8	Sandy loam
	+120.8	+118.8	Coarse sand, gravel
	+118.8	+110.3	Very fine sand
	+110.3	+ 67.5	Very fine sand (quicksand)
98 G10	+124.8	+121.5	Sandy loam
	+121.5	+118.9	Coarse sand, gravel
	+118.9	+ 73.0	Very fine sand (quicksand)
99 F10	+130.8	+127.6	Sand and gravel fill
	+127.6	+115.8	Fine sand, fine gravel
100 F9	+129.5	+127.7	Sand and gravel fill
	+127.7	+114.5	Very fine sand
101 E9	+126.0	+125.0	Fill; sand, fine gravel
	+125.0	+111.0	Very fine sand
102 E9	+122.8	+119.8	Fill; sand, gravel, loam
	+119.8	+107.8	Yellow clay, gravel, sand
103 E9	+123.0	+118.4	Fill; sand, gravel, loam
	+118.4	+114.3	Very fine sand
	+114.3	+108.0	Coarse sand, gravel, little clay
104 E9	+127.4	+125.3	Loam, sand, gravel
	+125.3	+112.4	Coarse sand, gravel
105 D5	+148.3	+145.9	Sand, loam
	+145.9	+141.1	Coarse sand, gravel, clay Boulder below
106 D5	+149.7	+147.7	Sandy loam, stones
	+147.7	+143.9	Fine sand, stones
	+143.9	+134.7	Yellow clay, gravel, sand

No. and location	Elevations or depths From	To	Formation
107	+147.4	+144.2	Stiff brown peat
D3	+144.2	+132.4	Sand, fine gravel, little clay
108	+148.0	+126.8	Stiff brown peat
D4	+126.8	+125.6	Very fine sand (quicksand) Boulder below
109	+147.2	+142.3	Stiff brown peat
D4	+142.3	+140.3	Sand, gravel
110	+147.5	+139.2	Stiff brown peat
D4	+139.2	+132.5	Fine sand, loose gravel, some clay
111	+147.5	+138.8	Stiff brown peat
D4	+138.8	+137.5	Fine sand
	+137.5	+132.5	Coarse gravel, fine sand; loose
112	+134.7	+133.2	Road bed
I8	+133.2	+128.2	Coarse sand, gravel
	+128.2	+122.2	Sand, gravel
	+122.2	+114.0	Fine sand
113 I8	+135.4	+114.5	Sand, gravel
114	+136.4	+129.8	Sand, gravel
I8	+129.8	+124.8	Fine sand, gravel
	+124.8	+115.8	Fine sand
115	+136.7	+129.7	Coarse sand, gravel
H8	+129.7	+118.7	Fine sand
116	+138.2	+130.2	Sand, gravel
H8	+130.2	+117.2	Fine sand, gravel
157	+149.2	+140.8	Coarse sand, gravel
O3	+140.8	+139.8	Fine sand
	+139.8	+133.8	Coarse sand, gravel
158	+147.2	+146.6	Road bed
O2	+146.6	+141.0	Sand, gravel
	+141.0	+132.2	Peat, little sand
159	+153.4	+152.8	Road bed
O2	+152.8	+148.0	Sand, gravel, stones Ledge below
160	+144.0	+143.4	Road bed
O2	+143.4	+138.4	Sand, gravel
	+138.4	+132.4	Coarse sand, gravel
	+132.4	+130.4	Stone, sand, clay Ledge below
161	+116.2	+115.6	Road bed
O1	+115.6	+109.6	Coarse sand, gravel
	+109.6	+104.0	Sand, gravel, stones Ledge below
162	+108.0	+107.4	Road bed
O1	+107.4	+103.0	Sand, loam, gravel
	+103.0	+ 93.0	Sand, gravel, stones
163	+ 91.7	+ 91.1	Road bed
O1	+ 91.1	+ 81.1	Coarse sand, gravel
	+ 81.1	+ 77.5	Sand, gravel, stones
164	+ 78.6	+ 77.8	Road bed
O1	+ 77.8	+ 71.8	Sand, gravel
	+ 71.8	+ 66.4	Coarse sand, gravel
	+ 66.4	+ 62.8	Gravel, sand, stones
165 E9	+126.5	+106.5	Coarse sand, gravel; very hard
166 F9	+123.5	+116.5	Coarse sand, gravel Ledge below
167	+123.5	+113.5	Sand, gravel, clay
F9	+113.5	+109.5	Sand, gravel; very hard
168	+123.5	+110.5	Coarse sand, gravel
E9	+110.5	+105.0	Coarse sand, gravel; very hard
169	+122.8	+115.3	Loose sand
F9	+115.3	+105.3	Coarse sand, gravel; very hard
	+105.3	+104.7	Sand, gravel, clay
170	+119.0	+117.0	Soft loam
F9	+117.0	+109.5	Loose fine sand
	+109.5	+104.0	Coarse sand
171	+118.2	+116.2	Soft loam
G9	+116.2	+111.7	Sand, gravel; hard
	+111.7	+108.2	Gravel, clay; hard
172 G9	+123.8	+118.8	Fill; sand, gravel, stones
	+118.8	+116.8	Sand, gravel
	+116.8	+111.8	Gravel
	+111.8	+109.8	Fine sand, little gravel
173 H9	+124.5	+118.0	Fill; sand, gravel, loam
	+118.0	+115.0	Gravel, little sand
	+115.0	+113.0	Fine sand, clay
	+113.0	+109.5	Gravel, clay

Boring Data—Brookline

No. and location	Elevations or depths From	To	Formation
1 P8	+55.0	+52.5	Fill
			Struck rock
2 P8	+51.5	+50.5	Fill
	+50.5	+44.8	Hardpan
			Struck rock
3 P8	+53.0	+50.2	Fill
	+50.2	+46.0	Hardpan
			Struck rock
4 P8	+48.0	+43.5	Fill
	+43.5	+34.5	Hardpan
			Struck rock
5 P8	+51.0	+45.8	Fill
	+45.8	+35.0	Hardpan
			Struck rock
6 P8	+49.7	+45.2	Fill
	+45.2	+40.5	Hardpan
			Struck rock
7 P9	Surface	2.0	Loam
	2.0	16.4	Compact fine sand
	16.4	21.0	Loose sand and gravel
8 P10	Surface	1	Loam
	1	11	Loose sand and gravel
	11	24	Hard packed sand and gravel
9 P10	+15.5	+14.5	Loam
	+14.5	+ 5.5	Loose sand and gravel
	+ 5.5	− 4.5	Medium blue clay
10 P10	+11.8	+11.2	Loam
	+11.2	+ 5.8	Loose sand and gravel
	+ 5.8	− 6.5	Medium blue clay
11 P8	+34.9	+26.9	Fill
	+26.9	+18.7	Hard packed sand and gravel
	+18.7	+ 4.4	Compact fine sand, very little clay
12 P8	+37.8	+32.8	Fill
	+32.8	+27.8	Compact fine sand
	+27.8	+ 8.3	Compact fine sand, very little clay
13 P8	+39.8	+36.8	Fill
	+36.8	+31.8	Hard packed sand and gravel
	+31.8	+ 9.8	Compact fine sand, very little clay
14 P8	+39.8	+34.3	Fill
	+34.3	+19.8	Hard packed sand and gravel, very little clay
	+19.8	+14.8	Hard packed coarse sand
15 P8	+39.4	+28.8	Fill
	+28.8	+17.4	Compact coarse sand
16 P8	+38.4	+27.7	Fill
	+27.7	+24.5	Peat mud
	+24.5	+18.3	Hard packed coarse sand
17 P8	+38.2	+31.0	Fill
	+31.0	+22.7	Peat mud
	+22.7	+16.8	Hard packed coarse sand
	+16.8	+13.0	Compact fine sand
18 Q8	+38.8	+29.0	Fill
	+29.0	+26.4	Peat mud
	+26.4	+18.6	Compact coarse sand
	+18.6	+13.8	Compact fine sand
19 Q8	+40.0	+30.2	Fill
	+30.2	+29.9	Peat mud
	+29.9	+21.0	Compact coarse sand
	+21.0	+15.0	Compact fine sand
20 Q7	+43.5	+38.0	Fill
	+38.0	+32.2	Hard packed sand and gravel
	+32.2	+25.6	Hard packed coarse sand
	+25.6	+18.4	Compact fine sand
21 P7	+47.3	+42.3	Fill
	+42.3	+35.3	Hard packed sand and gravel
	+35.3	+31.0	Compact fine sand
	+31.0	+22.3	Compact fine sand, little clay
22 Q7	+45.3	+39.3	Fill
	+39.3	+27.5	Hard packed sand and gravel
	+27.5	+25.8	Compact fine sand, clay
	+25.8	+20.3	Hard packed coarse sand
23 S9	+18.6	+14.6	Fill
	+14.6	+ 6.6	Hard packed coarse sand
	+ 6.6	− 0.4	Hard packed sand and gravel
	− 0.4	− 6.4	Medium yellow clay
24 S9	+15.3	+ 8.3	Fill
	+ 8.3	− 1.7	Medium yellow clay
	− 1.7	−19.7	Medium blue clay
25 S9	+14.3	+ 3.3	Sand and gravel fill
	+ 3.3	− 0.7	Fairly compact sand
	− 0.7	− 6.7	Medium yellow clay, layers of sand
	− 6.7	−15.7	Medium blue clay, layers of sand
	−15.7	−25.7	Medium blue clay
26 S9	+12.7	+ 3.7	Sand, coarse fill
	+ 3.7	− 5.3	Peat, mud
	− 5.3	−12.3	Mud and fine sand mixed
	−12.3	−17.8	Loose medium sand
	−17.8	−35.3	Compact fine sand and gravel
	−35.3	−41.3	Hard packed sand and gravel

No. and location	Elevations or depths From	To	Formation
27 S9	+12.5	+ 3.5	Sand, ashes, fill
	+ 3.5	- 5.0	Peat, mud
	- 5.0	-17.5	Loose medium sand
	-17.5	-27.5	Peat, mud, sand, mixed
	-27.5	-33.5	Peat, mud
	-33.5	-40.5	Medium blue clay
28 S9	+11.1	+ 3.1	Fill
	+ 3.1	- 5.9	Peet, mud
	- 5.9	-17.9	Loose fine sand, little mud
	-17.9	-47.9	Mud and fine sand mixed
	-47.9	-52.9	Hard packed sand and gravel
29 S9	+13.0	+ 4.0	Ashes, fill
	+ 4.0	- 4.5	Peat, mud
	- 4.5	-13.7	Loose medium sand
	-13.7	-21.7	Peat, mud
	-21.7	-34.7	Fine silty sand, mud
	-34.7	-39.7	Compact coarse sand
30 S9	+14.3	+ 3.8	Sand, ashes, fill
	+ 3.8	- 5.7	Peat, mud
	- 5.7	-10.7	Loose fine sand
	-10.7	-17.7	Compact medium sand
	-17.7	-28.7	Compact sand and gravel
	-28.7	-38.7	Compact fine sand
31 T9	+14.8	+ 5.8	Ashes, fill
	+ 5.8	- 1.2	Peat mud
	- 1.2	- 4.2	Fairly compact fine sand
	- 4.2	-23.2	Fairly compact fine sand, little clay
32 T8	+14.8	+ 9.8	Fill
	+ 9.8	- 6.2	Compact fine sand, traces of clay
	- 6.2	-15.2	Medium blue clay, little fine sand
	-15.2	-25.2	Medium blue clay
33 T8	+14.4	+ 8.4	Sand and clay fill
	+ 8.4	-15.6	Compact fine sand, very little clay
	-15.6	-25.6	Medium blue clay, layers of fine sand
34 T8	+18.2	+ 6.2	Fill
	+ 6.2	+ 3.2	Peat mud
	+ 3.2	-21.8	Medium blue clay, layers of fine sand
35 T8	+13.6	+ 7.6	Fill
	+ 7.6	-21.4	Compact fine sand, very little clay
	-21.4	-31.4	Medium blue clay
36 T8	+12.6	+ 1.1	Sand fill, peat mud
	+ 1.1	-26.4	Compact fine sand, very little clay
	-26.4	-32.4	Medium blue clay
37 T8	+12.4	+ 8.4	Fill
	+ 8.4	- 1.6	Peat mud
	- 1.6	-28.6	Compact fine sand, traces of clay
	-28.6	-33.6	Medium blue clay
38 T8	+12.4	+ 5.4	Fill.
	+ 5.4	- 2.1	Fine silty sand, mud
	- 2.1	- 8.6	Compact sand and gravel
	- 8.6	-24.6	Compact fine sand, little clay
	-24.6	-31.6	Medium blue clay
39 T8	+14.2	+ 8.2	Fill
	+ 8.2	- 4.3	Peat mud
	- 4.3	- 7.8	Compact medium sand
	- 7.8	-23.8	Compact fine sand, traces of clay
	-23.8	-30.8	Medium blue clay
40 T8	+13.3	+ 7.3	Fill
	+ 7.8	- 4.7	Peat mud
	- 4.7	-29.7	Medium blue clay, layers of fine sand
41 U8	+14.0	+ 6.0	Fill
	+ 6.0	- 4.0	Peat mud
	- 4.0	- 7.8	Compact coarse sand
	- 7.8	-26.0	Medium blue clay, layers of fine sand
42 T7	+ 8.7	- 9.6	Peat mud
	- 9.6	-13.3	Fine compact sand, little clay
	-13.3	-29.3	Medium blue clay
	-29.3	-32.8	Fine compact sand, little clay
43 T7	+ 8.6	+ 6.6	Fill
	+ 6.6	-13.4	Peat mud
	-13.4	-18.4	Soft blue clay
	-18.4	-35.4	Medium blue clay
44 T7	+ 8.0	+ 2.0	Fill, mud
	+ 2.0	-14.8	Peat mud
	-14.8	-19.6	Soft blue clay
	-19.6	-33.5	Medium blue clay, layers of fine sand
45 T7	+10.0	+ 2.0	Fill
	+ 2.0	-17.2	Peat mud
	-17.2	-25.2	Very soft blue clay
	-25.2	-35.5	Fine compact sand and clay
46 T8	+20.4	+14.4	Fill
	+14.4	+ 5.4	Peat mud
	+ 5.4	- 7.6	Soft blue clay
	- 7.6	-30.6	Medium clay, fine clay
	-30.6	-35.6	Coarse sand and gravel, hard packed
47 T8	+18.8	+ 7.8	Fill
	+ 7.8	+ 1.8	Peat mud
	+ 1.8	- 8.2	Fine sand and clay, soft
	- 8.2	-20.2	Fine sand and clay, compact
	-20.2	-31.2	Hard blue clay

No. and location	Elevations or depths From	To	Formation
48 T8	+17.8	+ 6.4	Fill
	+ 6.4	− 1.7	Peat mud
	− 1.7	−20.2	Medium clay, fine sand
	−20.2	−22.2	Coarse sand and gravel, compact
	−22.2	−32.2	Hard blue clay
49 T7	+26	+14	Fill
	+14	+ 2	Medium blue clay, little fine sand
50 T7	+16.8	+14.8	Loam
	+14.8	+ 4.8	Fill
	+ 4.8	−10.2	Peat mud
	−10.2	−17.2	Soft blue clay, little fine sand
	−17.2	−45.2	Hard blue clay
51 T7	+18.8	+ 6.8	Fill
	+ 6.8	− 1.2	Peat mud
	− 1.2	− 7.2	Sand, little clay
	− 7.2	−20.2	Hard blue clay
52 T8	+18.8	+16.8	Loam
	+16.8	+ 2.8	Sand and gravel, possibly fill
	+ 2.8	− 9.2	Coarse sand and gravel, compact
53 T8	+22.2	+18.2	Fill
	+18.2	+13.2	Hard packed sand, gravel and clay
	+13.2	+ 7.2	Medium clay, fine sand
	+ 7.2	− 7.8	Hard packed sand and gravel
54 T8	+22.5	+20.5	Loam
	+20.5	+ 9.5	Clay
	+ 9.5	+ 0.5	Fine sand and clay, soft
	+ 0.5	− 8.5	Hard blue clay
55 S8	+20	+19	Street material
	+19	+ 3	Loose sand, gravel fill
	+ 3	−13	Sand and gravel, compact
56 S8	+20.7	+ 7.2	Loose sand, gravel and ashes fill
	+ 7.2	− 4.0	Sand, little clay fill
	− 4.0	−11.1	Peat mud
	−11.1	−31.8	Compact coarse sand, little clay
57 S8	+19.3	− 8.7	Fill
	− 8.7	−28.7	Peat mud
	−28.7	−45.7	Loose fine sand, little clay
	−45.7	−55.7	Medium blue clay, fine sand
58 S8	+19.5	+ 4.5	Fill
	+ 4.5	− 0.5	Peat mud
	− 0.5	−22.2	Hand packed sand and gravel
59 S8	+20.4	+ 8.4	Fill
	+ 8.4	−34.6	Hard packed sand and gravel

