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FIRE TESTS OF BUILDING COLUMNS

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PREFACE

The investigation herein reported was conducted in cooperation with the Associated Factory Mutual Fire Insurance Companies and the National Board of Fire Underwriters, the tests being made at Underwriters' Laboratories, Chicago, Ill.

When the assistance of this Bureau on certain portions of the work was sought during the early part of 1914, the preparatory work had been in progress for about four years, a building adapted for the tests had been erected, and the design of equipment and test columns nearly completed.

As the safety of building supports is a matter of general public interest, participation in the proposed investigation appeared justified, and a basis of cooperation was arranged whereby the work was made a part of the investigation on the fire-resisting properties of building materials that was then being initiated by this Bureau.

By the method of presentation that follows, Section I gives a general outline of the tests; Sections II, III, and IV give descriptions of the columns, column coverings, and methods used in their preparation; Section V and Appendix D, results of auxiliary tests of materials; Sections VI, VII, VIII, and IX, descriptions of apparatus and method of testing; Sections X and XI and Appendices A, B, C, results of tests; Sections XII and XIII, discussion of test data and conclusions.

The full presentation of the results of the present investigation is necessarily of considerable extent, and familiarity with the general plan will be of material aid in reading and, if desired, in selecting for particular attention such portions as may best serve the reader's immediate purpose.

FIRE TESTS OF BUILDING COLUMNS

By S. H. Ingberg, H. K. Griffin, W. C. Robinson, and R. E. Wilson

CONTENTS

	Page
I. Introduction.....	12
1. Purpose.....	12
2. Scope of tests.....	12
3. Acknowledgment.....	13
II. Description of columns.....	15
1. Structural-steel columns.....	15
(a) Details of design.....	15
(b) Bearings.....	20
(c) Riveting.....	20
(d) Initial straightness.....	20
(e) Properties of the sections.....	20
2. Cast-iron columns.....	21
(a) Details of design and manufacture.....	21
(b) Bearings.....	23
(c) Initial straightness.....	23
(d) Variations in metal thickness.....	23
3. Pipe columns.....	23
(a) Details of design and manufacture.....	23
(b) Bearings.....	25
4. Reinforced-concrete columns.....	25
(a) Details of design.....	25
(b) Bearings and end restraint.....	25
5. Timber columns.....	28
(a) Species of timber.....	28
(b) Cap and bearing details.....	28
(c) Properties of the timber.....	28
III. Schedule of tests.....	30
1. General considerations.....	30
(a) Object and limitations.....	30
(b) Preliminary work on schedule.....	31
2. Schedule of fire tests.....	32
(a) Unprotected columns.....	32
(b) Partly protected columns.....	32
(c) Columns protected by plaster on metal lath.....	32
(d) Columns protected by concrete.....	37
(e) Columns protected by hollow clay tile.....	41
(f) Columns protected by gypsum block.....	45
(g) Columns protected by brick.....	47
(h) Reinforced-concrete columns.....	48
(i) Timber columns.....	49

III. Schedule of tests—Continued	Page
3. Schedule of fire and water tests.....	50
(a) Columns protected by concrete.....	50
(b) Columns protected by hollow clay tile.....	50
(c) Columns protected by gypsum block.....	50
(d) Plaster on metal-lath protection.....	50
(e) Reinforced-concrete columns.....	50
(f) Unprotected cast-iron columns.....	50
IV. Placing of coverings and concrete columns.....	54
1. Concrete protections and columns.....	54
(a) Forms and staging.....	54
(b) Method of proportioning.....	55
(c) Mixing and placing.....	55
2. Plaster on metal-lath protections.....	56
(a) Placing of lath.....	56
(b) Applying the plaster.....	56
3. Hollow clay-tile and brick protections.....	56
(a) Proportioning of mortar.....	56
(b) Placing of tile and brick.....	56
(c) Placing of concrete filling.....	56
4. Gypsum-block protections.....	57
(a) Proportioning of mortar.....	57
(b) Placing of block.....	57
(c) Placing of filling.....	57
5. Method of storage.....	57
V. Auxiliary tests of materials.....	58
1. Tests of structural, bar, and wire steel.....	58
2. Tests of cast iron.....	59
3. Tests of Portland cement.....	60
4. Tests of sand.....	60
5. Tests of coarse concrete aggregates.....	61
6. Source and classification of concrete aggregates.....	61
(a) Fox River sand.....	61
(b) Joliet sand.....	61
(c) Meramec River sand.....	62
(d) Long Island sand.....	62
(e) Pelee Island sand.....	62
(f) Plum Island sand.....	62
(g) Chicago limestone.....	63
(h) Joliet gravel.....	63
(i) Meramec River gravel.....	63
(j) New York trap rock.....	63
(k) Rockport granite.....	63
(l) Cleveland sandstone.....	64
(m) Hard-coal cinders.....	64
7. Tests of concrete.....	64
(a) Test specimens.....	64
(b) Per cent water in concrete mixture.....	65
(c) Testing of concrete cylinders.....	67
(d) Compressive strength of concrete.....	67
(e) Modulus of elasticity of concrete.....	70

V. Auxiliary tests of materials—Continued	Page
8. Tests of lime	72
9. Tests of calcined gypsum	72
10. Tests of mortar, plaster, and filling	72
11. Tests of hollow clay tile	73
(a) Classification and description	73
(b) Porosity and absorption	73
(c) Compressive and transverse strength	73
(d) Temperatures of vitrification and fusion	73
12. Tests of brick	74
13. Tests of gypsum block	74
14. Tests of gypsum wall board	74
VI. Description of furnace and related equipment	75
1. Building	75
2. Apparatus for handling columns	75
3. Loading apparatus	75
(a) Loading ram	75
(b) Controlling devices	77
(c) Restraining frame	79
(d) Bearing details	79
(e) Capacity and calibration	79
4. Testing furnace	79
(a) Combustion chamber	79
(b) Burners	80
(c) Operating details	80
5. Fire-stream apparatus	81
VII. Temperature measurements	81
1. Instruments	81
(a) Indicating potentiometer	81
(b) Recording potentiometer	83
(c) Accessories	83
2. Furnace temperatures	83
(a) Location of furnace thermocouples	83
(b) Thermocouple mountings	85
(c) Connections to instruments	85
3. Column temperatures	87
(a) Attachment of thermocouples to column	87
(b) Location of column thermocouples	88
(c) Connections to instruments	89
4. Method of taking and reducing observations	90
5. Calibration of thermocouples	91
(a) Furnace thermocouples	91
(b) Column thermocouples	91
VIII. Deformation measurements	92
1. General outline	92
(a) Attachment and protection of wires	93
2. Unit compression and expansion	93
(a) Micrometers and mounting	93
(b) Method of taking and reducing observations	93
3. Center deflection	95
4. Total depression or expansion	95
5. Calibration and accuracy	96