No. and location	Elevations or depths From	To	Formation
60 T8	+14.5	+12.5	Fine sand, clay
	+12.5	− 9.5	Sand and gravel, compact
	− 9.5	−20.0	Fine compact sand
	−20.0	−31.5	Medium yellow clay
61 T8	+14.0	+11.0	Loam
	+11.0	−12.0	Loose fine sand, little clay
	−12.0	−26.5	Clay with fine sand
62 T8	+13.6	+10.6	Loam, fill
	+10.6	−17.4	Loose fine sand, layers of clay
	−17.4	−40.9	Medium clay, fine sand
63 T8	+15.2	+ 5.7	Fill
	+ 5.7	− 3.8	Loose fine sand
	− 3.8	−17.8	Fine sand, clay
	−17.8	−32.8	Medium blue clay, little fine sand
64 T9	+15.5	+ 9.5	Fill
	+ 9.5	−17.0	Loose fine sand, layers of clay
	−17.0	−29.5	Medium blue clay
65 T9	+16	+ 8	Fill
	+ 8	−17	Loose fine sand, layers of clay
	−17	−29	Medium clay
66 T9	+13.5	+ 8.5	Fill
	+ 8.5	+ 2.5	Peat mud
	+ 2.5	−20.8	Loose fine sand, layers of clay
	−20.8	−31.5	Medium blue clay
67 T9	+10.0	+ 2.0	Mud
	+ 2.0	−17.5	Loose fine sand, layers of clay
	−17.5	−28.0	Clay with sand
68 S6	Surface	2.0	Loamy sand
	2.0	12.5	Hard sand, coarse gravel, hardpan
69 S5	+64	+54	Coarse sand and gravel
	+54	+43	Coarse sand, gravel, little clay
	+43	+37	Stiff yellow clay
	+37	+31	Coarse gravel, sand, little clay
	+31	+28	Sand
70 R5	+61	+41	Coarse sand and gravel
	+41	+26	Hard clay and gravel
71 Q6	+57	+25	Coarse sand and gravel, little clay
72 Q6	+53	+38	Coarse sand and gravel
	+38	+33	Coarse sand, gravel, clay
	+33	+26	Hard clay, little gravel
	+26		Rock — supposed boulder

No. and location	Elevations or depths From	To	Formation
73 P6	+59	+29	Hard clay and gravel to rotten rock
74 Q6	+54	+31	Sand, gravel
75 P6	+51	+26	Sand, gravel
	+26	+25	Stiff blue clay, little gravel
76 O7	+48	+43	Loam, sand, gravel
	+43	+28	Fine sand
77 O7	+49	+31	Sand, gravel
	+31	+29	Sand, gravel and little clay to rock
78 O7	+47	+41	Clay, fine gravel, sand
	+41	+28	Coarse gravel, fine sand, little clay
	+28	+26	Blue clay
79 O7	+52	+32	Coarse sand and gravel
80 O7	+55	+46	Loam, sand, gravel
81 O8	+60	+47	Sand, gravel
	+47	+39	Blue clay, sand and gravel to rock
82 O8	+58	+52	Clay, sand, gravel
	+52	+45	Sand, coarse gravel
	+45	+38	Coarse sand and gravel, some clay to rock
83 O8	+59	+37	Sand, gravel, little clay to rock
84 O8	+61	+46	Coarse sand and gravel, little clay
	+46	+35	Yellow clay, gravel
	+35	+32	Sand, gravel and clay to rock
85 O8	+67	+57	Fine sand and gravel to rock
86 O8	+62	+44	Coarse sand and gravel, little clay to rock
87 O8	+68	+65	Sand and gravel to rock
88 O8	+62	+54	Sand, gravel with little clay to rock
89 O8	+62	+46	Sand and gravel to rock
90 O8	+67	+57	Sand and gravel to rock
91 O8	+65	+51	Coarse gravel and sand to rock
92 N9	+60	+54	Coarse sand
	+54	+41	Sand, gravel, clay
	+41	+36	Sand to rock
93 N9	+57	+40	Sand and gravel to rock
94 N9	+56	+30	Sand and gravel to rock
95 N9	+57	+20	Sand and gravel to rock
96 N9	+59	+46	Sand and gravel to rock
97 N10	+53	+28	Sand, gravel
98 N10	+48	+28	Sand, gravel
99 S9	Surface	12	Fill, hard
	12	20	Mud, sand
	20	25	Loose sand
	25	35	Sand, clay
100 S7	+31.0	+ 7.5	Good fill
	+ 7.5	+ 4.5	Peat, wood
	− 4.5	−14.0	Hard coarse sand and gravel
101 S7	+28.0	+ 3.8	Good fill
	+ 3.8	−14.0	Peat, wood
	−14.0	−22.0	Hard coarse sand and gravel
102 U8	Surface +13	−132	Overburden. Ends in clay and sand
103 T8	Surface	17	Fill
	17	32	Fine sand
	32	37	Hard clay
104 W8	Surface	5	Mud
	5	9	Gravel
	9	18	Fine sand mixed with little clay
	18	48	Soft clay
105 W7	Surface	9	Fill
	9	14	Gravel, sand
	14	26	Hard clay
106 U7	Surface	9	Sand, stones, little clay
	9	30	Hard packed sand
107 U7	+25.4	+11.4	Sand and gravel fill
	+11.4	+ 6.9	Sand, gravel, clay
	+ 6.9	− 4.6	Compact sand and gravel
108 U7	+23.5	+ 1.0	Sand and gravel fill
	+ 1.0	− 7.8	Peat
	− 7.8	−13.3	Hard packed sand, gravel and stones
109 U7	+22.4	− 0.6	Sand fill
	− 0.6	− 7.3	Peat
	− 7.3	−13.1	Compact sand, gravel and stones
110 U7	+21.0	− 3.5	Sand and gravel fill
	− 3.5	−15.8	Peat, mud, thin layers of gravel

No. and location	Elevations or depths From	To	Formation
	−15.8	−30.5	Compact sand with some gravel, little clay
	−30.5	−33.5	Hard packed sand, gravel and stones
111 U7	+20.4	− 2.6	Sand and gravel fill
	− 2.6	−25.1	Peat, thin layers of gravel
	−25.1	−33.0	Medium yellow clay, some sand and gravel. Struck rock
112 U7	+20.4	+ 1.4	Sand and gravel fill
	+ 1.4	− 5.6	Peat, old wood
	− 5.6	−12.1	Medium clay, layers of sand and gravel
	−12.1	−17.3	Hard packed sand, gravel, stones
113 V7	+19.5	+ 0.3	Sand and gravel fill
	+ 0.3	− 9.7	Peat
	− 9.7	−16.2	Medium blue clay, sand, gravel
	−16.2	−23.2	Hard blue clay, sand, gravel
114 V7	+19.8	+ 2.2	Sand and gravel fill
	+ 2.2	− 8.1	Peat
	− 8.1	−15.7	Hard yellow clay, sand, gravel
	−15.7	−20.2	Hard packed sand, gravel and stones
115 V7	+20.4	− 1.1	Sand and gravel fill
	− 1.1	− 9.4	Peat
	− 9.4	−16.6	Medium yellow clay, sand, gravel
	−16.6	−24.6	Hard yellow clay, some sand and gravel
116 V7	+21.0	+ 2.0	Sand and gravel fill
	+ 2.0	− 8.5	Peat
	− 8.5	−19.8	Medium blue clay, sand, gravel
	−19.8	−23.5	Hard packed sand, gravel and stones
117 V7	+21.5	+ 6.5	Sand and gravel fill
	+ 6.5	− 1.0	Peat
	− 1.0	−10.0	Hard blue clay, sand, gravel
	−10.0	−13.5	Hard packed sand and gravel
118 V7	+23.0	+17.5	Sand and gravel fill
	+17.5	+10.5	Hard yellow clay, sand, gravel
	+10.5	+ 6.5	Hard packed sand, gravel and stones
119 V7	+14.8	− 6.2	Fill
	− 6.2	−18.4	Wood, mud
	−18.4	−26.9	Medium blue clay, little fine sand and gravel
	−26.9	−28.2	Hard blue clay, little sand and gravel
120 V7	+13.5	− 8.8	Coarse sand and gravel fill

No. and location	Elevations or depths From	To	Formation
	− 8.8	−19.0	Peat
	−19.0	−36.5	Medium blue clay, little sand and gravel
121 V7	+13.2	− 9.5	Fill
	− 9.5	−11.8	Mud, wood
	−11.8	−24.8	Mud
	−24.8	−27.8	Peat
	−27.8	−34.8	Medium blue clay, little sand
122 V7	+14.3	− 6.9	Fill
	− 6.9	−10.7	Mud
	−10.7	−15.7	Mud, wood
	−15.7	−26.7	Peat
	−26.7	−33.1	Medium blue clay, little fine sand and gravel
123 V7	+19.8	+ 4.3	Fill
	+ 4.3	− 5.7	Peat mud
	− 5.7	− 9.7	Clay, sand, gravel, stones
124 V7	+21.5	+ 3.5	Fill
	+ 3.5	− 6.2	Peat mud
	− 6.2	− 7.5	Sand, gravel, clay (struck rock)
125 V7	+21.8	+ 3.8	Fill
	+ 3.8	− 6.0	Peat mud
	− 6.0	− 8.8	Clay, sand, gravel (struck rock)
126 U6	Surface	15.0	Fill, hard
	15.0	21.5	Peat mud
	21.5	23.5	Fine loose sand
	23.5	30.0	Clay, stones
127 U7	Surface	18	Fill, hard
	18	32	Peat mud
	32	33	Coarse sand
	33	42	Clay, stones
128 U6	Surface	11	Sand fill
	11	29	Peat mud
	29	34	Clay
	34	37	Gravel
129 U6	+16.2	+13.0	Sand and gravel fill
	+13.0	+ 8.7	Mud
	+ 8.7	+ 4.1	Peat
	+ 4.1	+ 1.7	Peaty sand and gravel
	+ 1.7	− 5.8	Hard clay, sand and gravel
130 U6	+16.7	+12.9	Sand and gravel fill
	+12.9	+ 5.4	Coarse gravel, hard sand
	+ 5.4	− 7.3	Coarse loose sand, gravel
131 T6	+15.3	+ 9.6	Sand and gravel fill
	+ 9.6	+ 8.5	Black mud
	+ 8.5	− 9.7	Coarse sand, gravel
132 U6	+21.0	+18.8	Sand and gravel fill
	+18.8	+ 7.4	Firm coarse sand and gravel
	+ 7.4	+ 5.3	Sand, little clay
	+ 5.3	− 7.0	Coarse loose sand, gravel

No. and location	Elevations or depths From	To	Formation
133 U6	+19.4	+11.6	Fill; coarse loose sand, gravel
	+11.6	+ 8.4	Fine sand, clay
	+ 8.4	— 8.6	Coarse loose sand, gravel
134 U6	+17.0	+10.7	Sand and gravel fill
	+10.7	+ 6.1	Soft peat
	+ 6.1	+ 5.8	Peaty sand and gravel
	+ 5.8	+ 1.9	Clay, sand, gravel
	+ 1.9	—12.9	Coarse loose sand, gravel
135 U6	+19.8	+11.0	Fill; coarse sand, gravel
	+11.0	+ 6.7	Soft mud
	+ 6.7	+ 5.2	Peaty sand and gravel
	+ 5.2	+ 2.8	Blue clay, sand
	+ 2.8	—10.2	Coarse loose sand, gravel
136 U6	+17.9	+ 9.6	Sand and gravel fill
	+ 9.6	+ 0.4	Peat
	+ 0.4	— 2.9	Coarse loose sand, gravel
	— 2.9	— 5.4	Blue clay
	— 5.4	—27.1	Coarse sand, gravel (large stones)
137 U6	+15.1	+10.9	Sand and gravel fill
	+10.9	+ 8.1	Wood, peat
	+ 8.1	— 4.7	Peat
	— 4.7	—14.9	Coarse sand, gravel
138 U6	+17.3	+ 5.3	Sand and gravel fill
	+ 5.3	— 0.7	Peat, wood
	— 0.7	— 5.7	Coarse sand, gravel
	— 5.7	—10.7	Coarse gravel, hard clay
139 V7	+17.8	+11.8	Sand and gravel fill
	+11.8	+ 1.8	Peat mud
	+ 1.8	— 6.2	Compact coarse sand
140 V7	+17.8	+ 8.2	Fill
	+ 8.2	+ 4.4	Peat mud
	+ 4.4	— 8.9	Compact medium sand
141 V7	+19.0	+10.0	Fill
	+10.0	+ 7.0	Hard packed sand and gravel
142 V7	+18.2	+ 8.2	Fill
	+ 8.2	+ 3.2	Peat mud
	+ 3.2	—11.8	Compact coarse sand
143 V7	+19.0	+ 8.0	Sand and clay fill
	+ 8.0	+ 4.8	Peat mud
	+ 4.8	— 3.8	Compact coarse sand
144 V7	+20.6	+10.6	Sand and clay fill
	+10.6	+ 9.6	Peat mud
	+ 9.6	+ 5.6	Fine sand and clay
	+ 5.6	+ 2.6	Compact coarse sand
145 W7	+23.8	+12.6	Fill; sand, clay, stones
	+12.6	+ 8.4	Sand, gravel, clay, hard
146 U6	Surface	5	Fill
	5	20	Peat mud
	20	25	Sand and gravel, hard
147 V6	Surface	28	Fill
	28	33	Mud
	33	39	Fine loose sand
	39	42	Gravel
148 U5	Surface	16	Fill
	16	21	Fine loose sand
	21	24	Hard packed sand and gravel
149 U5	Surface	7	Sand, fill
	7	16	Hard packed gravel and stones
150 U4	Surface	16	Loose clay, apparently fill
	16	25	Clay, fine sand
	25	30	Hard clay, stones
151 V5	Surface	24	Fill
	24	31	Peat mud
	31	36	Sand
152 W5	+22.0	+ 4.0	Fill
	+ 4.0	— 8.0	Fine silty sand, mud
	— 8.0	—14.5	Compact sand, gravel, and stones
	—14.5	—24.5	Compact sand and gravel
153 W5	+22.2	+ 0.2	Fill
	+ 0.2	—32.8	Fine silty sand, mud
	—32.8	—40.3	Compact sand, gravel and stones
154 W5	+23.0	— 1.5	Fill
	— 1.5	—43.0	Fine silty sand, mud
	—43.0	—52.0	Compact sand, gravel and stones
155 W5	+24.1	— 2.9	Fill
	+ 2.9	—31.6	Fine silty sand, mud
	—31.6	—42.4	Compact sand, gravel and stones
156 W5	+25.5	+ 2.0	Fill
	+ 2.0	—14.5	Fine silty sand, mud
	—14.5	—23.5	Compact sand, gravel and stones
157 V5	+16.0	+13.0	Leaves and ash fill
	+13.0	+ 7.5	Fill, mud
	+ 7.5	+ 1.4	Hard packed gravel
	+ 1.4	—15.3	Fine compact sand, little clay
158 W5	+20	+ 5	Fill
	+ 5	0	Mud
	0	— 6	Fine sand
	— 6	—11	Clay, sand, stones

No. and location	Elevations or depths From	To	Formation
159 W5	+20.7	+ 2.7	Fill
	+ 2.7	− 7.3	Mud
	− 7.3	−16.3	Clay, sand, stones
160 W5	+18.5	+ 3.5	Fill
	+ 3.5	−12.5	Mud
	−12.5	−19.5	Clay, sand, stones
161 W5	Surface	11.8	Fill
	11.8	29.0	Mud
	29.0	38.0	Sand, gravel, little clay
162 V5	Surface	14.2	Fill, mud
	14.2	20.4	Mud
	20.4	24.8	Fine compact sand
	24.8	28.0	Yellow clay, sand, stones
163 V5	Surface	1.0	Loam
	1.0	5.8	Compact sand
	5.8	18.0	Clay, sand, stones
164 V5	+19	+ 6	Fill
	+ 6	− 9	Mud
	− 9	−13	Medium sand, traces of mud
	−13	−16	Hard yellow clay, stones
165 V5	+19.0	+ 6.5	Fill
	+ 6.5	−12.0	Mud
	−12.0	−14.0	Medium sand
	−14.0	−20.0	Clay, sand, stones
166 V5	+16.0	+ 5.5	Fill
	+ 5.5	+ 1.2	Peat mud
	+ 1.2	−19.0	Fine compact sand
167 V5	+23.5	+17.5	Fill
	+17.5	+11.5	Sand and gravel, loose
	+11.5	+ 1.5	Medium sand
	+ 1.5	− 1.5	Hard yellow clay and sand
168 V5	+23	+15	Fill
	+15	+ 3	Medium sand
	+ 3	− 2	Hard packed sand and gravel
169 W5	+19.0	+10.0	Fill
	+10.0	−40.0	Mud
	−40.0	−42.8	Hard packed sand and gravel
170 W5	+17	+10	Fill
	+10	−46	Mud
	−46	−50	Hard packed sand and gravel
171 W5	+16.5	+ 8.5	Fill
	+ 8.5	−37.0	Mud
	−37.0	−43.5	Hard packed sand and gravel
172 W5	+16.5	+ 3.5	Fill
	+ 3.5	−21.0	Mud
	−21.0	−27.5	Hard packed sand and gravel
173 W5	+18	+ 9	Fill
	+ 9	−57	Mud
	−57	−60	Hard packed sand and gravel
174 W5	+17.5	+ 9.0	Fill
	+ 9.0	−61.5	Mud
	−61.5	−66.5	Hard packed sand and gravel
175 W5	+17.5	+ 8.5	Fill
	+ 8.5	−45.5	Mud
	−45.5	−52.5	Hard packed sand and gravel
176 W5	+17	+ 8	Fill
	+ 8	−21	Mud
	−21	−31	Hard packed sand and gravel
177 W5	+18	+ 6	Fill
	+ 6	−60	Mud
	−60	−64	Hard packed sand and gravel
178 W5	+17	+10	Fill
	+10	−54	Mud
	−54	−58	Hard packed sand and gravel
179 W5	+17	+10	Fill
	+10	−48	Mud
	−48	−52	Hard packed sand and gravel
180 W5	+17	+ 9	Fill
	+ 9	−40	Mud
	−40	−52	Hard packed sand and gravel
181 W5	Surface	22	Fine hard packed sand
	22	51	Fine sand, little clay
182 V3	Surface	3	Mud
	3	6	Clay, stones
	6	8	Fine sand
	8	14	Hard packed sand, gravel and stones
183 V3	Surface	20	Hard packed dry sand, gravel and stones
184 N3	Surface	14	Fill
	14	20	Sand, gravel, stones. Hardpan
185 T3	Surface	19.6	Fill
	19.6	36.0	Fine silty sand
	36.0	45.2	Fine sand easily penetrated
	45.2	50.0	Hard packed sand and gravel
186 T3	Surface	20.0	Fill
	20.0	25.8	Peat mud
	25.8	27.8	Fine sand easily penetrated
	27.8	28.6	Sand and gravel hardpacked (Rock obstruction)
187 T3	Surface	20.6	Fill
	20.6	37.4	Peat mud
	37.4	51.7	Fine sand easily penetrated
	51.7	56.3	Hard packed sand and gravel