	Page
IX. Method of testing.	96
1. Loading formulas and applied loads.	97
(a) Working loads.	97
(b) Loading to failure.	98
2. Fire exposure.	98
(a) Character of fire.	98
(b) Preliminary panel tests.	98
(c) Time-temperature curve.	99
(d) Influence of pyrometer mounting on indicated temperatures.	101
(1) Temperature lag.	101
(2) Radiation effects.	105
3. Fire and water test procedure.	106
4. Observations during test.	106
5. Observations after failure.	107
6. Photographic records.	107
X. Results of fire tests.	107
1. Fire-test results in tables and figures.	107
2. Photographic records.	107
3. Furnace temperatures.	115
(a) Variations from average curve.	115
4. Column temperatures.	115
(a) Temperature variation over length of column.	115
(b) Temperature variation over cross section.	115
(c) Dehydration points.	116
5. Longitudinal deformation and average temperature.	117
(a) Computation of average effective temperature.	117
(b) Deformation under heat and load.	119
(c) Period of expansion.	120
(d) Maximum column temperatures.	120
6. Total vertical deformation.	123
(a) Before failure.	123
(b) At failure.	125
7. Lateral deflection.	125
(a) Before failure.	125
(b) Deflection at failure.	125
8. Log of fire tests.	127
(a) Unprotected columns.	128
(b) Partly protected columns.	129
(c) Plaster on metal-lath protections.	130
(d) Concrete protections.	131
(e) Hollow clay-tile protections.	136
(f) Gypsum-block protections.	143
(g) Brick protections.	146
(h) Reinforced-concrete columns.	147
(i) Timber columns.	148
XI. Results of fire and water tests.	151
1. Applied loads, duration and effect of fire and water.	151
2. Photographic records.	152
3. Furnace and column temperatures.	152
4. Longitudinal deformation.	152
5. Subsequent loading tests.	152

XI. Results of fire and water tests—Continued	Page
6. Log of fire and water tests.....	152
(a) Concrete protections.....	152
(b) Hollow clay-tile protections.....	154
(c) Gypsum-block protections.....	156
(d) Plaster on metal-lath protection.....	157
(e) Reinforced-concrete columns.....	158
(f) Unprotected cast-iron columns.....	160
XII. General summary and discussion.....	161
1. Characteristics of columns and their materials.....	161
(a) Structural-steel columns.....	161
(1) Material and fabrication.....	161
(2) Effect of slenderness ratio.....	161
(3) Lateral deflection.....	162
(4) Vertical deformation.....	162
(5) Load carried by covering.....	162
(6) Average effective temperatures.....	162
(7) General cause of failure.....	163
(b) Cast-iron columns.....	163
(1) Material and manufacture.....	163
(2) Deformation and temperature.....	164
(3) Cause and character of failure.....	164
(c) Pipe columns.....	164
(1) Material and manufacture.....	164
(2) Deformation and temperature.....	165
(d) Reinforced-concrete columns.....	165
(1) Mechanical properties of the concrete.....	165
(2) Deformation and temperature.....	166
(3) Character of failure.....	166
(e) Timber columns.....	166
(1) Quality of material.....	166
(2) Deformation and temperature.....	166
(3) Cause of failure.....	167
2. Useful limits.....	167
(a) Structural-steel and cast-iron columns.....	167
(b) Pipe columns.....	168
(c) Reinforced-concrete columns.....	168
(d) Timber columns.....	169
(e) Practical application.....	169
3. Discussion of test data.....	171
(a) Difference in columns, test exposure, and service conditions.....	171
(1) Variations due to difference in columns.....	171
(2) Variations due to difference in load and fire conditions.....	172
(b) Unprotected columns.....	173
(1) Structural-steel.....	173
(2) Cast-iron.....	174
(3) Pipe columns.....	175
(c) Partly protected columns.....	175
(1) Effect of section and size.....	175
(2) Effect of concrete aggregate and ties.....	175

XII. General summary and discussion—Continued	Page
(d) Plaster on metal-lath protections.	176
(1) Material and design.	176
(2) Test results.	177
(3) Cracking due to expansion of covering.	177
(4) Effect of variation in details of application.	178
(5) Effect of water application.	178
(e) Concrete protections.	178
(1) Mechanical properties of the concrete.	178
(2) Function of concrete as a covering material.	179
(3) Variations due to concrete aggregate.	179
(4) Comparison of 2 and 4 inch protections.	181
(5) Effect of size.	182
(6) Effect of strength of concrete.	182
(7) Influence of shape of section and covering.	183
(8) Function of the wire tie.	183
(9) Effect of water application.	184
(f) Hollow clay-tile protections.	184
(1) Mechanical properties of the tile.	184
(2) Test results.	184
(3) Variations due to type of clay and details of manufacture.	185
(4) Comparison of 2 and 4 inch protections.	185
(5) Effect of size.	186
(6) Effect of ties and filling.	186
(7) Effectiveness of plastering.	188
(8) Effect of water application.	188
(g) Brick protections.	189
(1) Properties of the brick.	189
(2) Test results.	189
(h) Gypsum-block protections.	189
(1) Strength and porosity of the gypsum block.	189
(2) Comparison of 2 and 4 inch protections.	189
(3) Characteristic fire effects.	190
(4) Heat-insulating properties.	191
(5) Effect of water application.	191
(i) Reinforced-concrete columns.	191
(1) Influence of concrete aggregate.	192
(2) Effect of form of column and reinforcement.	192
(3) Recovery of strength after fire test.	192
(4) Effect of water application.	193
(j) Timber columns.	193
(1) Unprotected timber columns.	193
(2) Protected timber columns.	194
(3) Strength after fire test.	195
XIII. Fire-resistance periods derived from the test results	196
1. Basis of derivation.	196
(a) Method of computation.	196
(b) Intervals.	196
(c) Table of fire-resistance periods.	196
(d) Derivation of method.	196
(e) Resistance to water application.	200
(f) Size limitations.	200
(g) Application to building conditions.	200

XIII. Fire-resistance periods derived from the test results—Continued	Page
2. Derivation of fire-resistance periods.....	202
(a) Unprotected structural-steel columns.....	203
(b) Partly protected structural-steel columns.....	203
(1) Solid-section columns.....	203
(2) Open latticed section.....	204
(c) Structural-steel columns with plaster on metal-lath pro- tections.....	204
(1) Single-layer protection.....	204
(2) Double-layer protection.....	204
(d) Concrete protections on structural-steel columns.....	205
(1) Siliceous-gravel concrete protection.....	205
(2) Granite, sandstone, or hard-coal cinder concrete protection.....	205
(3) Trap-rock concrete protection.....	207
(4) Limestone or calcareous-gravel concrete pro- tection.....	207
(e) Hollow clay-tile protections on structural-steel columns..	209
(1) Unfilled protection.....	210
(2) Shale or surface-clay tile protection with con- crete filling reentrant spaces.....	210
(3) Semifire-clay or surface-clay tile protection with full concrete filling.....	210
(4) Double 2-inch tile protection.....	211
(f) Brick protections.....	211
(g) Gypsum-block protections.....	211
(h) Cast-iron columns.....	212
(1) Unprotected columns.....	212
(2) Plaster on metal-lath protection.....	212
(3) Concrete protection.....	212
(4) Hollow clay-tile protection.....	213
(i) Unprotected pipe columns.....	213
(j) Reinforced-concrete columns.....	213
(k) Timber columns.....	214
(1) Unprotected timber columns.....	214
(2) Protected timber columns.....	214
3. Conditions governing fire duration in buildings.....	215
XIV. Appendixes:.....	216
Appendix A. Views ^a of columns before and after test—Figs. 58 to 89.....	216-248
Appendix B. Time-temperature curves ^a —Figs. 90 to 145.....	249-306
Appendix C. Deformation and average temperature curves ^a — Figs. 146 to 171.....	307-333
Appendix D. Tables ^a of auxiliary tests of materials—Tables 5 to 40.....	334-364
Appendix E. Previous investigations ^a	365-374
Appendix F. Centigrade and Fahrenheit conversion table.....	375

^a See contents, in detail, given at beginning of the appendix.

TABLES

	Page
1. Nominal and measured areas of structural steel sections.	21
2. Properties of timber in test columns.	30
3a. Schedule of fire tests. Unprotected structural-steel columns.	33
3b. Schedule of fire tests. Unprotected cast-iron and pipe columns.	34
3c. Schedule of fire tests. Columns partly protected by concrete.	35
3d. Schedule of fire tests. Columns protected by plaster on metal lath.	36
3e. Schedule of fire tests. Columns protected by concrete.	38-40
3f. Schedule of fire tests. Columns protected by hollow clay tile.	42-44
3g. Schedule of fire tests. Columns protected by gypsum block.	46
3h. Schedule of fire tests. Columns protected by brick.	47
3i. Schedule of fire tests. Reinforced-concrete columns.	48
3j. Schedule of fire tests. Timber columns.	49
4a. Schedule of fire and water tests. Columns protected by concrete.	51
4b. Schedule of fire and water tests. Columns protected by hollow clay tile. .	52
4c. Schedule of fire and water tests. Columns protected by gypsum block. . .	52
4d. Schedule of fire and water tests. Column protected by plaster on metal lath.	53
4e. Schedule of fire and water tests. Reinforced-concrete columns.	53
4f. Schedule of fire and water tests. Unprotected cast-iron columns.	54
5 to 40. Auxiliary tests of materials. Appendix D.	334-364
41. Computed and applied working loads.	97
42a. Results of fire tests. Unprotected columns.	108
42b. Results of fire tests. Columns partly protected by concrete.	109
42c. Results of fire tests. Columns protected by plaster on metal lath.	109
42d. Results of fire tests. Columns protected by concrete.	110
42e. Results of fire tests. Columns protected by hollow clay tile.	111
42f. Results of fire tests. Columns protected by gypsum block.	112
42g. Results of fire tests. Columns protected by brick.	112
42h. Results of fire tests. Reinforced-concrete columns.	113
42i. Results of fire tests. Timber columns.	113
43. Time to failure, period of expansion, and maximum column temperatures	122-123
44. Results of fire and water tests.	facing 153
45. Compressive strength of timber after fire test.	195
46. Fire-resistance periods derived from the test results.	197-199

FIGURES

1. Details of structural-steel columns. Rolled H and plate and angle sections.	16
2. Details of structural-steel columns. Plate and channel and latticed chan- nel sections.	17
3. Details of structural-steel columns. Z bar and plate and I beam and chan- nel sections.	18
4. Details of structural-steel columns. Latticed angle and starred angle sec- tions.	19
5. Calipers for measuring steel shapes.	facing 20
6. Details of cast-iron columns.	22
7. Details of pipe columns.	24
8. Details of vertically reinforced-concrete columns.	26
9. Details of hooped concrete column and column-head protection.	27
10. Details of timber columns.	29
11. Forms and staging for placing concrete.	facing 54
12. Concrete mixer.	facing 55
13. Placing of clay-tile and gypsum-block protections.	facing 55