No. and location	Elevations or depths From	To	Formation
188 T3	Surface	19.8	Fill
	19.8	33.7	Peat mud
	33.7	51.2	Fine sand easily penetrated
	51.2	57.3	Hard packed sand and gravel
189 T3	Surface	20.0	Fill
	20.0	28.6	Peat mud
	28.6	34.0	Fine sand easily penetrated
	34.0	42.5	Hard packed sand and gravel
190 T3	Surface	19.7	Fill
	19.7	37.5	Peat mud
	37.5	47.7	Fine sand easily penetrated
	47.7	55.0	Hard packed sand and gravel
191 U3	Surface	12.5	Sand and gravel fill
	12.5	23.5	Soft peat
	23.5	30.0	Silty sand
	30.0	38.5	Fine sand
	38.5	42.0	Fine loose sand, little clay
	42.0	48.0	Coarse sand and gravel
192 U3	Surface	15.5	Sand and gravel fill
	15.5	30.5	Soft peat, wood
	30.5	35.5	Silty sand
	35.5	43.0	Loose silty sand
	43.0	50.0	Coarse sand and gravel
193 U3	Surface	8.5	Fill
	8.5	12.5	Soft peat
	12.5	18.5	Silty sand
	18.5	32.5	Medium loose sand
	32.5	41.0	Fine sand, little clay
	41.0	48.0	Sand, gravel, little clay
194 T3	Surface	12.5	Fill
	12.5	35.5	Soft peat
	35.5	43.0	Silty sand
	43.0	45.5	Fine sand
	45.5	52.0	Coarse sand and gravel
195 T3	Surface	15.5	Fill
	15.5	27.0	Soft peat
	27.0	39.5	Soft silty sand
	39.5	47.5	Fine sand, very little clay
	47.5	54.0	Medium sand, little fine gravel
196 U3	Surface	15.5	Fill
	15.5	25.5	Silty sand
	25.5	38.0	Medium sand
	38.0	48.0	Medium loose sand, little fine gravel
197 R3	+58	+34	Sand, gravel
198 R3	+47	+39	Sand, gravel
	+39	+26	Peat
	+26	+21	Stiff blue clay, coarse gravel
199 R3	+47	+36	Coarse sand and gravel
	+36	+30	Peat
	+30	+26	Stiff blue clay, coarse gravel
200 R2	+52	+49	Sand, gravel
	+49	+37	Sand, gravel, yellow clay
201 L9	+84.5	+83.5	Loam
	+83.5	+73.5	Hard packed sand, gravel and stones
202 L9	+83.0	+79.0	Loam, sand
	+79.0	+78.6	Mud
	+78.6	+75.8	Sand and clay, hard
	+75.8	+73.0	Compact coarse sand
203 L9	+83.8	+81.3	Loam, sand
	+81.3	+73.8	Sand, gravel, clay, stones, hard
204 L9	+83.8	+81.3	Loam, sand
	+81.3	+75.8	Hard packed sand and gravel
	+75.8	+73.8	Compact coarse sand
205 L9	+84.5	+82.0	Loam, sand
	+82.0	+74.5	Sand, gravel, clay, hard
206 L9	+88.3	+84.7	Loam, sand
	+84.7	+78.3	Compact sand and clay
207 L9	+89.5	+88.0	Loam, sand
	+88.0	+77.2	Hard packed sand and gravel, little clay
208 L10	+86.3	+85.3	Loam
	+85.3	+80.3	Hard packed sand and gravel
	+80.3	+78.3	Compact fine sand and clay
209 L10	+90.2	+89.2	Loam
	+89.2	+82.5	Hard packed sand and gravel (struck rock)
210 L10	+91.0	+89.5	Loam, sand
	+89.5	+84.0	Hard packed sand and gravel
	+84.0	+82.0	Sand, gravel, clay, stones, hard (struck rock)
211 L10	+112.2	+111.2	Loam
	+111.2	+102.5	Sand, gravel, clay, stones, hard (struck rock)
212 C9	+182.0	+179.0	Fill
	+179.0	+172.5	Peat mud
	+172.5	+166.0	Very soft blue clay,
			Very soft blue clay, layers of fine sand
	+166.0	+164.5	Hard packed sand, gravel and stones (struck rock)

No. and location	Elevations or depths From	To	Formation
213 D9	+181.8	+179.8	Peat mud
	+179.8	+177.0	Sand, gravel, stones, clay (struck rock)
214 D9	+182.3	+164.8	Peat mud
	+164.8	+157.1	Compact sand, gravel and stones
215 D9	+183.0	+163.0	Peat mud
	+163.0	+159.7	Hard packed sand, gravel and stones (struck rock)
216 D9	+182.5	+164.2	Peat
	+164.2	+161.0	Hard packed sand, gravel and stones (struck rock)
217 C9	+181	+173	Mud
	+173	+168	Sand, gravel, clay
	+168	+166	Hard packed sand and stones (struck rock)
218 C9	+181.0	+174.5	Peat mud
	+174.5	+172.2	Soft blue clay (struck rock)
219 C9	+181.3	+179.3	Loam, mud
	+179.3	+170.5	Medium yellow clay, some sand, gravel and stones (struck rock)
220 C9	+180.6	+175.6	Loam, mud
	+175.6	+171.6	Medium sand, some clay and gravel
	+171.6	+165.4	Hard packed sand, gravel, stones and clay
221 A5	+158	+152	Fill
	+152	+150	Peat mud
	+150	+140	Hard packed sand and gravel, some clay
222 B5	+156.9	+151.4	Fill
	+151.4	+147.4	Peat mud
	+147.4	+138.9	Hard packed sand and gravel, some clay
223 B5	+158.8	+151.8	Fill
	+151.8	+146.6	Peat mud
	+146.6	+136.8	Hard packed sand and gravel, some clay (rock obstruction)
224 B5	+157.1	+151.6	Fill
	+151.6	+148.1	Peat mud
	+148.1	+141.1	Hard packed sand, gravel and clay
225 B5	+157.4	+149.4	Fill
	+149.4	+142.4	Hard packed sand, gravel and clay
226 E3	Surface	7.2	Peat mud
	7.2	10.6	Loose fine sand, little clay
	10.6	16.2	Hard packed sand and gravel
227 G3	+168.8	+165.1	Peat, mud
	+165.1	+159.0	Compact fine sand
	+159.0	+138.7	Compact sand and clay (struck clay)
228 G3	+168.3	+155.1	Peat mud
	+155.1	+143.3	Hard packed sand, gravel, stones
230 R9	+19	+19	Fill
231 R9	+17.0	+14.5	Fill
	+14.5	+ 5.0	Loam, peat
	+ 5.0	0	Sand
232 R9	+19.5	+11.5	Fill
	+11.5	+ 9.0	Soft mud
	+ 9.0	0	Blue gravel
233 R9	+19.5	+12.0	Fill
	+12.0	+11.0	Soft mud
	+11.0	+ 8.0	Gravel
234 R10	+18.5	+ 3.5	Mud
	+ 3.5	0	Fine sand, clay
235 S8	+22.0	+ 4.5	Peat
236 S8	+22	− 4	Peat
237 S8	+22	− 8	Peat
238 S8	+22	− 3	Peat
239 S8	+22	− 9	Peat
240 S8	+22	0	Peat
241 S8	+22	− 4	Peat
242 S9	+11	− 2	Mud
243 S9	+11	−11	Mud
244 S8	+11	−17	Mud
245 S8	+11	+ 3	Mud
246 S9	+18.5	+ 8.0	Fill
	+ 8.0	+ 3.0	Blue and yellow clay
	+ 3.0	− 2.0	Peat
	− 2.0	− 7.5	Clay, sand
247 S9	+19.5	+11.5	Fill
	+11.5	+ 6.5	Yellow clay
	+ 6.5	− 8.5	Clay, sand
248 S9	+17.0	− 4.5	Peat
	− 4.5	−15.0	Clay, sand

No. and location	Elevations or depths From	To	Formation
249 S9	+19.0	— 4.5	Peat
250 S9	+19.0	— 4.0	Peat
251 S9	+18.0	— 2	Fill
	— 2	— 5	Peat
	— 5	—10	Clay, sand
252 S9	+17	+ 8	Fill
	+ 8	+ 5	Blue and yellow clay
	+ 5	— 5	Peat
	— 5	—10	Sand, clay
253 S9	+16	+ 6	Fill
	+ 6	+ 4	Sand
	+ 4	— 1	Peat
	— 1	— 5	Sand, clay
	— 5	—11.5	Sand
254 S9	+17	+11	Fill
	+11	+ 8	Clay
	+ 8	+ 4	Clay, gravel
	+ 4	— 4	Sand
255 S9	+19	+ 7	Peat
256 S9	+19.0	— 1.5	Peat
257 S9	+19	+ 6	Peat
258 S9	+19	— 2.5	Peat
259 S9	+19	+ 4	Peat
260 S9	+19	— 4	Peat
261 S9	+19	— 3.5	Peat
262 S9	+19	— 2	Peat
263 S9	+26	+ 3	Mud
	+ 3	0	Fine sand, clay
264 S7	+24	+ 7	Peat
265 S7	+24	+ 6	Peat
266 T7	+30.0	+21.0	Loam, fine sand
	+21.0	+14.5	Coarse sand and gravel
	+14.5	+13.5	Clay
	+13.5	+10.0	Fine sand
	+10.0	0	Fine sand, clay
267 T7	+31.0	+26.0	Sandy loam
	+26.0	+21.0	Coarse sand
	+21.0	+18.5	Fine sand, gravel
	+18.5	+ 8.0	Clay
	+ 8.0	+ 5.0	Fine sand, clay
268 T7	+30.5	+20.0	Sandy loam
	+20.0	+16.0	Fine sand
	+16.0	+11.0	Sand
	+11.0	+10.0	Black mud
	+10.0	+ 1.0	Clay
269 U8	+14.5	+ 7.0	Sand, gravel
	+ 7.0	+ 4.0	Peat
	+ 4.0	— 2.5	Peat, sand
	— 2.5	— 5.0	Sand, gravel
	— 5.0	—22.0	Sand, clay
	—22.0	—26.0	Clay
	—26.0	—30.0	Sand, clay
270 U8	+14.5	+ 7.0	Fill
	+ 7.0	— 3.0	Sand, peat, loam
	— 3.0	—10.0	Sand
	—10.0	—16.0	Sand, clay
	—16.0	—20.0	Sand, clay
271 U8	+13.5	+10.0	Sand, gravel
	+10.0	— 1.0	Fill
	— 1.0	— 3.0	Gravel, peat, sand
	— 3.0	— 8.5	Peat, sand
	— 8.5	—10.0	Sand, clay
	—10.0	—17.0	Sand
	—17.0	—22.0	Sand, clay
272 U8	+16.0	+ 5.0	Fill
	+ 5.0	+ 3.0	Sand
	+ 3.0	—14.0	Sand, clay
	—14.0	—15.5	Clay, gravel
	—15.5	—22.0	Sand
	—22.0	—24.0	Sand, clay
284 T6	+29	+10	Hardpan
285 T6	+25	+11	Fill
286 T6	+21.0	— 7.5	Fill
	— 7.5	—16.0	Peat
	—16.0	—20.0	Sand
	—20.0	—23.0	Coarse sand
	—23.0	—31.5	Sand, clay, hard
	—31.5	—36.0	Yellow clay, hardpan, sand
287 T6	+21.5	+ 3.0	Fill
	+ 3.0	—21.0	Peat
	—21.0	—29.0	Fine sand, little clay
	—29.0	—33.0	Coarse sand
288 T6	+21.5	+ 1.0	Fill
	+ 1.0	— 8.0	Gravel
	— 8.0	—14.5	Peat
	—14.5	—48.5	Sand, clay
289 T6	+21.5	+ 1.0	Fill
	+ 1.0	—14.0	Bedrock
290 T6	+15.0	— 7.5	Fill
	— 7.5	—14.0	Gravel
	—14.0	—26.0	Sand
291 T6	+15	0	Fill
	0	— 6	Clay
	— 6	—10	Sand
	—10	—15	Gravel

No. and location	Elevations or depths From	To	Formation
292 U6	+15	+ 2	Fill
	+ 2	− 6	Sand
	− 6	−10	Fine sand
293 U6	+20	+ 8	Fill
	+ 8	0	Clay, sand
294 T6	+22	+13	Gravel
	+13	+11	Peat
	+11	0	Gravel, clay
	0	− 3	Hard gravel
295 T6	+21.0	+10.5	Fill
	+10.5	+ 2.5	Sand, gravel
	+ 2.5	− 5.0	Gravel
296 T6	+19.0	−16.0	Peat
	−16.0	−19.5	Fine sand
	−19.5	−22.0	Hard fine sand
	−22.0	−27.5	Hard granite sand
297 T6	+19	−13	Gravel, hard gravel
298 T6	+19	+ 1	Peat
	+ 1	−13	Hard gravel
299 T6	+18	+ 5	Fill
	+ 5	−14	Hard gravel
	−14	−20	Sand, gravel
300 U6	+17.0	+ 1.5	Fill
	+ 1.5	− 1.5	Peat
	− 1.5	− 3.5	Coarse gravel
	− 3.5	− 5.5	Sand
	− 5.5	−11.5	Coarse gravel
301 U6	+20.0	+14.5	Fill
	+14.5	+ 1.0	Coarse gravel
	+ 1.0	0	Sand, gravel
302 U6	+18.5	+12.0	Fill
	+12.0	+ 8.0	Peat
	+ 8.0	+ 3.0	Gravel, sand
	+ 3.0	− 3.5	Fine sand, clay
303 U6	+20.0	+ 2.5	Mud
304 U6	+19	− 2	Mud
305 U7	+16.0	+ 3.0	Fill
	+ 3.0	− 6.0	Peat
	− 6.0	−11.0	Sandy clay
	−11.0	−13.5	Fine sand
	−13.5	−16.0	Sand, gravel
306 U7	+18.5	+ 3.0	Fill
	+ 3.0	− 4.5	Sand, gravel
307 U7	+16.0	+ 4.0	Fill
	+ 4.0	− 5.0	Peat
	− 5.0	− 7.5	Sand
308 U7	+16.0	+ 2.5	Fill
	+ 2.5	− 9.0	Peat
	− 9.0	−12.0	Fine sand, clay
	−12.0	−18.0	Clay, sand
309 U7	+16.5	+ 8.5	Fill
	+ 8.5	− 1.0	Peat
	− 1.0	− 6.0	Sand
	− 6.0	−10.0	Gravel
310 U7	+15.5	− 3.0	Peat
311 U7	+15.5	+ 3.5	Peat
312 U7	+15.5	+ 3.0	Peat
313 V7	+15.5	+ 7.0	Peat
314 V7	+15.5	+ 3.0	Peat
315 V7	+15.5	− 2.0	Peat
316 V7	+15.5	− 8.0	Peat
317 V7	+15.5	− 6.0	Peat
318 V7	+15.5	− 5.0	Peat
319 V7	+15.5	+ 9.0	Fill
	+ 9.0	+ 4.5	Fine sand
	+ 4.5	−10.5	Wood, sand
	−10.5	−16.0	Peat, clay
	−16.0	−31.5	Clay, sand
	−31.5	−36.0	Sharp sand, fine
	−36.0	−86.0	Sharp sand, coarse
	−86.0	−91.5	Sharp fine sand, little clay
	−91.5	−97.5	Fine sand
320 V7	+20.0	+10.0	Fill
	+10.0	+ 1.0	Fine sand
	+ 1.0	− 1.5	Wood
	− 1.5	− 9.5	Peat
	− 9.5	−19.5	Sand, clay, gravel
	−19.5	−23.5	Clay, gravel
	−23.5	−28.0	Hard gravel
321 V7	+20.0	+12.0	Fill
	+12.0	0	Sand
	0	−11.5	Peat
	−11.5	−27.0	Soft sand and clay
	−27.0	−32.0	Gravel
322 V7	+16	0	Fill
	0	−13	Peat
	−13	−20	Sand, clay
	−20	−25	Gravel
	−25	−28	Hard clay
323 V7	+15.5	+ 3.0	Fill
	+ 3.0	− 8.0	Peat, wood
	− 8.0	−20.0	Fine sharp sand, little clay
	−20.0	−26.5	Clay, sand
	−26.5	−42.5	Coarse sand, clay
	−42.5	−49.0	Coarse sand, gravel clay
	−49.0	−70.0	Coarse sand
	−70.0	−80.5	Fine sand
	−80.5	−100.5	Fine sand, little clay
	−100.5	−106.0	Fine sand, much clay, hard

No. and location	Elevations or depths From	To	Formation
324	+20	− 7	Fill
V7	− 7	−19	Peat
	−19	−27	Sand, clay
	−27	−31	Hard, gravel
325	+15	− 4	Fill
V7	− 4	−15	Peat
	−15	−26	Fine sand
	−26	−29.5	Hard gravel
326	+15.0	− 3.5	Fine soft sand
V7	− 3.5	−17.0	Peat
	−17.0	−25.0	Fine sand
	−25.0	−27.5	Hard gravel
327	+20.0	+ 4.0	Fill
V7	+ 4.0	−11.5	Peat
	−11.5	−24.5	Sand, clay
	−24.5	−28.0	Hard gravel

No. and location	Elevations or depths From	To	Formation
328	+45	+39	Fill
T4	+39	+21	Peat
	+21	+19	Hardpan
329	+45	+36	Fill
T4	+36	+22	Peat
	+22	+20	Hardpan
330	+40	+ 3	Soft silty peat
S2	+ 3	0	Gravel
331	+40	+13	Soft silty peat
S2	+13	+ 9	Sand and gravel, hard
332	+16.7	+10.4	Fine sand and gravel fill
U6	+10.4	− 3.3	Coarse loose sand and gravel

WELLS

No. and location	Elevations or depths From	To	Formation
W-2	+30	−35	Sand—no rock
Q9			
W3	+10	−95	Overburden
R9	−95	−154	Bedrock
W-4	+48	−83	Overburden.
Q8	−83	−180	Bedrock

No. and location	Elevations or depths From	To	Formation
W-5	+85	−29	Overburden
O6	−29	−100	Bedrock
W-6	+95	+15	Overburden
N5	+15	−85	Bedrock

SHELL STRUCTURES

By A. L. Delaney,* Member

(Presented at a meeting of the Structural Section, BSCE, held on January 13, 1954.)