Fire Tests of Building Columns

II

FIGURES—Continued

	Page
14. Concrete cylinder after test.....	65
15. Average and range of compressive strength of 1 : 2 : 4 and 1 : 2 : 5 concrete..	66
16. Average and range of compressive strength of 1 : 3 : 5 concrete.....	67
17. Effect of consistency on compressive strength of concrete; average age, 28 days.....	68
18. Effect of consistency on compressive strength of concrete; average age, 490 days.....	68
19. Effect of time of mixing on compressive strength of concrete.....	69
20. Modulus of elasticity of concrete at 450 to 850 pounds per square inch....	70
21. Modulus of elasticity of 1 : 2 : 4 concrete at 650 pounds per square inch. Variation with ultimate compressive strength.....	71
22. Effect of consistency on modulus of elasticity of concrete at 650 pounds per square inch.....	71
23. Average and range of compressive strength of mortar and plaster.....	72
24. Plan of testing room.....	76
25. Elevation of testing machine.....	78
26. General view of testing machine.....	facing 80
27. Control board.....	facing 81
28. Fire-stream apparatus.....	facing 81
29. Temperature-measuring instruments.....	facing 81
30. Wiring diagram of indicating potentiometer.....	82
31. Location of furnace and column thermocouples.....	84
32. Detail of furnace thermocouple mounting.....	86
33. Diagram showing method used for measuring deformation.....	92
34. Testing furnace with furnace couples, column, and deformeter in place..	facing 93
35. Detail of insulating tube and insert.....	94
36. Apparatus for measuring deformation.....	facing 95
37. Furnace temperatures, preliminary panel tests.....	99
38. Preliminary tile panels after test.....	facing 99
39. Comparison of furnace temperatures used in fire tests.....	100
40. Lag correction for furnace pyrometers.....	102
41. Effect of radiation on indication of furnace pyrometers.....	104
42. Time to failure of columns in fire-test series.....	114
43. Temperature variation over cross section of typical columns.....	116
44. Assumed temperature variation between thermocouple points.....	117
45. Expansion period of steel, cast iron, and concrete columns in fire-test series.	121
46. Total expansion, test Nos. 10A, 76, 77, 110, 114, and 115.....	124
47. Depression of top of timber columns.....	126
48. Results of fire tests compared by groups.....	170
49. Effect of load on fire resistance, unprotected structural-steel columns....	174
50. Effect of size, partly protected columns.....	176
51. Comparison of 2-inch and 4-inch concrete protections.....	181
52. Effect of size, concrete protections.....	182
53. Comparison of 2-inch and 4-inch hollow clay-tile protections.....	185
54. Effect of size, clay tile and brick protections.....	186
55. Effect of ties and filling, hollow clay-tile protections.....	187
56. Comparison of 2-inch and 4-inch gypsum-block protections.....	190
57. Blocks from gypsum coverings after test.....	facing 191
58 to 89. Views of columns before and after test. Appendix A.....	217-248
90 to 145. Time-temperature curves. Appendix B.....	308-333
146 to 171. Deformation and average temperature curves. Appendix C....	251-306

I. INTRODUCTION

1. PURPOSE

The purpose of this investigation is to ascertain (1) the ultimate resistance against fire, of protected and unprotected columns as used in the interior of buildings; (2) their resistance against impact and sudden cooling from hose streams when in a highly heated condition.

While columns form the most important element in the strength of a building, few representative tests have been made to determine their ability to support load when exposed to fire, and fire experience has only a limited value due to the many unknown variables involved. As a consequence, wide differences in requirements relating to the protection of columns against fire exist between different municipal codes and other published regulations. This investigation was undertaken to obtain information on which proper requirements for the more general types of columns and protective coverings can be based.

2. SCOPE OF TESTS

The present series consists of 106 tests of columns, of which 91 were fire tests and 15 fire and water tests.

The fire test series includes (1) tests of representative types of unprotected structural steel, cast iron, concrete-filled pipe, and timber columns; (2) tests wherein the metal was partly protected by filling the reentrant portions or interior of columns with concrete; (3) tests wherein the load-carrying elements of the columns were protected by 2-inch or 4-inch thicknesses of concrete, hollow clay tile, clay brick, gypsum block, and also single or double layer of metal-lath and plaster; (4) reinforced-concrete columns with 2-inch integral concrete protection.

The covering materials for each class of protection were obtained from the main producing regions of the country, the object being to include samples from the principal mineralogical subdivisions that find general application in building construction. A large number of auxiliary tests of constituent materials were made, including several hundred compression tests of the concrete employed.

The test columns were designed for a working load of approximately 100 000 pounds, as calculated according to accepted formulas, the amount varying somewhat for the different sections.

The load was maintained constant on the column during the test, the efficiency of the column or its covering being determined by the length of time it withstood the combined load and fire exposure.

The latter was produced by placing the column in the chamber of a gas-fired furnace whose temperature rise was regulated to conform with a predetermined time-temperature relation. Measurements were taken of the temperature of the furnace and test column and of the deformation of the latter due to the load and heat.

In the fire and water tests the column was loaded and exposed to fire for a predetermined period, at the end of which the furnace doors were opened and a hose stream applied to the heated column, the duration of the application and the pressure at the nozzle varying with the length of time the corresponding type of column withstood the regular fire tests.

3. ACKNOWLEDGMENTS

The tests were jointly conducted by the Associated Factory Mutual Fire Insurance Companies, the National Board of Fire Underwriters, and the Bureau of Standards. The fire tests were made at Underwriters' Laboratories, Chicago, Ill. The furnace and related equipment were designed and constructed by Underwriters' Laboratories during the period 1912 to 1917, the work being coordinated with a general building program that was carried out by them at that time. Its use, except for repairs and replacements, was donated for the tests. Preliminary work relative to the testing schedule was begun as early as 1910 by the Associated Factory Mutual Fire Insurance Companies and Underwriters' Laboratories, the cooperation of this Bureau beginning in 1914. A final schedule of tests was adopted in March, 1916. The preparatory work of covering and placing the test columns extended from May, 1916, to May, 1917. With the completion of the equipment, testing of columns began in June, 1917, and was completed in December, 1918. This paper in its final form was approved by the cooperating parties in December, 1920.