The first reinforced concrete shell roof in the United States was built in 1933 at the Century of Progress Exposition in Chicago. In the twenty years that have elapsed since then over 15,000,000 sq. ft. of shell construction has been erected on this continent. With this in mind, I think we are justified in saying that the practicability and economy of this type of construction is past the experimental stage.

But large as this 15,000,000 sq. ft. may seem at first glance, it might not appear so great compared to what it should be when it is realized that only two of the many outstanding engineering firms in this country are responsible for the design of practically all of this shell construction. This being so, it makes us wonder whether or not an important structural method—suitable in long-span roofs for industrial buildings, hangars, auditoriums, gymnasiums, exhibition halls, markets, sports arenas, bus garages and the like—is being overlooked simply because of the lack of general design knowledge. Apparently others thought so, and in an attempt to remedy this condition, the ASCE set up a committee—about six years ago—to develop a manual on the design of shell structures. This past year the committee issued a report—Manual #31 entitled "Design of Cylindrical Shell Roofs".

My purpose is to try and explain the behavior of barrel shells and compare its action with other structural elements in common use by the average designer. If I can arouse your interest to the point of investigating the use of the manual for the design of simple barrel shells, I shall have achieved my purpose.

Almost all of the past literature on shell structures deals exclusively with the mathematical aspects of its design while neglecting to explain the geometrical and physical relationships involved. As a result, the simplicity with which shells function has been obscured.

The average structural engineer with his ability to handle the design of continuous frames, his knowledge of statics and the common

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flexural theories can understand the behavior of a shell. With the aid of this manual he can design these shells if he will but allow his mind to be a little more flexible in considering the problem. For instance, in designing a one-way slab the first thing we do, subconsciously, is to consider a strip one foot wide and then treat it as we would a beam problem.

Some engineers, when thinking of shells, tend to do the same and immediately begin to wonder how a 3″ or 3½″ thick element, one foot wide, can span distances of 30, 40 or 50 ft. or more. Yet, if they were designing a ribbed floor, they would never think of an element of the 2″ top as spanning the same distance as the rib. They immediately consider the tee-beam effect of the topping and rib acting together. Or in the case of channel plank, they consider the section modulus of the entire section. So it should be with shells. Instead of considering a foot wide element, let us consider the transverse section as a whole. We have T-shape beams, so why not arc shape beams.

Let us consider an arc shape beam supported by and integral with transverse stiffeners. We can compute the bending moment at the various points along the span just as we would in conventional beam analysis. If we assume, for the time being, that the ordinary theory of flexure applies, we can determine the stress at any point in the cross section by applying the regular flexure formula:

$$f = \frac{Mc}{I}$$

and for a section at mid-span we would find that the compressive stresses at the crown and the tensile stresses at the bottom edges are not of great magnitude.

This is not exactly the manner in which the shells behave, but for certain long-span shells the deviation is not too great. Before we start to examine the stresses in a shell, let us consider the structure as a whole and map out the procedure we wish to follow.

If we consider the shell acting as an ordinary beam between the supports (fig. 1A) it must deflect downward, and if a section parallel to the supports is cut at mid-span; any and all points in that section must deflect the same amount for the ordinary beam theory to be applicable. That is, point "a" at the crown should deflect the same amount as point "b" at the free edge. But in an arc shape beam this cannot be so, without the use of special diaphragms, because in the transverse direction we will have an additional deflection downward and inward at the same time as shown in Section A-A. Point b′ is the position b would tend to take under the beam theory and point b″ is the position b would tend to take with the additional transverse

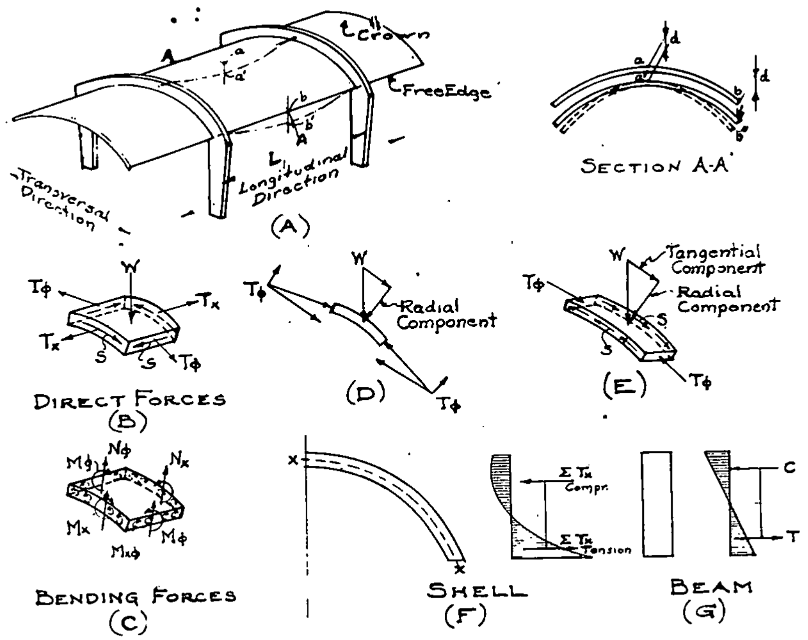

FIG. 1

deflection. Since these two points must coincide it is evident that the lower edge will be strained more than would be the case if the beam theory was strictly applicable.

To express exactly the relationship of the internal forces of a shell the mathematics becomes very complicated. This is because a shell is a space structure and these forces act in three directions instead of the two we are used to handling in conventional beams. The knowledge of the mathematics involved is not necessary with the design data now available in Manual #31, but a clear idea of how a shell carries its load, coupled with an intelligent use of the manual, is necessary if we wish to design shell structures.

Let us examine the internal forces in a shell (stress x thickness or force per unit length) acting on a small segment of that shell. For convenience we will divide them into two parts direct forces (fig. 1B) and bending forces (fig. 1C).

The load is carried principally by direct stress so the direct forces will be considered first. These forces are:

Tx, a longitudinal force acting parallel to the longitudinal axis (it is analogous to the fiber stress in a beam); $T\phi$, a tangential force acting in the transverse direction normal to the longitudinal axis; and S, two equal shearing forces acting in the longitudinal and transverse directions, all acting in the centroid or mid-depth plane.

Consider the small segment as a free body with an external vertical load and the internal direct forces acting on it (fig. 1D). If we take the summation of forces in the radial direction, the radial component of the load W is resisted by the radial components of the tangential forces $T\phi$. (This is similar to the hoop forces caused by water pressure in a tank—where the tension equals the radial pressure times the radius). Because the radial component of the external load in a barrel shell roof varies, the tangential force $T\phi$ will vary.

The tangential component of the load is resisted by the difference between the tangential shears acting on the two parallel faces and the difference in the tangential forces acting on the two radial faces of the segment (fig. 1E). Since the sum of the radial and tangential components of the load is equal to the total load acting on the segment, it follows that any load which can be defined (weight of shell, wind, snow, etc.) can be carried by direct stress and shear, without bending as long as the shell supports supply the necessary reactions.

By way of contrast it is well to recall that an arch can carry only one type of load by direct stress, a load having a thrust line coincident with the centroidal axis of the arch. The presence of the tangential shears in a shell is what enables the shell to carry its load by direct stresses. It should also be noted that the tangential shears in a shell are analogous to the vertical shears in a beam because at any transverse section A-A the summation of the vertical component of these shears equals the loads between that section and the support minus the end reaction. And further, just as the shears in a shell are comparable to the shears in a beam, the longitudinal forces Tx in a shell are comparable to the fiber stresses in a beam.

In fact for certain long shells, specifically shells where the ratio of $\frac{\text{radius}}{\text{length}}$ is about 1/5, the distribution of these longitudinal forces at x-section A-A (if divided by thickness to obtain unit stress) are about equal to what would be obtained by applying the ordinary beam theory $f = \frac{Mc}{I}$ (fig. 1G).

As the $\frac{\text{radius}}{\text{length}}$ ratio increases (1/5 to 1/4 etc.) (the transverse supports get closer together) the distribution departs further and further from the straight line. But regardless of this, the summation of the longitudinal forces (stress x cross-sectional area) at any cross-section must equal zero. And the moment of these longitudinal forces about any transverse axis must equal the statical moment of the loads and reactions (fig. 1F).

If the necessary internal forces $T\phi$ and S are supplied to a small segment of a shell by reactions from the abutting portion of the shell, or by suitable supports, the shell will carry any load by these direct forces (Tx, $T\phi$ and S). But in most shells there are no vertical supports along the longitudinal edges, or else there are only shallow edge beams incapable of fully supplying the required internal forces $T\phi$ and S to the adjacent segment—in other words incapable of resisting in full the reactions from the shell. As a result there is a readjustment of the distribution of the direct stresses in the vicinity of the edge and bending forces are induced.

The bending forces are:

$N\phi$ radial shears on the longitudinal faces of the segments.

Nx radial shears on the transverse faces of the segments.

Mx longitudinal moments on the transverse faces.

$M\phi$ transverse moments on the longitudinal faces, and

$Mx\phi$ torsional moment acting on the four faces as shown.

These moments and radial shears are always small and for this reason a thin shell, only 3″ or 4″ thick, may be used to span distance of 100 ft. or more. The reason that they are small is that bending strains are resisted by direct strains. For a given distortion of a shell the direct forces developed are much larger than the bending forces. Since this is so, I think it is clear that the loads on a shell are resisted mainly by direct stresses analogous to the flexural and shearing stresses in a beam. Tx is like the fiber stress. S is like the shearing stress. $T\phi$ may be compared to the vertical stresses in the web of a beam caused by distributed vertical loads.

So, it is logical to visualize the shell as an ordinary beam with modifications for the effect of change of the curvature of its section. These modifications are explained as follows: Assume the shell acts like a beam and cut a section ABCD as shown in figure No. 2A. The

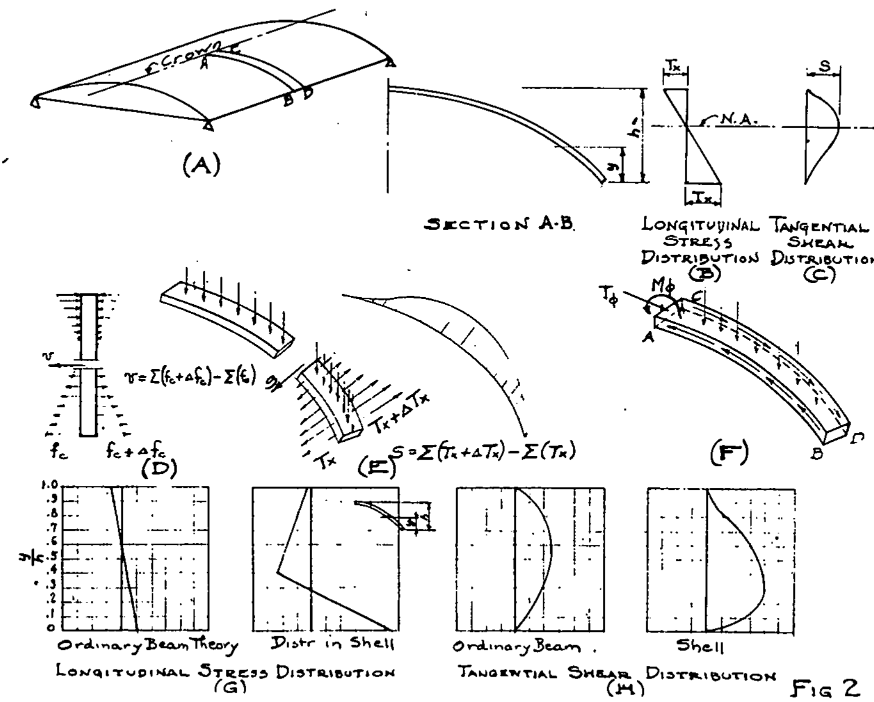

Fig 2

longitudinal force Tx on AB will vary with vertical distances as a straight line. From the difference of the longitudinal forces Tx acting on the two opposite faces, dx distance apart, the longitudinal shear can be obtained (fig. 2D, 2E). This shear S acts in a radial plane and is parallel to the Tx forces.

Since the longitudinal shear at any point equals the tangential shear at that point, we can obtain the value of the tangential shear at any point (from crown to bottom edge). Note that the shear is zero both at the crown and at the bottom edge with the maximum value at the neutral axis of the cross section.

With these values of the tangential shears the value of the transversal moment Mϕ acting on face AC can be obtained (fig. 2F).

$M\phi$ equals the algebraic sum of the moment of the external load and the moment of the tangential shears acting on the two transverse faces. For most distributed loads this algebraic sum of the moments will be in a direction causing tension in upper fibers of shell and compression in lower fibers. In addition to $M\phi$ a normal thrust $T\phi$ is induced on the face AC. This thrust is equal to the sum of the unbalanced horizontal and vertical forces acting on the free body. Consistent with the direction of the internal transverse moment, $M\phi$, induced in the shell, point B deflects downward and inward with respect to A. But with the ordinary beam theory points A and B are supposed to deflect the same amount, that is the vertical distance between A and B is supposed to remain constant. And the assumption of stress distribution in the beam theory is in error by an amount consistent with the distance point B deflects vertically relative to A. Because the lower edge deflects more than the crown, the strain and the force Tx at the edge must be greater than that found by the conventional beam theory. But the sum of the Tx forces on the cross-section must still equal zero. So the increase in the tensile stresses in the region of the edge must be balanced by an increase in the compressive stresses above the neutral axis.

Furthermore, the moment of these longitudinal forces about any axis must equal the sum of the statical moment of the external load and its reaction about that axis. To satisfy this condition the moment arm of the couple must be smaller.

Figs. 2G and 2H serve to illustrate how the longitudinal stresses in a shell vary in comparison to the stresses in an ordinary beam. Note how much the tensile stress at the edge is increased (consistent with the increased strain). And how the compressive stresses above the neutral axis are greater to satisfy the condition of equilibrium.

$$\Sigma \text{ Tx compr.} = \Sigma \text{ Tx tension}$$

Also, since these sums of forces (stress x area) are greater, the moment arm of the couple must be smaller to satisfy the moment condition at this section.

$$\Sigma \text{ M (internal forces)} = \Sigma \text{ M (external forces)}$$

In other words, the effective depth of the shell is smaller.

The increase in the stresses and the accompanying decrease in effective depth is dependent on the ratio of $\frac{\text{radius}}{\text{span}}$. (One might compare the decrease in effectiveness of the flange of a T-beam as the ratio of width to span is increased.

Note on the graph showing the distribution of stress that while the sum of the compressive stresses above the neutral axis is greater, the stress at the crown is not increased; in fact it has become smaller showing that the portion of the shell in the region of the crown is less effective. The effectiveness is decreased more as the ratio $\frac{\text{radius}}{\text{length}}$ is increased (1/4 to 1/3 etc.).

Still another effect occasioned by the redistribution of the longitudinal stresses is that of the shearing stresses near the lower edge. Since the shearing forces vary directly as the longitudinal stresses, the tangential shears in this region are increased.

This tends to cause the resultant of the tangential shears to approach the resultant of the external loads, thus reducing the value of the transversal moment, $M\phi$. This reduction in $M\phi$ is relatively large for small changes in the distribution of the shearing forces because this moment is a function of the difference of these almost like quantities.

Up to this point the effect of the longitudinal bending moments and the corresponding radial shears have not been discussed so as not to complicate the basic behavior of shells, i.e. the carrying of the loads by direct stress. Longitudinal bending stresses do occur, but are not of serious consequence. They are secondary stresses caused by radial displacements as the shell deforms under load. The radial displacements are mainly resisted by the direct stresses and thus are generally small. Since the longitudinal bending moment is directly proportional to the displacement, it will generally also be small except in very short barrels where they may become important. To sum up, when a shell unsupported along the longitudinal edges is subjected to an external distributed load, the free edges of it tend to deflect downward and inward. This composite deflection causes an increase in intensity of the longitudinal stresses as well as the shearing stresses near the lower edges, thereby causing a reduction in the transversal moments.