The apparatus for measuring temperature and deformation was supplied by this Bureau. Auxiliary physical and chemical tests of the materials of the test columns and their protective coverings were made at the Washington and Pittsburgh laboratories of the Bureau.

The expense for labor and material was shared on an equal basis by the three cooperating units. The materials were in part purchased under the usual specifications, and in part supplied gratis by the following companies, whose cordial cooperation is herewith acknowledged:

American Bridge Co., Chicago, Ill., steel test columns.
 Bethlehem Steel Co., South Bethlehem, Pa., steel test columns.
 Chicago Bridge & Iron Works, Chicago, Ill., steel test columns.
 R. D. Cole Manufacturing Co., Newnan, Ga., steel test columns.
 Pittsburgh-Des Moines Steel Co., Pittsburgh, Pa., steel test columns.
 U. S. Cast Iron Pipe & Foundry Co., Burlington, N. J., cast-iron test columns.
 Lally Column Co., Cambridge, Mass., steel-pipe test columns.
 National Lumber Mfrs. Assn., Chicago, Ill., timber test columns.
 Southern Pine Assn., New Orleans, La., timber test columns.
 West Coast Lumbermen's Assn., Seattle, Wash., timber test columns.
 Corrugated Bar Co., Buffalo, N. Y., reinforcing steel.
 Associated Metal Lath Mfrs. Chicago, Ill., expanded metal lath.
 Zander-Reum Co., Chicago, Ill., woven-wire lath.
 Chicago Portland Cement Co., Chicago, Ill., Portland cement.
 Marblehead Lime Co., Chicago, Ill., lime.
 Pelee Island Sand & Gravel Co., Cleveland, Ohio, sand.
 Phoenix Sand & Gravel Co., New York, N. Y., sand.
 American Sand & Gravel Co., Chicago, Ill., sand and gravel.
 Chicago Gravel Co., Chicago, Ill., sand and gravel.
 Union Sand & Material Co., St. Louis, Mo., sand and gravel.
 Haverstraw Crushed Stone Co., New York, N. Y., crushed stone.
 Ohio Quarries Co., Cleveland, Ohio, crushed stone.
 Rockport Granite Co., Rockport, Mass., crushed stone.
 White Fireproof Construction Co., New York, N. Y., hard coal cinders.
 Camp Conduit Co., Cleveland, Ohio, hollow clay tile.
 National Fireproofing Co., Pittsburgh, Pa., hollow clay tile.
 Whitacre Fireproofing Co., Waynesburg, Ohio, hollow clay tile.
 Gypsum Fireproofing Co., Chicago, Ill., gypsum block.
 Keystone Fireproofing Co., New York, N. Y., gypsum block.
 Bestwall Manufacturing Co., Chicago, Ill., gypsum wall board.

The participation of the Associated Factory Mutual Fire Insurance Companies was under the direction of H. O. Lacount, assistant secretary and engineer, and they were represented on the preliminary conference and preparatory work by C. W. Mowry, and at different times in conducting the tests and preparing the results for publication, by R. E. Manning, W. G. Lawrence, and R. E. Wilson.

The National Board of Fire Underwriters participated through Underwriters' Laboratories, the work being under the direction of W. C. Robinson, vice president, assisted by F. W. Frederick, W. G. Howell, and F. Taylor in designing and constructing the testing furnace and related equipment, and at different times by G. W. Riddle and R. K. Porter in conducting the tests.

The cooperation of this Bureau was under the administrative direction of C. W. Waidner, chief of the division of heat and thermometry. S. H. Ingberg and H. K. Griffin were actively associated with the work for the periods 1914 to 1920 and 1917 to 1920, respectively, the former being in direct charge of the experimental program.

Technical assistance for extended periods was given by A. J. Steiner and R. F. Zeunnert.

Mineralogical analyses of concrete aggregates were made by R. S. Knappen of the department of geology of the University of Chicago.

Acknowledgment is also due to a number of engineers, contractors, architects, and public officers who kindly examined a preliminary draft of the schedule of tests which was submitted to them. Their criticisms and suggestions were duly considered in formulating the final program.

II. DESCRIPTION OF COLUMNS

1. STRUCTURAL STEEL COLUMNS

(a) DETAILS OF DESIGN

In Figs. 1 to 4 are shown details of all structural sections employed in the tests. The lower 12 feet 8 inches constitute the test column proper, the upper enlarged extension, 3 feet in length, serving merely as a means for transmitting the load to the column. This head is protected by concrete as shown in Fig. 9, and, being above the ceiling line, is not directly exposed to the heat of the testing chamber.

The bottom base angles are designed to develop about one-half of the transverse strength of the column considered as a beam at ordinary temperature, and during the test are embedded in the fireproofing for the base plates of the testing machine. The top anchorage is designed to develop the full transverse strength of the column.

The test columns are provided with brackets near the top to introduce conditions affecting the application of the protective coverings similar to those obtaining in building construction.

On account of the prevailing use of the solid rolled and built-up H sections, more than one-half of the total number of steel columns used in the tests were of these types.

FIG. 1.—Details of structural-steel columns. Rolled H and plate and angle sections

FIG. 2.—Details of structural-steel columns. Plate and channel and latticed channel sections

FIG. 3.—Details of structural-steel columns. Z-bar and plate and I-beam and channel sections

FIG. 4.—Details of structural-steel columns. Latticed angle and starred angle sections

(b) BEARINGS

The column bases were specified to be finished by milling. While this had been done in almost all cases, it was necessary to improve most of the bearings by grinding, in order not to induce uneven stress distribution in the columns when loaded.

(c) RIVETING

This was examined by striking the rivet heads with a hammer. Very few loose rivets were found, and these were redriven before testing.

(d) INITIAL STRAIGHTNESS

Before being covered or tested, the columns were examined for straightness by measuring from points on the column as placed in vertical position to a fine wire stretched from base angles to bracket. The columns were generally straight within $\frac{1}{8}$ of an inch and in all cases within $\frac{3}{16}$ of an inch.

(e) PROPERTIES OF SECTIONS

The area of one or more sections of each type was obtained by calipering. The special gages used are shown in Fig. 5. The large gage (a), having a range from 0 to 12 inches, was used for outside measurements, the distance between the point of the pin and the point of the dial stem at zero reading being obtained by means of calibrated end measuring rods. The inside caliper (b) has a range from $3\frac{1}{2}$ to $6\frac{1}{2}$ inches, the length between its points at zero reading being determined by measurement in the outside gage. Shape and area of fillets and corners were obtained from plaster impressions.

A comparison of nominal or handbook areas and measured areas is given in Table 1, the two being generally in agreement within 1 per cent and in all cases within 4 per cent. In calculating the loads to be carried during the fire test, the nominal area was used for all columns, the main dimensions of the section members being measured to ascertain their nominal size. The latter was found to be as called for on the details, except in the case of two plate-and-angle and two plate-and-channel columns, where the plates were $\frac{1}{8}$ of an inch heavier than required.

Values of other essential properties of the sections are given in Table 3a (p. 33).

FIG. 5.—Calipers for measuring steel shapes

TABLE 1.—NOMINAL AND MEASURED AREAS OF STRUCTURAL STEEL SECTIONS

Test No.	SECTION	SECTION MEMBERS	Nominal Area, Sq. In.		Measured Area, Sq. In.	
15	Rolled H.....	Solid Rolled H 34.5 lb.....	10 17		10.06	
30	Rolled H.....	Solid Rolled H 34.5 lb.....	10.17		10.05	
50	Plate and Angle.....	1 Plate $\frac{1}{2}$ by 6 in..... 4 Angles 3 by $2\frac{1}{2}$ by $\frac{1}{2}$ in.....	3.00 10.00	13.00	2.99 9.97	12.96
51	Plate and Angle.....	1 Plate $\frac{1}{2}$ by 6 in..... 4 Angles 3 by $2\frac{1}{2}$ by $\frac{1}{2}$ in.....	3.00 10.00	13.00	2.92 9.94	12.86
110	Plate and Angle.....	1 Plate $\frac{1}{2}$ by 6 in..... 4 Angles 3 by $2\frac{1}{2}$ by $\frac{1}{2}$ in.....	3.00 10.00	13.00	2.99 10.21	13.20
39	Plate and Channel....	2 Plates $\frac{3}{4}$ by 8 in..... 2 Channels 6 in.—8 lb.....	4.50 4.76	9.26	4.51 4.73	9.24
107	Plate and Channel....	2 Plates $\frac{1}{4}$ by 8 in..... 2 Channels 6 in.—8 lb.....	4.00 4.76	8.76	4.19 4.57	8.76
54	Latticed Channel.....	2 Channels 9 in.—13 $\frac{1}{4}$ lb.....	7.78		7.80	
41	Z-bar and Plate.....	1 Plate $\frac{1}{4}$ by $5\frac{3}{4}$ in..... 4 Z-bars 3 by $\frac{1}{4}$ in.....	1.44 7.88	9.32	1.46 7.85	9.31
56	Z-bar and Plate.....	1 Plate $\frac{1}{4}$ by $5\frac{3}{4}$ in..... 4 Z-bars 3 by $\frac{1}{4}$ in.....	1.44 7.88	9.32	1.48 7.59	9.07
44	I-beam and Channel.....	1 I-beam 7 in.—15 lb..... 2 Channels 7 in.—9 $\frac{1}{4}$ lb.....	4.42 5.70	10.12	4.54 5.71	10.25
58	I-beam and Channel.....	1 I-beam 7 in.—15 lb..... 2 Channels 7 in.—9 $\frac{1}{4}$ lb.....	4.42 5.70	10.12	4.59 5.76	10.35
46	Latticed Angle.....	4 Angles 3 by 3 by $\frac{3}{8}$ in.....	8.44		8.45	

2. CAST-IRON COLUMNS

(a) DETAILS OF DESIGN AND MANUFACTURE

Structural details are given in Fig. 6. The columns shown in (a) and (b) were made by a Chicago foundry experienced in the making of building castings. They were cast horizontally, with continuous core supported by chaplets, and single gate and riser, the gate being at one end and the riser at the other.

Five of these columns were tested with ends restrained by bolting to base plates at top and bottom as shown in (a), and two were tested with ends not restrained, as shown in (b). In the latter case the bolts at the bottom were omitted, and at the top the

FIG. 6.—Details of cast-iron columns

column was cut at the junction with the head, a bearing plate being inserted between the cut surfaces. A U bolt and strap on each side served to hold the column end in case the column should break at failure.

Three columns were of the cast-iron-pipe type with detached cap, as shown in (c) and (d) of Fig. 6. These columns were cast in vertical position. In testing, substantially the same bearing details were used as for the horizontally-cast columns that were tested with ends not restrained.

(b) BEARINGS

The ends of the columns were milled, the bearings being in all cases fairly even and true. All bearing surfaces of caps and bearing plates were machined. The top and bottom bearings were protected by fireproofing, the same as for the structural-steel columns, the length exposed to the fire being 12 feet.

(c) INITIAL STRAIGHTNESS

The amount and direction of initial curvature was determined in the same manner as for the structural-steel columns. The columns that were cast horizontally were straight within $\frac{1}{8}$ of an inch, and those cast on end were straight within $\frac{1}{16}$ of an inch.