To illustrate how size and shape of shell effects the distribution of stresses as compared to an ordinary beam and how this stress distribution in shells varies from a straight line, a series of curves has been plotted for several $\frac{\text{radius}}{\text{span}}$ ratios (fig. 3a). Note that when $\frac{r}{L}$ < .2 the shell acts as an ordinary beam. And as the $\frac{r}{L}$ is increased

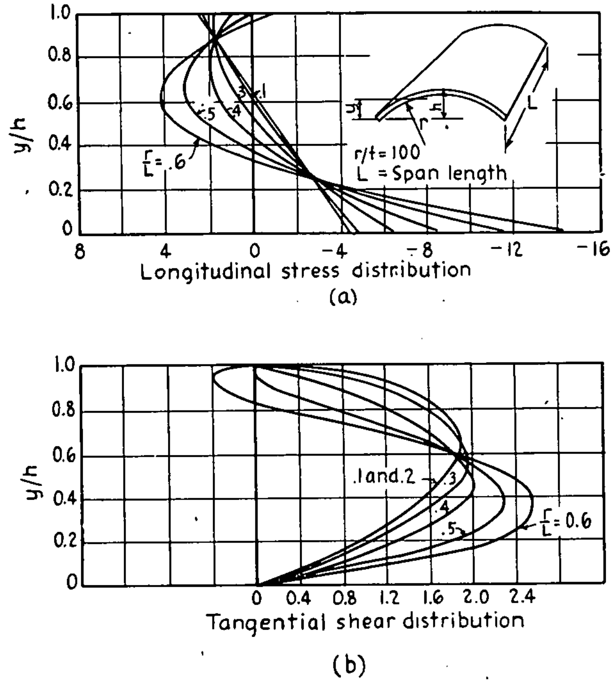

FIG. 3.

the stress distribution departs more and more from the straight line with the intensity of stress near the lower edge increasing.

Fig. 3b shows how the shearing stress intensity is increased near the lower edge as the $\frac{r}{L}$ is increased.

So far we have been concerned with single shells. What happens when two shells have a common edge? We spoke of how the normal tendency was for the edge of a single shell to deflect downward and inward. If two shells have a common edge the tendency for this horizontal deflection is offset by the adjacent shells. Consequently the intensity of stresses near the edge is reduced. This can be seen in fig. 4. The full lines represent the stress distribution in a single shell while the dotted lines represent the stress distribution in a shell having two edges restrained by adjacent shells. Note how for these particular shells the intensity of longitudinal stress is reduced from 1,710 lbs./

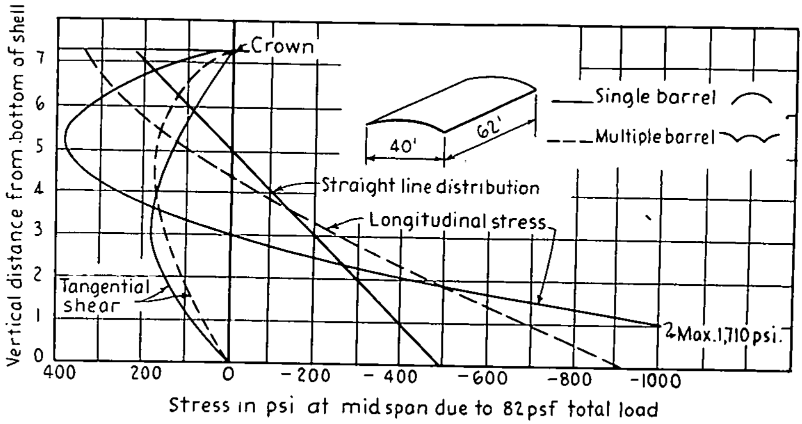

FIG. 4.

sq. in. to 880 lbs./sq. in. simply by the restraint of one shell on the other.

Where does this stress go? Note that the area near the crown becomes more effective in the case of the restrained shell. The shell thickness affects the stress distribution as shown in fig. 5. The curves are for three 60′ radius shells, with a chord width of 50′ and a span of 50′, with the thicknesses of 3″, 4″ and 5″. As the thickness is increased greater resistance is offered to the transversal rotation caused by $M\phi$. Therefore, as the thickness is increased, the longitudinal stresses become smaller and the distribution approaches that of the distribution for an ordinary beam. On the other hand, the transversal moment $M\phi$ increases with the increase in thickness.

Fig. 6 shows the stress distribution for a shell having a span length equal to one-fifth the chord width. The longitudinal stresses are very large near the lower edge and are reduced sharply as we move up on the shell.

Now what about the cross supports of the barrel shell? Since we have seen that the barrel acts principally as a beam between the supports, the loads supported by the shell are carried to the supports by tangential shears and radial shears—mainly by tangential shears. Therefore, the supports must be designed to carry the reaction from these shears in addition to its own dead weight. Note that these shear reactions act along the line of intersection of the barrel and the cross

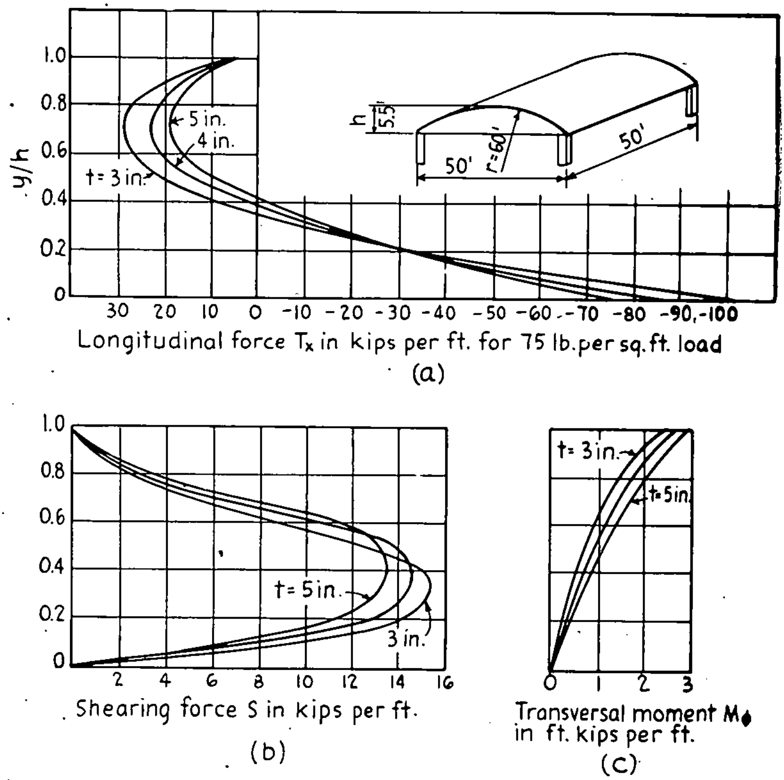

FIG. 5.

support; and on account of the direction in which they act they will induce moments into the beam. Remember also that these loads are not uniformly distributed.

These tangential and radial shears could be considered directly as external loads on the cross beam, but this would involve the determination of the distribution together with the summation of the moments at any point along the beam. This would entail a lot of laborious work. To avoid this work, surface loads can be substituted for the effect of these shears. Although these surface loads differ from the shears, both in the direction they act and in the way they are distributed, the effects produced are similar. This can be seen in fig. 7.

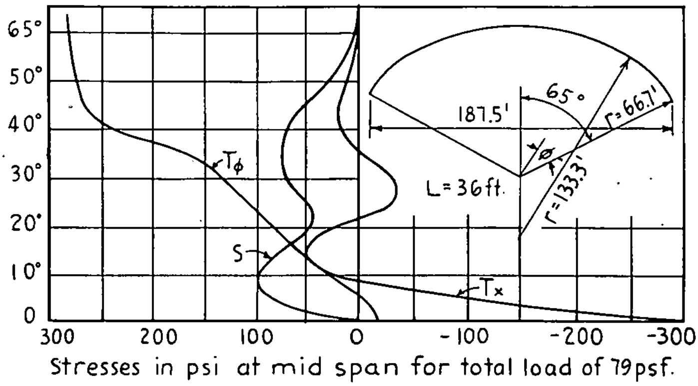

FIG. 6.

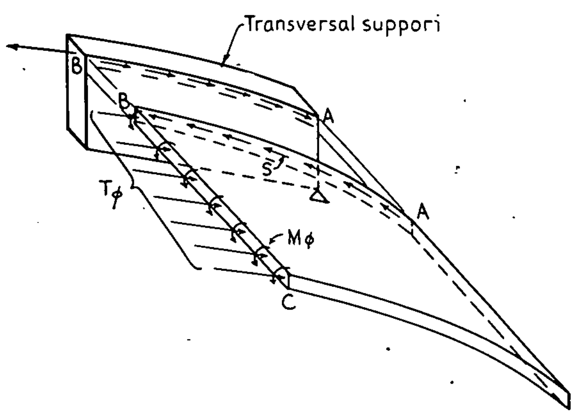

FIG. 7.

Consider this portion of a shell ABCD as a free body. The boundary forces must hold in equilibrium the surface loads. If edge CD is in the plane of zero shear at the middle of the span, then the moment of the shearing forces along line AB (at the junction of the shell and the beam) about any longitudinal axis such as BC parallel to the edge AD, is equal and opposite to the sum of the moments of the surface loads of the shell and the transverse moment Mϕ acting at line BC. This moment Mϕ is small compared to the moment of the surface loads about BC and may be neglected. Thus the moment of the surface loads in area ABCD, about BC is equal to the moment of the shearing forces at the beam about BC. This relationship is so for any point, therefore the moment induced in the beam may be computed directly from the surface loads. The reactions from these surface loads are distributed on the beam in the same manner they are distributed in the shell.

Finally a horizontal thrust from the tangential shears is imparted to the beam and this thrust is equal to the sum of the horizontal components of the tangential shears or the sum of the horizontal forces acting on line BC.

Conclusion

This treatment of shells was solely to explain the manner in which shells carry their loads. Because shells act primarily as beams with curved cross sections, the loads are carried almost wholly by direct compression and direct tension and by tangential and longitudinal shears. The direct forces are analogous to the flexural stresses and shearing stresses in a beam with the exception that the longitudinal stresses are not proportional to their distances from the neutral axis.

The important difference between the design of a shell and that of a beam is that the distribution of flexural stress vs. depth is linear in a beam, while the longitudinal stress distribution in a shell is non-linear and is dependent on shell proportions and edge conditions. The determination of this distribution of the longitudinal stresses is simplified considerably by the various tables in Manual #31. With this manual it is felt that barrel shell designs may be undertaken by engineers without much more time being used than it takes to design an indeterminate structure. All significant direct forces and bending forces are readily obtained from the tables, and it is possible to vary proportions in trial designs for economy without excessive labor.

This paper, of course, has only covered the bare essentials of shell design, but it is hoped that it will whet your appetites for a broader knowledge of the subject contained in the booklet.

Acknowledgment

This paper was prepared with the assistance of A. L. Parme, of the General Office of the Portland Cement Association, who was largely responsible for the preparation of Manual #31.

BUCKLING CONSIDERATIONS IN THE DESIGN OF STEEL BEAMS AND PLATE GIRDERS

By J. M. BIGGS,* Member

(Presented at a meeting of the Structural Section of the Boston Society of Civil Engineers, held on October 13, 1954.)

INTRODUCTION

ELASTIC stability has long been recognized as an essential requirement for the successful design of structural members. In general, however, the average engineer had only a very superficial knowledge of the subject and for design purposes he used certain rules of thumb plus a considerable amount of engineering intuition. The demands of modern engineering with its emphasis on lightness and simplicity and the use of higher stresses require a more precise approach to the buckling problems encountered in structural design.

Our design specifications rely upon the engineer's ability to determine the actual stresses under load but he is allowed very little leeway in the determination of allowable stresses and is forced to accept rather arbitrary values. When a formula for allowable stress is intended to apply to a wide variety of cases it must necessarily be conservative for all but one case and uneconomical designs result. It makes very little sense to compute the bending moment in a beam by fairly exact methods of analysis and then select the beam size by use of an allowable stress which is a crude approximation. The time must eventually come when the designer is given more freedom in the determination of allowable stresses but this cannot be accomplished until the designer is prepared to accept the additional responsibility.

This paper is a discussion of the buckling phenomena which must be considered in the design of steel beams and plate girders. It presents a review of the available theory and on this basis attempts to evaluate the methods and criteria currently used for design.

LATERAL BUCKLING

Theory

When an I-beam which is laterally unsupported except at the ends is loaded in the plane of the web (see Fig. 1 (a)) failure may

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occur in the form of lateral buckling as indicated in Fig. 1 (b). If the lateral rigidity (EI) is much smaller than the vertical rigidity this type of failure may occur at a stress well below the yield point of the material and the allowable stress for design must be reduced accordingly.

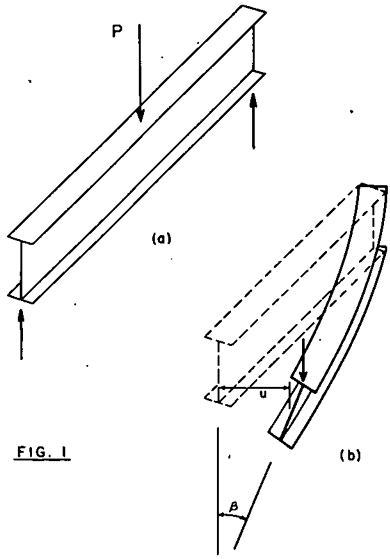

FIG. 1

The problem of lateral buckling was first considered in 1899 by A. G. M. Michell (1).[1] This work was confined to beams of rectangular cross section and the first solution for the buckling of I-beams was published by S. Timoshenko in 1906. A summary of the voluminous work of Prof. Timoshenko in this field is contained in Reference (2). A more complete history of the theoretical developments may be found in Reference (3).

The top flange of a beam is analogous, in some respects, to a column and being in compression it has a tendency to buckle. Instability of the top flange results in a lateral motion of the beam. When this occurs the bending moments are no longer confined to the plane of the web and the beam is twisted as indicated in Fig. 1 (b). Since the twist helps to resist the lateral motion of the top flange it is obvious that the critical or buckling load depends upon the torsional rigidity of the beam as well as the lateral bending rigidity.

[1]Numbers in parentheses refer to Bibliography.

It is assumed throughout this paper that the end cross sections of the beam are prevented from twisting about the longitudinal beam axis. This is the usual case and indeed if such restraint were not provided a beam loaded on the top flange would be unstable for any value of the load. Since the flanges do not move laterally at the ends of the span, a twisting of the beam must be accompanied by lateral bending in each of the flanges. This bending increases the torsional rigidity significantly and must be considered when computing the critical buckling load of an I-beam.

One of the procedures which may be used to obtain the buckling load of a beam is an application of the energy method devised by Timoshenko (2a). By this procedure a buckled shape defined by μ and β in Fig. 1(b) is assumed. When the beam moves into the buckled position the internal strain energy is increased due to the torsional stresses and the lateral bending of the flanges. At the same time the load lowers and thereby produces work. If the change in internal strain energy is exactly equal to the work done by the load the system must be unstable because there would be no tendency for the beam to return to its original position. In the application of the method the strain energy and external work are computed in terms of the load, P, and equated. This equation when solved for P yields the value of the critical load at which buckling occurs. For convenience the critical load is usually converted into critical stress intensity on the extreme fibre at the point of maximum moment.

The critical stress intensity for an I-beam on a simple span may be expressed as follows:

$$f_{cr} = AE \frac{I_{yy}}{I_{xx}} \frac{d^2}{l^2} \left(\frac{1}{a}\sqrt{1 + B\frac{a^2}{l^2}} + C\right) \tag{1}$$

in which, $f_{cr} = \dfrac{Mc}{I_{xx}}$ = critical stress intensity at point of maximum moment

E = Young's Modulus

I_{yy} = moment of inertia in the weak plane

I_{xx} = moment of inertia in the strong plane

d = depth of beam

l = unsupported length of span

$$a^2 = \frac{EI_{yy}d^2}{4\ G\ K}$$

$$G = \text{Shearing Modulus} = \frac{E}{2(1+\nu)} \approx \frac{E}{2.6}$$

$$K = \text{Torsional Constant}$$

A, B, and C are constants depending upon the distribution and point of application of the load and the location of lateral restraints. Value of these constants for several cases are given in Table I.

TABLE I

Values of Constants A, B, and C in Eq. (1). Simple-span I-beams

	Lateral Support	Load	Point of Load Application	A	B	C
I	At Ends Only	Pure Bending	—	0.785	π^2	0
II		Concentrated Load at Midspan	Centroid	1.071	π^2	0
III			Top Flange	1.071	12.88	—1.742
IV			Bottom Flange	1.071	12.88	+1.742
V		Uniform Load	Centroid	0.888	π^2	0
VI			Top Flange	0.888	11.93	—1.440
VII			Bottom Flange	0.888	11.93	+1.440
VIII	At Ends and Mid-Span	Conc. Load	—	1.475	π^2	0
IX		Uniform Load	Centroid	1.060	π^2	0
X			Top Flange	1.060	10.03	—0.429
XI			Bottom Flange	1.060	10.03	+0.429

Of the terms appearing in Eq. (1), the parameters "K" and "a" perhaps require some explanation. The torsional constant, K, is a measure of the resistance to twist and is related to the torque M_t, and the resulting angle of twist per unit length, θ, by the equation, $K = \frac{M_t}{\theta G}$. Values of K are tabulated in Reference (4) for all standard rolled I-beams. A good approximation may be obtained by,

$$K = 1.1\left(\frac{2}{3}\,b\,t^3 + \frac{1}{3}\,d\,t_w^3\right) \tag{2}$$

in which, b = width of flange
d = depth of beam
t and t_w are thicknesses of flange and web respectively.

The parameter, a, is used merely for convenience and may be obtained from Reference (4) or computed by the equation given.

Eq. (1) is applicable only within the elastic range of the material for the same reason that the Euler equation is valid for column buckling only within that range. Above the proportional limit assumed herein to be 25 kips per sq. in.) the values of E and G decrease and Eq. (1) must be modified accordingly. There is no completely satisfactory method for determining buckling loads in the inelastic region but the procedure described below is believed to produce reasonable results.