(d) VARIATIONS IN METAL THICKNESS

In the case of the horizontally cast columns, the core at the midpoint of the length was found to have been displaced by amounts varying from $\frac{1}{16}$ to $\frac{1}{4}$ of an inch for the individual columns. The minimum thickness of metal was $\frac{1}{2}$ an inch against a nominal thickness of $\frac{3}{4}$ of an inch. The area of metal exceeded the nominal area by amounts up to about 1 square inch. The exact effective area was difficult to determine on account of the roughness of the interior surface.

For the vertically cast columns the thickness varied from $\frac{5}{8}$ to $\frac{3}{4}$ of an inch against a nominal thickness of $\frac{1}{2}$ of an inch, the area being generally about $\frac{1}{2}$ square inch in excess of the nominal.

3. PIPE COLUMNS

(a) DETAILS OF DESIGN AND MANUFACTURE

Two concrete-filled pipe columns were tested, one made with a standard 7-inch pipe, and the other with a standard 8-inch pipe reinforced in the fill with four $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{3}{8}$ inch angles riveted back to back. Details of columns and bearings are given in Fig. 7. The details at the top are similar to those of the

FIG. 7.—Details of pipe columns

cast-iron columns that were tested with unrestrained ends except that a short strut was placed between the top of the cap and the upper bearing plate to obtain conditions more nearly representative of use in buildings.

The 7-inch pipe column was filled at the manufacturer's plant especially for the test and specimens of concrete and concrete aggregates were secured. The concrete mixture was 1 part Portland cement, $1\frac{1}{2}$ parts Cambridge (Mass.) bank sand, and 3 parts crushed blue trap rock quarried at Westfield, Mass. The reinforced-pipe column was obtained from a stock of completed columns, and specimens of the concrete aggregates could not be obtained, but they were said to be the same as for the 7-inch column.

(b) BEARINGS

The bearings were square and unrestrained. The bearing surfaces of the caps were unfinished; base and top bearing plates were machined. The pipes had sawed ends that were fairly even. The concrete on the bottom bearing of the 7-inch pipe column projected about $\frac{1}{8}$ of an inch below the pipe.

4. REINFORCED CONCRETE COLUMNS

(a) DETAILS OF DESIGN

The types tested include square and round longitudinally reinforced columns, and round columns with lateral reinforcement of spiral hooping and longitudinal bar reinforcement. Details are shown in Figs. 8 and 9. The spiral reinforcement constitutes in volume about 1 per cent of the contained concrete. The size and spacing of the lateral ties in the longitudinally reinforced columns represent current practice with respect to this detail.

The concrete was of 1 : 2 : 4 mixture. In the columns for the fire tests, Fox River (Ill.) sand with Chicago limestone and Long Island (N. Y.) sand with New York trap rock were the two combinations of aggregates used. For the fire and water tests each column was cast in three 4-foot sections with concrete of different aggregates in each section.

(b) BEARINGS AND END RESTRAINT

The lower ends of the vertical bars were ground true and abutted on the base plate on which the column was cast, the upper ends of the bars terminating $\frac{1}{2}$ an inch below the top bearing plate. The head was cast monolithic with the test column proper, and was

FIG. 9.—Details of hooped concrete column and column-head protection

suitably reinforced and anchored into the top bearing plate. The latter was set in Portland cement mortar. At the bottom, the the column was tied to the base plate with four $\frac{7}{8}$ -inch bolts.

5. TIMBER COLUMNS

(a) SPECIES OF TIMBER

Tests of timber columns include four with long-leaf yellow pine and two with Douglas fir, these two species being chosen on account of wide use in heavy timber construction. Details of columns and bearings are given in Fig. 10.

The yellow-pine columns were cut from timber grown in Pike County, Miss. The Douglas-fir columns came from the northern Douglas-fir region of the Pacific coast.

(b) CAP AND BEARING DETAILS

In the construction shown at (a) in Fig. 10, the load is transmitted to the column through a cast-iron pintle and cap. The side of the pintle and the top bearing of the cap are covered by timbers and flooring that frame into them in a manner similar to that used in standard applications of mill construction.

In the construction shown at (b) the load is transmitted through a timber strut and steel-plate cap, this method of column and beam support typifying another form of standard practice with regard to these details.

The bearings were finished so as to be fairly even and perpendicular to the axis of the column.

(c) PROPERTIES OF THE TIMBER

The chief characteristics of the timber are given in Table 2. The columns were select structural material with few knots or other defects. They were surfaced on the sides and the corners were slightly beveled. A nominal section of $11\frac{3}{8}$ by $11\frac{3}{8}$ inches was assumed in calculating the working load.

The number of annual rings per inch and the percentage of summer wood were determined on a representative line over the third, fourth, and fifth inches from the pith, the values given being the average for the two end faces. The resin content was obtained by extracting borings from representative points in the section with benzole and drying to constant weight at 70° C.

The moisture content and dry weight were determined from disks 1 inch thick cut 2 feet and 4 feet from the end of a timber of the same size as the test columns and which had been subjected to

FIG. 10.—Details of timber columns

TABLE 2.—PROPERTIES OF TIMBER IN TEST COLUMNS

Test No.	Species	Dimension of Section, In.	Number of Rings per In.	Summer-wood, Percent	*Rosin Content, Percent	*Moisture Content, Percent	Weight per Cu. Ft., Pounds	
							**As Tested	*Oven Dry
78	Longleaf pine.....	11 $\frac{1}{8}$ by 11 $\frac{1}{2}$	14	35	7.07	17.1	41.3	34.7
79	Longleaf pine.....	11 $\frac{1}{4}$ by 11 $\frac{1}{4}$	17	50			-----	
80	Longleaf pine.....	11 $\frac{1}{4}$ by 11 $\frac{1}{8}$	11	35			41.3	
81	Longleaf pine.....	11 $\frac{3}{8}$ by 11 $\frac{3}{8}$	14	35			46.7	
82	Douglas fir.....	11 $\frac{3}{8}$ by 11 $\frac{3}{8}$	9	33	1.34	26.2	36.1	30.1
83	Douglas fir.....	11 $\frac{3}{8}$ by 11 $\frac{3}{8}$	10	33			39.0	

*Determined from representative samples.