If it is assumed that the ratio E/G remains constant for any stress value and that the variation in E along the span may be ignored, then Eq. (1) indicates that f_{cr} is directly proportional to E(3a). Letting f'_{cr} be a critical stress intensity above the proportional limit and E′ the value of Young's Modulus at that stress, we may assume the following,[2]

$$\frac{E'}{E} = (36500 - f'_{cr})\,\frac{f'_{cr}}{28.6 \times 10^7} \tag{3}$$

$$f'_{cr} = \frac{E'}{E}\,f_{cr} \tag{4a}$$

$$\text{or, } f'_{cr} = 36500 - \frac{28.6 \times 10^7}{f_{cr}} \tag{4b}$$

not to exceed 33000.

The value of f_{cr} is obtained from Eq. (1) using the normal value of E. It is a hypothetical stress and may exceed the yield point. Eqs. (3) and (4) apply only to a material having a yield point of 33000 lbs.

[2]Eq. (3) is based upon the following assumptions: (1) The proportional limit is 25 k/in² and the yield point is 33 k/in²; (2) The critical stress intensity for a column may be expressed as $\Pi^2 E'/(\frac{l}{r})^2$ or $A - B(\frac{l}{r})^2$; (3) The critical stress for a column reaches the yield point at $\frac{l}{r} = 60$. For a similar treatment see Reference (2d).

per sq. in. and a proportional limit of 25000 lbs. per sq. in. It is customary to assume the yield point to be the upper limit of critical stress even though the strength of very short beams may slightly exceed this value.

As indicated by the constants given in Table I, the point of application of the load has a considerable effect on the critical stress. As a beam twists into its buckled position (see Fig. 1 (b)), a load applied at the bottom flange produces a restoring torque which re-

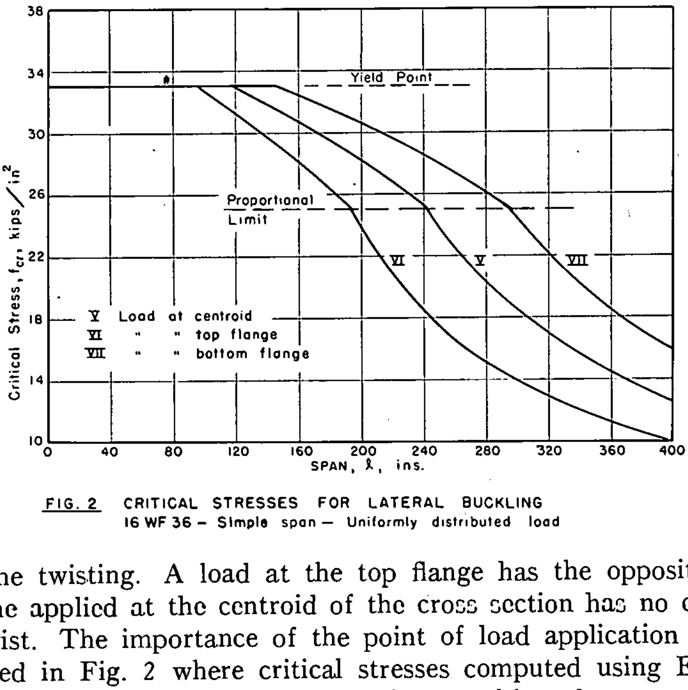

FIG. 2 CRITICAL STRESSES FOR LATERAL BUCKLING
16 WF 36 – Simple span – Uniformly distributed load

sists the twisting. A load at the top flange has the opposite effect and one applied at the centroid of the cross section has no effect on the twist. The importance of the point of load application may be observed in Fig. 2 where critical stresses computed using Eqs. (1) and (4) are plotted for a typical I-beam subjected to a uniformly distributed load on a simple span. The effect varies with span but the critical stress for the beam loaded at the bottom flange may be 75 per cent greater than that for the beam loaded on the top flange.

Critical stresses for several other cases are plotted in Figs. 3 and 4. It is not possible to compute the critical stress in general terms and therefore values are given for a particular I-beam. The variation for different I-beam sections is discussed below. It may be observed in Fig. 3 that for a simple beam laterally supported only at the ends the effect of load distribution is not great. The maximum difference in the critical stresses for Cases I and VI is about 22 per cent.

When the beam is laterally supported at mid-span the critical stresses become considerably greater. For example, in the case of a concentrated load at mid-span (Case VIII in Fig. 3) the buckling

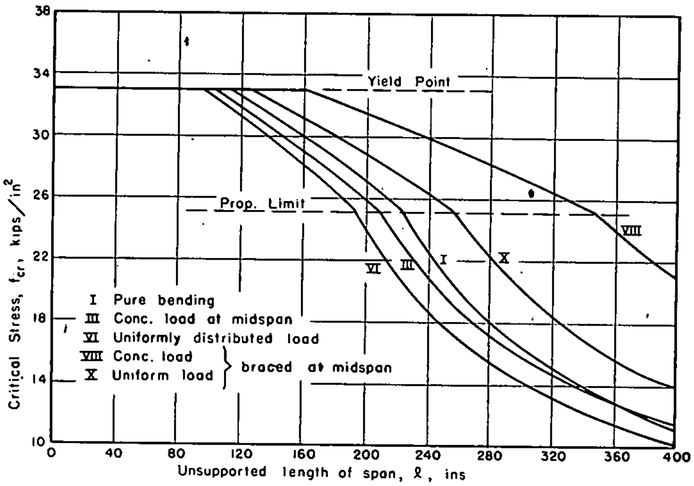

FIG. 3 CRITICAL STRESSES FOR LATERAL BUCKLING
16 WF 36 — Simple span — Loads applied at top flange

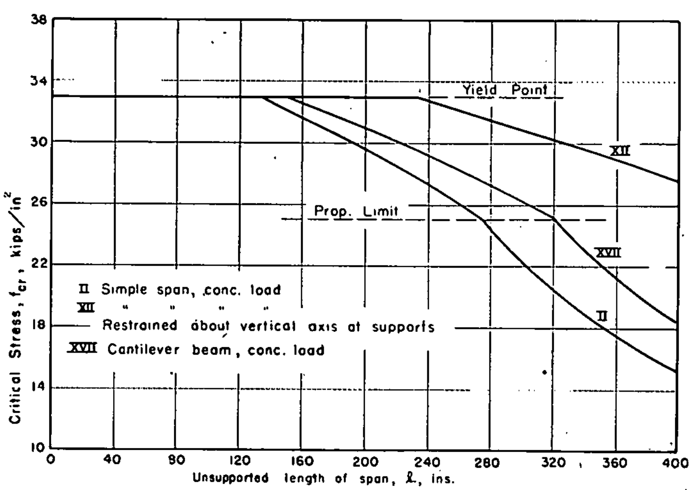

FIG. 4 CRITICAL STRESSES FOR LATERAL BUCKLING
16 WF 36 Loads applied at centroid of cross section

stress for a beam braced at the center may be 84 percent greater than that for an unbraced beam having the same unsupported length or half the span. In this regard it is important to understand what constitutes lateral support. If two beams are braced as shown in Fig. 5(a) no lateral support is provided because the beams may

(a) (b)

FIG. 5

buckle together. However, if diagonals are added as in Fig. 5(b) the beams are adequately restrained at this point because twisting is prevented. If the tie in Fig. 5(a) is continued to a fixed point the beams are properly supported provided that the tie has sufficient strength. In general the tie should be capable of developing a force equal to about 10 per cent of the strength of the compression flange.

It is also important to note that beams must be prevented from twisting at the supports. This restraint may be provided by a web connection or by anchor bolts through the bottom flange acting together with web stiffeners. It is probable that in some cases the strength of a beam is reduced by the failure of the designer to provide adequate rotational restraint at the supports.

If the ends of an I-beam are restrained from rotation about the vertical axis rather than being free as indicated in Fig. 1(b), the critical stress is considerably increased. This results from an increased bending stiffness of the flanges and consequently an increase in resistance to twisting of the beam. The magnitude of this effect is indicated by comparison of Cases II and XII(2b) in Fig. 4.

Also shown in Fig. 4 are critical stresses for a cantilever beam completely free at the end and subjected to a concentrated load (2c). The critical stresses are higher than for a simple span having the same unsupported length.

Specifications

The widely used A.I.S.C. specification (5) for allowable compressive stresses in laterally unsupported beams is given by,

$$f = \frac{12{,}000{,}000}{(ld/bt)} \text{ lbs. per sq. in.} \tag{5}$$

Eq. (5) is basically a good specification but since it is intended to apply to all types of beams and loadings it obviously is inaccurate for many cases.

Eq. (5) was first recommended by K. DeVries (6) and is intended to be a simplification of the theoretical solutions discussed above. It was obtained by plotting a series of curves similar to those in Figs. 2, 3, and 4 for several standard I-beam sections and then selecting an empirical equation which would be conservative for all cases. An excellent discussion of the paper by DeVries was contributed by O. G. Julian, Member, BSCE (6b).

It may be demonstrated that the parâmeter ld/bt is significant although it is not an exact indication of the critical stress. Consider Eq. (1) with the constants A, B, and C corresponding to the most severe case (Case VI in Table I) and make the following substitutions:

$$I_{yy} \cong \frac{tb^3}{6} \qquad I_{xx} \cong \frac{btd^2}{2} \qquad K \cong \frac{2}{3}\,bt^3 + \frac{1}{3}\,dt^3_w.$$

The resulting expression may be written in the form,

$$f_{cr} = \frac{22 \text{ x } 10^6}{(ld/bt)}\left[\sqrt{1 + \frac{dt^3_w}{2bt^3} + 1.94\,\frac{b^2d^2}{l^2t^2}} - 0.58\,\frac{bd}{lt}\right] \tag{6}$$

The term in brackets depends upon beam section and span but for beams which would buckle elastically its value does not differ greatly from unity. The difference between the numerical factor, 22 x 10^6, and that in Eq. (5) represents a factor of safety. It may therefore be concluded that, in the elastic range, Eq. (5) is a reasonable design formula for the most severe condition.

A further appraisal of the A.I.S.C. specification is given by Fig. 6. Assuming the same factor of safety as in tension Eq. (5) when converted to failure stresses becomes,

$$f_{cr} = \frac{12{,}000{,}000}{(ld/bt)} \text{ x } \frac{33}{20} = \frac{19{,}800{,}000}{(ld/bt)} \tag{7}$$

This equation is plotted in Fig. 6 along with curves of critical stresses for two standard I-beams subjected to the most severe case of loading. The two I-beams were selected because they provide extreme values of critical stress and the curves for most other I-beam sections fall between the two curves shown. The maximum difference between the critical stresses for the two I-beams is about 26 per cent which is equivalent to the maximum variation in the bracket term of Eq. (6). This difference in critical stress for various I-beams having the

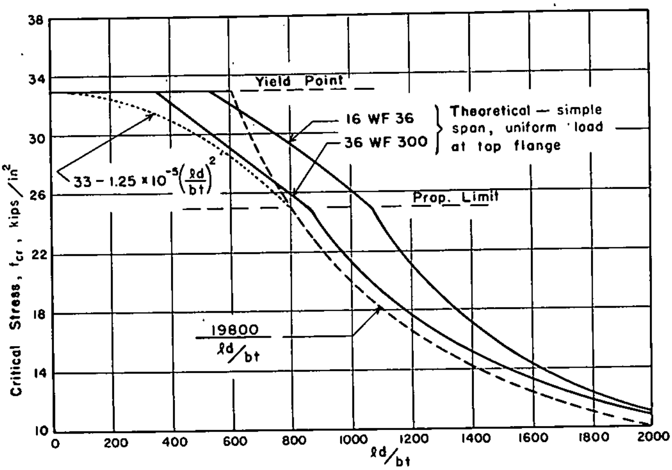

FIG. 6 COMPARISON OF THEORETICAL BUCKLING STRESS AND SPECIFICATIONS

same ld/bt is in no way indicated by the specification. However, as may be observed in Fig. 6, the specification was intended to be safe for the most severe case.

The fact that the three curves of Fig. 6 are similar in shape below the proportional limit is further proof that ld/bt is a significant parameter in the elastic range. However, the AISC specification makes no allowance for the reduction in E above the proportional limit. By assuming elastic behavior up to the yield point the formula becomes unconservative for ld/bt values less than approximately 800. In DeVries' original proposal (6) the equation

$$f_{cr} = 33000 - 0.0125 \left(\frac{ld}{bt}\right)^2 \tag{8}$$

was used as a transition curve in the region of ld/bt values below 800 (see Fig. 6). Although completely arbitrary, use of Eq. (8) would maintain approximately the same factor of safety through the entire range of ld/bt. However, for the sake of simplicity the AISC Committee on Specifications saw fit to use the elastic formula (Eq. (7)) for all stresses below the yield point (6a). This decision seems unfortunate since the use of a transition equation would have made the whole procedure more logical without unduly complicating the design.

In the derivation of Eq. (7) it was assumed that the factor of safety for compression in beams should be the same as that for tension elements. The plot of this equation in Fig. 6 lies below the theoretical curves but this does not necessarily mean that the specification is too conservative. These theoretical curves are derived for an "ideal" beam and it is well known that imperfections in the beam and eccentricities of the loading tend to reduce the critical stresses. As in the case of columns (7), it would seem desirable to make some allowance for this effect.

It has been suggested (8) that a secant formula taking into account possible imperfections be used for beams in the same manner that is used for column design (7). However, since the secant formula is not a rigorous indication of the critical buckling stress for a beam and since an exact solution is extremely difficult, it would seem more logical as well as expedient to allow for the imperfections by simply increasing the factor of safety.

The reduction in critical stress due to imperfections is most pronounced in the case of short beams and if imperfections are considered the critical stress would never reach the yield point. An inspection of Fig. 6 reveals that the AISC specification is probably satisfactory in the elastic range but for shorter beams it is unconservative.

The discussion of the AISC specification thus far has been based on a comparison with the most severe case of loading which is a uniformly distributed load applied to the top flange of a simple span beam. As previously mentioned in the discussion of Figs. 2, 3, and 4, the critical stress varies radically with the type of loading and the manner in which the beam is supported. The specification makes no allowance for this fact.

Assuming that Eq. (7) is adequate for design purposes, differences in loading and beam supports may be taken into account by use of reduced lengths in that equation. Values of the reduction factor, k, are suggested in Table II. These factors are developed from the theoretical solutions available in the literature. However, in the cases indicated, values are obtained by estimation and are presented only as an approximate indication of the magnitude of k for those cases.

TABLE II

Values of k in the Formula, Allow. $f = \dfrac{12{,}000{,}000}{\dfrac{kld}{bt}}$

Case	Beam Type	Load	Point of Application	k
I	Simple Span	Pure Bending	—	0.91
II		Conc. Load at Mid-Span	Centroid	0.71
III			Top Flange	0.93
IV			Bott. Flange	0.58
V		Unif. Load	Centroid	0.82
VI			Top Flange	1.00
VII			Bott. Flange	0.70
VIII	Simple Span Braced at Midspan	Conc. Load	—	0.57
IX		Unif. Load	Centroid	0.72
X			Top Flange	0.76
XI			Bott. Flange	0.68
XII	Simple Span Flanges Restrained at Ends	Conc. Load	Centroid	0.42
XIII			Top Flange	0.55*
XIV			Bott. Flange	0.34*
XV		Unif. Load	Centroid	0.45
XVI			Top Flange	0.55*
XVII			Bott. Flange	0.38*
XVIII	Cantilever	Conc. Load	Centroid	0.62
XIX			Top Flange	0.81*
XX			Bott. Flange	0.51*
XXI		Unif. Load	Centroid	0.39*
XXII			Top Flange	0.48*
XXIII			Bott. Flange	0.33*

*Estimated.

The value of k varies with span and those given in Table II are the maximums for spans within practical limits. However, this variation with span is not great and the error in these k-values is never serious. By this procedure the allowable stress for a given case is given by,

$$\text{allow. f} = \frac{12{,}000{,}000}{\dfrac{\text{kld}}{\text{bt}}} \text{ lbs. per sq. in} \qquad (9)$$

The reduced length, kl, may be defined as the length of a Case VI beam having the same critical stress as the beam of the case under consideration having a length of l. There are many other cases which would be of interest (such as continuous beams) but exact solutions for I-beams are not available and approximations would be unreliable. It is probable, however, that theoretical solutions for all significant conditions would be rapidly developed if the need were apparent.

An investigation of Table II reveals that considerable economy in design would result from its use. Such a procedure could easily be included in specifications without unduly complicating design. In one particular case, the AISC specification does provide such a factor. For a cantilever a k-value of 2 is specified but Table II shows that this value is totally wrong and very conservative.

Two other commonly used formulas for allowable stress intensities in the compression flange of a beam are,

$$f = \frac{22500}{1 + \dfrac{l^2}{1800\ b^2}} \text{ lbs. per sq. in.} \qquad (10)$$

not to exceed 20000

and,

$$f = 18000 - 5 \left(\frac{l}{b}\right)^2 \text{ lbs. per sq. in.} \qquad (11)$$

Both of these formulas are based upon the assumption that the compression flange behaves as a column which is incorrect since the torsional stiffness of the beam is neglected. The ratio $\frac{l}{b}$ in Eqs. (10) and (11) is equal to $\frac{l}{r\sqrt{12}}$ where r is the radius of gyration of the compression flange in the lateral direction. Substituting for $\frac{l}{b}$ in Eqs. (10) and (11) we obtain two very familiar column formulas: Eq. (10) becomes the Rankine formula and Eq. (11) becomes the parabolic formula. Both equations are very crude approximations for the critical stress intensity in a beam subject to lateral buckling.

By assuming the same factor of safety as in tension, Eqs. (10) and (11) may be converted into failure formulas as follows:

$$f_{cr} = \frac{37200}{1 + \frac{l^2}{1800\ b^2}} \text{ lbs. per sq. in.} \tag{12}$$

and,

$$f_{cr} = 33000 - 9.16 \left(\frac{l}{b}\right)^2 \text{ lbs. per. sq. in.} \tag{13}$$

Eqs. (12) and (13) are plotted in Figs. 7(a) and 7(b) for comparison with the theoretical solution for the two typical beam sections.

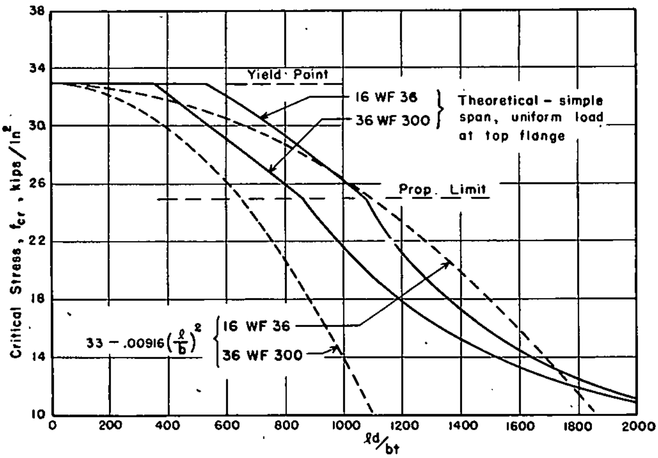

FIG. 7a COMPARISON OF THEORETICAL BUCKLING STRESS AND SPECIFICATIONS

It may be observed that both equations exaggerate the difference between the two beams. Eq. (12) produces a curve of approximately the correct shape but may be seriously in error in some cases. Eq. (13) is reasonable for short spans but is completely irrational for longer spans. Neither of these equations displays much merit except for short spans.

Plate Girders

When computing the theoretical critical stress intensity for a plate girder by Eq. (1) the only complication is in the computation of the torsional constant, K. This value may be easily obtained, however, by rewriting Eq. (2) in the form,

$$K = \Sigma \left(\frac{1}{3}\, bt^3\right), \tag{14}$$

in which b and t are the larger and smaller sides, respectively, of the solid rectangles making up the cross section. In the flanges of a

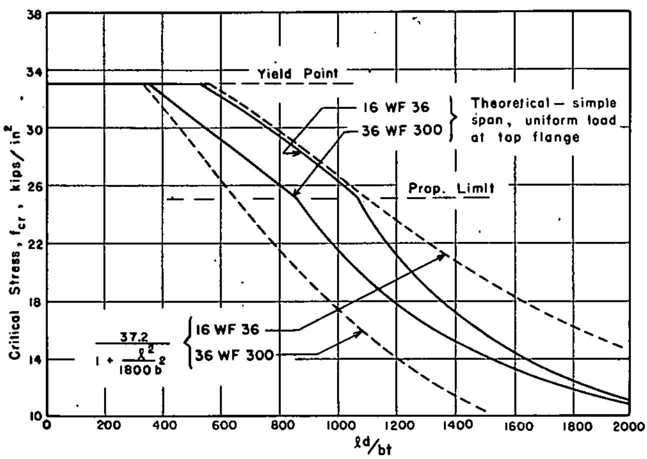

FIG. 7b COMPARISON OF THEORETICAL BUCKLING STRESS AND SPECIFICATIONS

riveted plate girder it may be assumed that the plate elements between rivet lines are effectively joined. The value of K for the girder shown in Fig. 8 is therefore the sum of the quantities $\frac{1}{3}\, bt^3$ for the 21 rectangles in the cross section. The above procedure has been verified experimentally (9). In the case of welded plate girders Eq. (2) may be applied directly.

In the use of the AISC design formula, Eq. (5), for riveted plate girders the appropriate value of the flange thickness, t, is uncertain. It has been shown (6) however, by a comparison of Eq. (5) to theoretical solutions for plate girders that a reasonable value is given by,

$$t = \frac{5\, I_{yy}}{b^2} \tag{15}$$

in which I_{yy} is the lateral moment of inertia of the entire girder and b is the maximum flange width. If the number of cover plates varies along the span an average value of I_{yy} should be used.

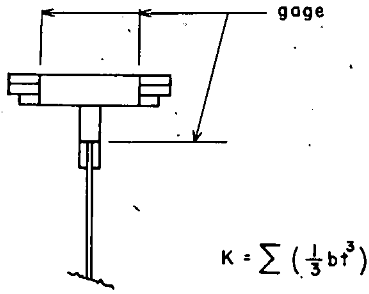

FIG. 8

Buckling of Flange Elements

Theory

In the design of steel beams and plate girders one must consider not only the over-all stability discussed above but also the local stability of individual plate elements in compression. Figs. 9(a), (b), and (c) illustrate three types of local buckling which may occur in the compression flange of a plate girder. In (a) the outstanding leg

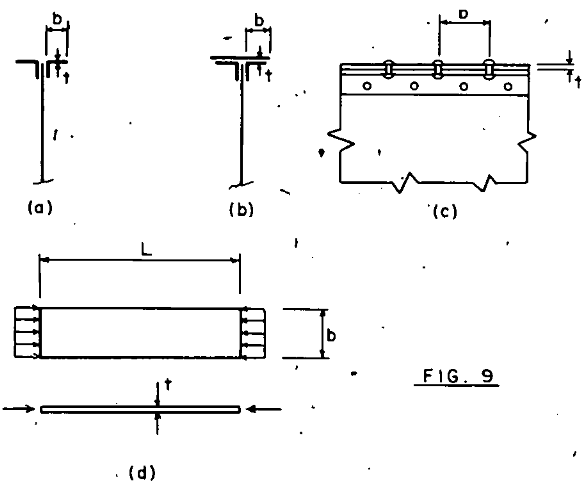

FIG. 9

of the angle may buckle if the ratio b/t is excessive; in (b) the outstanding portion of the cover plate may buckle along with the angle leg; and in (c) the cover plate may buckle between rivets. The type of buckling shown in (a) may also occur in welded plate girders but would never occur in rolled sections because the b/t ratios for the flanges of all standard sections are small enough to prevent buckling.

All of the cases shown in Figs. 9(a), (b), and (c) may be investigated by a consideration of the idealized condition shown in Fig. 9(d). This is the case of a rectangular plate subjected to uniform compression in one direction. The buckling of such a plate was first investigated by G. H. Bryan in 1891 and the critical stress intensity is given by,

$$f_{cr} = \frac{k\,\pi^2\,E}{12(1-\nu^2)\left(\frac{b}{t}\right)^2} \qquad (16)$$

in which, ν = Poisson's Ratio = 0.3.

The numerical factor k depends upon the length to width ratio, L/b, and the edge conditions along the four sides of the plate. Values of this factor for various edge conditions and for other types of load distribution have been determined (2e).

Eq. (16) applies only within the elastic range and for stresses above the proportional limit it must be modified to allow for the change in mechanical properties of the material. The critical stress in the inelastic range may be obtained by,

$$f'_{cr} = f_{cr}\sqrt{\frac{E'}{E}} \qquad (17)$$

where $\frac{E'}{E}$ is given by Eq. (3) and f_{cr} by Eq. (16). The solution must be obtained by trial and error. Eq. (17) differs from Eq. (4a) because the buckling of a plate involves stresses in two directions and E does not have the same value both ways (2d). Eq. (17) is an approximation but experimental evidence indicates that it is a reasonable approach (3b).

Critical stresses for plates compressed in one direction are plotted in Fig. 10 as computed by Eq. (16) modified by Eq. (17) above the proportional limit. The values of k indicated (3c) are for long plates (say $L > 5b$) since the cases under discussion fall in that category. The stress in all cases is perpendicular to the width b.

Specifications

In the application of Fig. 10 to design it is possible to determine the maximum b/t ratio required to prevent buckling at any particular

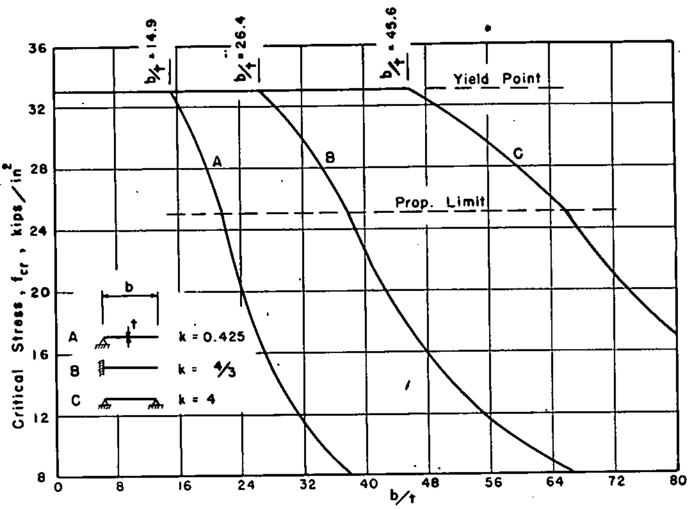

FIG. 10 CRITICAL STRESSES FOR LONG PLATES IN COMPRESSION

working stress. For example, suppose that due to bending the working stress in the outstanding legs of the angles shown in Fig. 9(a) is 12 kips per sq. in. Using a factor of safety of 1.65 this implies an ultimate stress of 12 x 1.65 or 19.8 kips per sq. in. Assuming for the moment that Curve A in Fig. 10 applies to this case, it may be seen that a b/t ratio of 24 would prevent buckling at this ultimate stress and provide the desired factor of safety.

The above approach neglects the fact that the working stress in bending $\left(\frac{MC}{I}\right)$ represents an average flange stress and, due to various inaccuracies, the local stress in a particular element may be somewhat higher. In view of this fact, and also the fact that in most cases the working stress times the factor of safety is close to the yield point, the specifications establish maximum b/t ratios such that buckling can never occur at a stress below the yield point. Although this procedure is conservative it probably does not severely handicap the designer. However, in cases where the working stress is below the value normally used, b/t ratios larger than the limiting values given by the specifications are permissible.

In the case of outstanding compression elements (Figs. 9(a) and 9(b)) the maximum b/t ratio permitted by specifications ranges from 12 to 16. Referring to Fig. 10, it is uncertain whether Curve A or Curve B applies to this case. The correct values lie somewhere between the two curves depending upon the amount of restraint offered by the web of the girder. The degree of restraint is difficult to determine but let us assume that the correct solution lies midway between Curves A and B. Thus no buckling would occur below the yield point if b/t is equal to 20.6. The limiting ratios given by specifications are well below this value.

Considering the case shown in Fig. 9(c), the specifications limit the b/t ratio of the cover plate where b is the distance between the gage lines measured perpendicular to the line of stress. The maximum permitted value of this ratio ranges from 24 to 32. This case is roughly equivalent to Curve C in Fig. 10 where the two supports represent the rivet lines. In the actual condition the support varies from considerable fixity at the individual rivets to very little support between rivets and the critical stress obviously depends on the spacing of rivets in the direction of stress (10). For the present purpose, however, it is probably reasonable to assume that the condition is equivalent to a long plate with simply supported edges. By this assumption Curve C indicates that buckling would not occur below the yield point if b/t is less than 45.6. The specifications therefore appear to be conservative but in view of the uncertainties involved they are probably reasonable.

Another specification requirement is that the distance between rivets in the direction of stress (Fig. 9(c)) shall not exceed 16, or in some cases 12, times the cover plate thickness. This type of buckling is not covered by the previous theoretical discussion. However, we may reasonably consider the plate to be a column between rivets. Again, the end conditions are uncertain and depend upon the distance between gage lines but let us conservatively assume that the column is pin-ended. Experiments indicate that the yield point may be attained in a column without buckling if l/r is less than 60. Letting $l = b$ and $r = t/\sqrt{12}$ this is equivalent to $b/t = 17.3$. On this basis the specifications appear to be reasonable although the limitations imposed are at best rather crude approximations.

Buckling of Web Plates

Theory

Although the theory of elastic stability with relation to web plates is complex, it has been rather thoroughly investigated (2f) (3d) (11). The discussion below is limited to plate girders since the webs of standard rolled beams are sufficiently stiff to prevent buckling under normal conditions.

The two extreme stress conditions which may occur in the web of a plate girder are shown in Fig. 11 where a = clear distance be-

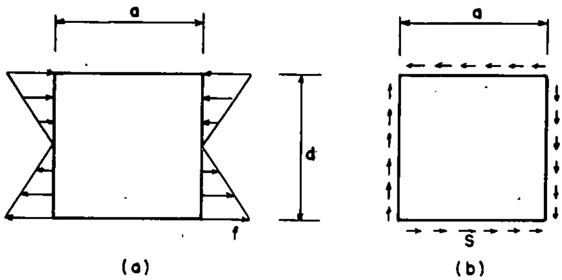

FIG. 11

tween stiffeners and d = clear distance between flanges. For example, in the case of a simple span symmetrically loaded, a pure bending condition exists at mid-span (Fig. 11(a)) while at the end of the span the condition is one of pure shear (Fig. 11(b)). At intermediate points a combination of these two stress patterns occurs and in some cases the combined effect may be more severe than either of the extreme conditions.

Pure Bending.—In the case of pure bending buckling may occur on the compression side of the web if the ratio d/t is excessive relative to the bending stress. The critical stress intensity may be expressed by Eq. (16) using k = 23.9. Therefore,

$$f_{cr} = \frac{64.9 \times 10^7}{(d/t)^2} \text{ lbs. per sq. in.} \tag{18}$$

in which, f_{cr} = critical stress at toe of flange angle
d = clear depth of web
t = thickness of web.

Above the proportional limit Eq. (18) must be modified by use of Eq. (17) in the procedure described previously. The value of k used above is for a plate simply supported on four edges. Actually, the flanges offer some restraint but the degree is uncertain and it is customary to assume the more severe simply supported case. Also, k varies with the ratio a/d but the value used above is a minimum which is not greatly exceeded in practical cases (2g). Eq. (18), although slightly conservative, is sufficiently accurate for design purposes. Values of critical stresses for the pure bending case are plotted in Fig. 12.

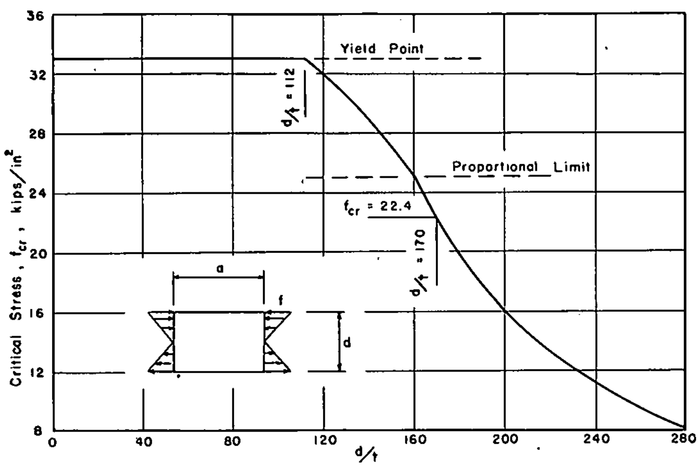

FIG. 12 CRITICAL STRESS FOR SIMPLY SUPPORTED PLATE IN BENDING

Pure Shear. Considering now the case of a simply supported plate in pure shear, Eq. (16) may again be used but in this case the variation of k with the ratio d/a may not be ignored. The critical stress in the elastic range for a plate in pure shear may be expressed by,

$$S_{cr} = \frac{k\ 27.1 \times 10^6}{(d/t)^2} \text{ lbs. per sq. in.} \tag{19}$$

where, $k = 5.35 + 4\left(\frac{d}{a}\right)^2 \qquad \frac{d}{a} < 1$

$k = 4 + 5.35\left(\frac{d}{a}\right)^2 \qquad \frac{d}{a} > 1$

Stresses from this equation are plotted in Fig. 13 where it should be noted that the dimensions d and a are interchangeable.

It should be recognized that the buckling of a plate in shear is not the result of the shear directly but rather of the diagonal com-

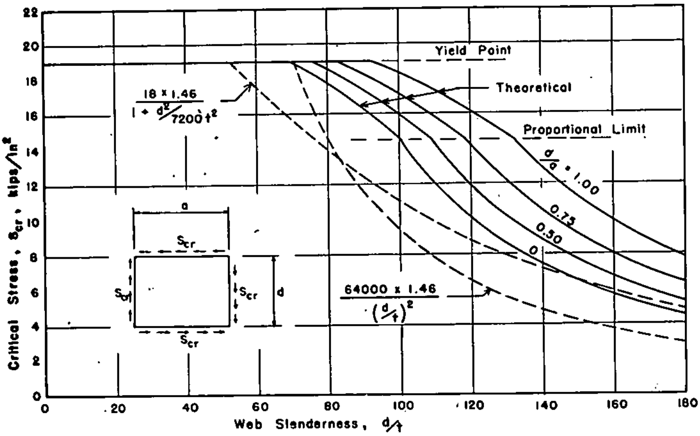

FIG. 13 CRITICAL STRESS FOR SIMPLY SUPPORTED PLATE IN PURE SHEAR

pression which accompanies the shear. For this reason the inelastic behavior of this case is more complex than those considered previously. Applying a recent theory to the problem (3e) it is assumed that the critical stress above the proportional limit is given by the equation,

$$S'_{cr} = S_{cr}\sqrt{\frac{E'}{E}} \tag{20}$$

in which S_{cr} is the hypothetical elastic value given by Eq. (19). The ratio E'/E is given by Eq. (3) in which it is assumed that

$$f'_{cr} = S'_{cr}\sqrt{3}$$

The plasticity theory on which this procedure is based indicates that the proportional limit and yield point for carbon steel in shear are 14.5 kips per sq. in. and 19.0 kips per sq. in. respectively. These values may be open to question.

The critical stress intensities in the inelastic region as given by Eq. (20) are plotted in Fig. 13.

Combined Shear and Bending. The problem of the stability of a plate in combined shear and bending was first solved by Timoshenko (2). A very good approximation to this solution (3f) is given by the interaction formula,

$$\left(\frac{S_{cr}}{S^{\circ}_{cr}}\right)^2 + \left(\frac{f_{cr}}{f^{\circ}_{cr}}\right)^2 = 1 \qquad (21)$$

in which S°_{cr} and f°_{cr} are critical stresses in pure shear and bending and S_{cr} or f_{cr} are critical stresses in the combined case. Thus, in a given case for which S°_{cr} and f°_{cr} are known and in which there is a certain tensile stress, the critical shear stress at which buckling occurs may be determined by Eq. (21).

Specifications

Web Slenderness. Most specifications require that the web slenderness, d/t, should not exceed 170. The purpose of this requirement is to prevent web buckling due to bending stresses (see Fig. 11(a)). Unfortunately it makes no allowance for the combined effect of bending and shear.

The theoretical solution for the case of pure bending is given by Eq. (18) and plotted in Fig. 12 where it may be observed that a web slenderness of 170 indicates a critical stress intensity of 22.4 kips per sq. in. At first thought the specification appears to be unconservative because in the case of flange elements (see above) the required slenderness ratios were such that buckling could not occur at stresses below the yield point. In this instance, however, a lower critical stress is permissible for two reasons: (1) the actual stress involved is that at the toe of the flange angle rather than the maximum bending stress, and (2) buckling of the web due to bending stresses does not necessarily imply failure of the girder. If the maximum design bending stress is 20 kips per sq. in. the stress at the toe of the flange angle is about 16 kips per sq. in. in the average girder. Apparently a slenderness ratio of 170 provides a factor of safety against buckling of about $\frac{22.4}{16} = 1.4$. Although somewhat lower than normal, the factor of safety is adequate because if the web were to buckle its stress would simply be transferred to the flange. Since the web resists only a small portion of the bending moment, the resulting increase in flange stress would be small and the factor of safety for bending in the whole girder would not be appreciably reduced.

An important defect in the specifications lies in the fact that when shear accompanies the maximum bending stress the critical d/t ratio may be considerably reduced. This stress combination may occur in a simple span girder subjected to concentrated loads or in a continuous girder when the points of maximum shear and moment may be the same. This condition may be investigated by Eq. (21) which may be written,

$$S_{cr} = \sqrt{\frac{E'}{E}} \quad S^{\circ}_{cr}\sqrt{1 - \left(\frac{f_{cr}}{f^{\circ}_{cr}}\right)^2} \tag{22}$$

Consider a given case in which d/t and a/t are known (a = stiffener spacing). and in which the bending stress is the maximum allowed. S°_{cr} and f°_{cr} may then be determined by Eqs. (18) and (19) assuming purely elastic behavior. Letting $f_{cr} = 22.4$ (bending stress intensity times factor of safety) Eq. (22) may be solved for S_{cr}, the shear stress at which buckling occurs. The value of $\sqrt{\frac{E'}{E}}$ is given by Eq. (3) in which f'_{cr} may be taken as $\sqrt{22.4^2 + 3\ S^2_{cr}}$. The results of this procedure, which involves a trial-and-error solution in the inelastic range, are shown in Fig. 14.

Referring to Fig. 14, when there is zero shear the required d/t ratio is 170 as provided by the specifications which implies a factor of safety against buckling of 1.4. As the shear increases the d/t ratio must decrease in order to maintain this factor of safety. The reduction depends upon d/a since the critical shearing stress, S°_{cr}, also depends on this ratio. If the actual bending stress is less than the allowable the web slenderness could of course be increased.

A separate calculation indicates that if both stresses are at the allowable values ($f = 16$ and $S = 13$ kips per sq. in.) and if the stiffener spacing is in accordance with specifications there is little or no factor of safety against buckling of the web. Although it would not mean failure of the girder, it is surprising to realize how close to buckling the webs of many girders must be. In cases where large shears and moments are in combination it is desirable to design the web slenderness and the stiffener spacing somewhat more conservatively than indicated by the specifications.

Vertical Stiffeners. The purpose of vertical stiffeners is to prevent buckling of the web due to shearing stresses. It is obvious, however, from the foregoing discussion that the required spacing depends upon bending as well as shear stresses—a fact ignored by specifications.

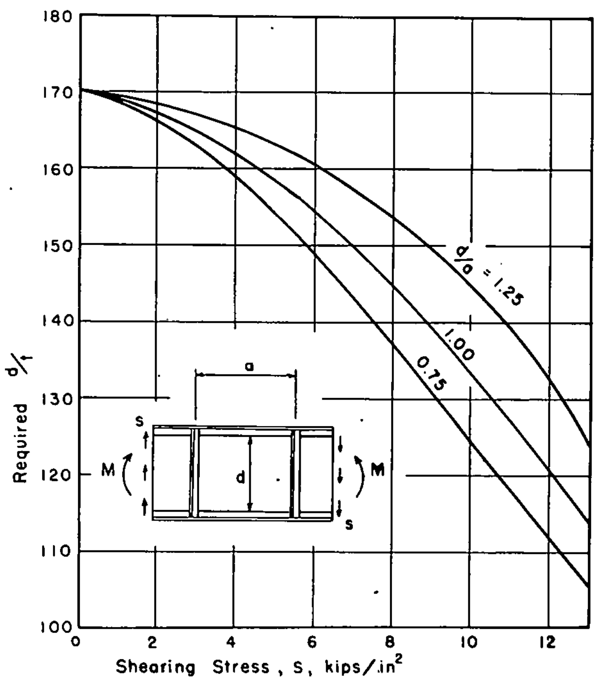

FIG. 14 REQUIRED WEB SLENDERNESS FOR COMBINED SHEAR AND MAXIMUM BENDING

Stiffeners are not required by specifications if the web slenderness $\left(\frac{d}{t}\right)$ is less than 60 or 70. Referring to Fig. 13 where theoretical critical shearing stresses are plotted, it may be observed that this requirement prevents buckling below the yield point for all values of d/a. This concept is the same as that used in connection with compression flange elements (see above) but differs from that used in determining required d/t ratio for webs in pure bending. It is entirely proper, however, to completely prevent the buckling of unstiffened web plates in shear because this would precipitate failure of the whole girder. This is not true of webs reinforced by stiffeners because, in that case, after buckling occurs the web carries diagonal tension opposed by compression in the stiffeners thus forming a trusswork capable of carrying the girder shear.

Since unstiffened webs are not permitted to buckle, the factor of safety in shear is based upon the yield point. For an allowable stress of 13 kips per sq. in. the factor of safety is 19/13 or 1.46.

If the shear stress in the girder is less than the maximum allowed, stiffeners may be omitted at higher d/t ratios. Two formulas which are used to relate the shear to the maximum permissible web slenderness are,

$$\text{(AISC)} \quad S = \frac{64000}{(d/t)^2} \text{ kips per sq. in.} \tag{23}$$

$$\text{(Boston Code)} \quad S = \frac{18}{1 + d^2/7200t^2} \text{ kips per sq. in.} \tag{24}$$

These equations multiplied by the factor of safety in shear are plotted in Fig. 13. Eq. (23) has the correct form for elastic buckling (see Eq. 19) although it neglects the effect of the d/a ratio. Eq. (24), although empirical, has approximately the correct shape when plotted. Both of these equations appear to be conservative. However, it should be borne in mind that the presence of bending stresses would reduce the theoretical values shown in Fig. 13.

The equation for required stiffener spacing may be obtained from Eq. (19).

Substituting for k and solving,

$$\frac{a}{t} = \frac{1.15}{\sqrt{\dfrac{S_{cr}}{10.84 \times 10^7} - \left(\dfrac{t}{d}\right)^2}} \tag{25a}$$

$$\text{for } \frac{d}{a} > 1$$

$$\text{and, } \frac{a}{t} = \frac{0.865}{\sqrt{\dfrac{S_{cr}}{14.5 \times 10^7} - \left(\dfrac{t}{d}\right)^2}} \tag{25b}$$

$$\text{for } \frac{d}{a} > 1$$

where a is the required clear distance between stiffeners for a shear stress of S_{cr} in lbs. per sq. in. In the inelastic range S_{cr} is replaced by $S_{cr}\sqrt{\frac{E}{E'}}$. Results obtained from these equations are plotted in Fig. 15. A factor of safety has been included by replacing S_{cr} with 1.46 x S in Eq. (25).

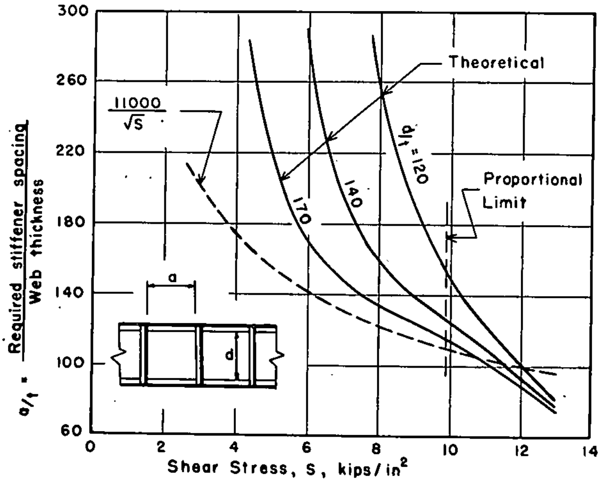

FIG. 15 REQUIRED STIFFENER SPACING IN PLATE GIRDERS

Typical of specifications is the AISC formula for stiffener spacing,

$$\frac{a}{t} = \frac{11000}{\sqrt{S}} \tag{26}$$

If t/d in Eqs. (25) is neglected those equations become identical in form to Eq. (26). The specification formula is plotted in Fig. 15 together with the theoretical values. This formula is conservative for low stresses because of the omission of the t/d term. The importance of this term is reflected by the difference between the three theoretical curves in Fig. 15. There is no apparent reason for neglecting the effect of d/t in the specification. This is another example of how the desire to simplify design procedures may be carried to extreme.

As shown in Fig. 15 the specifications become slightly unconservative for high shear stresses. This is due to the fact that Eq. (26) does not take into account the decrease in modulus above the proportional limit as do the theoretical curves.

As noted previously, the presence of bending stresses increases the probability of web buckling and should decrease the required stiffener spacing. Since the specifications do not recognize this fact, the factor of safety in shear is apparently reduced in such cases. We may investigate this condition by applying Eq. (22) in the same manner as used in the derivation of Fig. 14. For particular values of d/t, a/t, and the bending stress f_{cr}, Eq. (22) may be solved for the critical shearing stress. In Fig. 16 it is assumed that the shearing stress and the web slenderness $(\frac{d}{t})$ are the maximum values allowed. The stiffener spacing ratio $(\frac{a}{t})$ is the theoretical required value as given by Eq. (25) neglecting the effect of bending. At a particular bending stress the factor of safety in shear is the critical shear stress computed as described above, divided by 13.

Fig. 16 shows that the factor of safety against buckling decreases

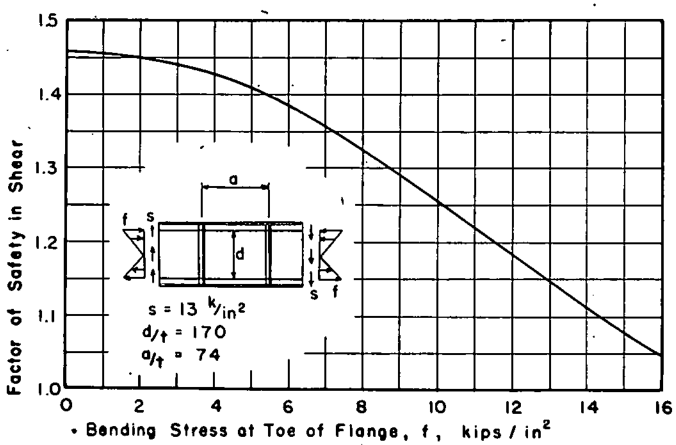

FIG. 16 REDUCTION. IN FACTOR OF SAFETY DUE TO COMBINED BENDING AND SHEAR

steadily as the bending stress increases. Specifications are therefore unconservative when shear and bending stresses are acting together. This is the same conclusion as was stated in the discussion of web slenderness.

SUMMARY

This paper considers the various buckling phenomena which may occur in steel beams and plate girders and which must be considered in the design of such members. Three types of buckling are included: (1) lateral buckling of whole member, (2) local buckling of compression flange elements, and (3) buckling of the web plate. In each case the available theory is reviewed and current design procedures evaluated on that basis.

When lateral buckling is considered it is found that only one of the several design formulas in use is based directly on theory. None of the specifications take into account the rather appreciable effect of type of loading and lateral support on the critical stress.

In the case of local buckling of compression elements it is customary to limit the width-thickness ratio so that no buckling may occur at a stress below the yield point. In general this criteria is sound but it should be permissible to use larger width-thickness ratios when the member is not fully stressed.

The factor of safety included in the design of web plates is somewhat less than that used elsewhere. In particular, the specifications fail to consider the rather severe effect of combined bending and shear.

In conclusion it may be observed that because of a superficial treatment of the buckling problem the specifications place severe restrictions on the designer. The policy of making design procedures as simple as possible has been carried to extreme. The time has come when the designer must be given more freedom and he in turn must be prepared to use more elaborate methods of design.

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1884 - 1953

Everett Nelson Hutchins was born in the Hutchins homestead in Dover, New Hampshire, on August 18, 1884. He was the son of John W. Hutchins, former headmaster and head of the science department of Malden High School, and Ardelia M. Perkins. He died October 26, 1953. He is survived by his wife and daughter.

He moved with his family to Malden, Mass., in 1888 and lived there until he made Melrose his home in 1932. He graduated from Harvard University with an A.B. degree in 1908 and a B.S. degree in

Civil Engineering in 1909. He married Marion A. Crocker in 1919. They had one daughter, Ardelia (Mrs. Victor Lee Hamke).

Mr. Hutchins' entire professional career was spent in public service. He first worked with the Charles River Basin Commission in Boston, followed by two years with the New York City Board of Water Supply. During this period he was associated with the design and construction of the Rondout and Peak tunnel near High Falls, N. Y. There followed employment with the city of Pittsfield, Mass., on the Farnum Dam and Reservoir.

In 1913 Mr. Hutchins joined the engineering staff of the Commonwealth of Massachusetts where he served for a period of forty years in a number of capacities. He was in charge of many state projects including the design and construction of the drydock in South Boston, piers, wharves and bulkheads, and projects for shore protection along the coast of Massachusetts including sea walls, breakwaters, and jetties.

In 1942 he was made District Waterways Engineer where he had the responsibility for the engineering work in the Division of Waterways in the Department of Public Works. This work included the expansion of the Logan International Airport at East Boston, shore protection projects, river and harbor improvements, maintenance and operation of state piers, projects involving stream clearance and studies of water resources.

His professional work was characterized by sound judgment, careful attention to details, unquestioned integrity, and boundless energy.

Mr. Hutchins was elected a member of the Boston Society of Civil Engineers in 1924 and became secretary of the society and editor of the society JOURNAL in 1929 in which capacities he served for 18 years. He was then elected vice president for 2 years but, because of ill health, he was unable to continue to the presidency. He was a member of the Harvard Engineering Society, the Harvard Club of Eastern Middlesex, and the Massachusetts State Engineering Society. He was clerk of the First Congregational Church in Malden for five years and of the Melrose Highlands Congregational Church for a period of years. He served as a member of the Malden Planning Board and as trustee of the Malden Public Library.

Mr. Hutchins' life was one of unselfish devotion to his work, his community, his family, and to his host of friends.

Of General Interest

NEWS OF MEMBERS

Professor Emil A. Gramstorff, chairman of the Department of Civil Engineering at Northeastern University, was appointed Dean of the Graduate Division of the College of Engineering and Professor Charles O. Baird, Jr. became the new head of Civil Engineering, in an announcement by President Carl S. Ell.

Gramstorff and Baird became members of the Northeastern faculty in 1921 and 1922, respectively, and are well known in engineering circles throughout New England and other sections of the country.

The new dean is replacing Dr. Herbert K. Brown, former dean of the Graduate Division who after an extended illness has requested a full-time teaching assignment in the Department of Mechanical Engineering.

Professor Kenneth C. Reynolds, Head, Department of General Engineering, University of Southern California, has received a Fulbright award as a Lecturer in Civil Engineering at the College of Engineering in Baghdad, Iraq, for the academic year October 1954 to June 1955.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETING

STRUCTURAL SECTION

June 2, 1954.—Chairman Edward W. Moore called the meeting to order at 7:10 P.M. at the Society Rooms after an informal dinner at Patten's Restaurant. Eighteen members and guests attended the dinner and thirty-eight members and guests attended the meeting.

Chairman Moore announced that this meeting was in lieu of the Annual Outing, inasmuch as the Executive Committee had decided there was no suitable site for an outing at this time. The Chairman also asked for any suggestions or comments as to future policy of the Section in connection with a June meeting.

The speaker of the evening was Charles A. Turner, Chief Mechanical Engineer of Thomas Worcester Co., Inc., and he presented a paper entitled "Problems of a Metropolitan District System of Refuse Disposal Incinerators". He discussed the report presented by his firm to the Metropolitan District Commission in reference to setting up sufficient number of incinerators to take care of the whole metropolitan Boston area, the locations of these incinerators, the approximate cost thereof, and other pertinent data relative to the establishment of incinerators throughout the metropolitan district area. At the conclusion of the paper there was a lively and lengthy discussion relative to the proposed incinerators and their functionings.

The meeting was adjourned at 8:20 P.M.

Darrell A. Root, *Clerk*

DEATHS

Robinson Abbott, June 18, 1954
Karl T. Compton, June 22, 1954
William N. Parsons, June 4, 1954
Walton H. Sears, August 7, 1954
Frank H. Stuart, July 28, 1954

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