**Determined by weighing the test columns.

the same storage conditions. These were dried to constant weight at 100° C, the percentage moisture, which includes besides water other substances volatile at the given temperature, being based on the dry weight.

In point of general quality of material, the test columns conformed with the requirements of published specifications for structural timber. The dimensions of the finished section were smaller by about $\frac{1}{8}$ of an inch than as specified by some regulations.

III. SCHEDULE OF TESTS

1. GENERAL CONSIDERATIONS

(a) OBJECT AND LIMITATIONS

The present investigation, in point of method of testing and types of protection tested, applies particularly to the interior columns of a structure. Due to greater exposure and smaller amount of protection, these form, in general, a more critical element in the column strength of a building as affected by fire than do the wall columns.

The size of the test columns in the present series is representative of those present in buildings of moderate height and of those under upper floors in higher buildings. The sizes involved were deemed to be best adapted for the initial investigation of the many variables pertaining to the fire resistance of building columns as designed and protected according to methods of modern building practice. Subsequent investigation of columns designed for the

heavier loads should be materially simplified by results of tests with columns of the proportions chosen for the present series.

The number of variables presented by the variety in material and form of columns, material and amount of protection, as well as the different methods of application, is greater than can be fully covered by a single series of tests, considering the limitations in time and expense incident with such an effort. It was necessary, therefore, to limit the investigation to the main forms of construction and protection, and in amount of protection, to the minimum and maximum as generally used.

(b) PRELIMINARY WORK ON SCHEDULE

This consisted of (1) investigation of existing methods of column design and column protection, including comparative study of the requirements of municipal building codes; (2) inquiry into field methods of erection and placing of columns and coverings; (3) study of the geographical distribution of production and use of materials for protective coverings of each type as an aid in selecting representative materials for the tests; (4) preparation of a preliminary schedule of tests which was submitted to engineers, contractors, architects, and public officers for criticisms and suggestions; (5) consideration of criticisms offered and formulation of a final testing program.

In amount of protection for interior columns, building ordinances require in general from 2 to 4 inch thicknesses of material outside of load-carrying elements, the requirements for the same grade of construction varying between these limits, as prescribed by different city codes. As it was considered desirable to determine the fire-resisting value of constructions in general use, it was decided to test protections of 2 and 4 inch commercial thicknesses, which, in connection with tests of unprotected and partly protected columns, as also of plaster on metal-lath protections, were deemed to include the general range of protection occurring in current building construction.

The schedule, as finally adopted at the conference of representatives of the cooperating units held in March, 1916, embodied many suggestions received in criticism of the preliminary schedule and was considered by all concerned to be the best procedure practicable, considering the extent of the field and the number of tests to be made.

2. SCHEDULE OF FIRE TESTS

(a) UNPROTECTED COLUMNS

Details of design of structural-steel, cast-iron, and pipe columns are shown in Figs. 1, 2, 3, 4, 6, and 7, and the calculated and applied working loads pertaining to each section are given in Table 41.

The unprotected structural-steel columns, which comprise one of each section type, are scheduled in Table 3a, together with the chief properties of their sections. These hold for the lower 12 feet, 8 inches of their length.

The list of unprotected cast-iron and pipe columns is given in Table 3b. Of the unprotected cast-iron columns, one was tested with ends restrained and three with unrestrained ends. Of the latter, one column, No. 11, was filled with concrete to increase its fire resistance. They were all horizontally cast, except No. 10A, which was a cast-iron pipe column.

The tests with pipe columns comprise one with a standard 7-inch pipe having plain concrete fill and one with an 8-inch pipe reinforced in the fill with structural angles.

(b) PARTLY PROTECTED COLUMNS

These include nine structural steel columns partly protected by filling the reentrant portions or interior with concrete and are scheduled in Table 3c, where also are shown the detail sections.

The proportions of the concrete mixtures used throughout this investigation are based on volume parts of the respective materials, Portland cement, sand, and coarse aggregate, taken in the given order. One bag of Portland cement weighing 94 pounds net was taken to be 1 cubic foot. The sand and coarse aggregate were measured by volume in the condition of density incident with shoveling them into the measures. The concrete sands as used had an average moisture content of 3 per cent. The Portland cement used in all tests of the whole series, except in those with pipe columns, was supplied from a mill in the Chicago district.

(c) COLUMNS PROTECTED BY PLASTER ON METAL LATH

Details of protection are given in Table 3d. The metal lath in Nos. 23, 25, and 26 was No. 24 expanded metal weighing 3.4 pounds per square yard including paint. That for test No. 24 was of 0.046-inch diameter wire, woven into $\frac{3}{8}$ -inch square mesh and weighing, painted, 3.2 pounds per square yard. The ribbed, expanded-metal lath in No. 27 had ribs $\frac{3}{4}$ inch high spaced $3\frac{1}{2}$ inches apart, the weight being 7.9 pounds per square yard.

TABLE 3a.—SCHEDULE OF FIRE TESTS
Unprotected Structural Steel Columns
1—Effective Length, 12 ft. 8 in.

Test No.	SECTION	SECTION MEMBERS	Nominal Area, Sq. In.	Least Radius of Gyration, r, In.	$\frac{1}{r}$
1	Rolled H	Solid Rolled H, 8 in.—34½ lb..	10.17	2.01	75.6
2	Plate and Angle	1 Plate, ½ by 6 in..... 4 Angles, 3 by 2½ by ½ in.....	13.00	1.36	111.8
3	Plate and Channel	2 Plates, ¼ by 8 in..... 2 Channels, 6 in.—8 lb.....	8.76	2.85	64.7
4	Latticed Channel	2 Channels, 9 in.—13¼ lb..... Single lattice, ¼ by 2 in.....	7.78	3.43	44.0
5	Z-bar and Plate	1 Plate, ¼ by 5½ in..... 4 Z-bars, 3 by ¼ in.....	9.32	1.86	81.7
6	I-beam and Channel	1 I-beam, 7 in.—15 lb..... 2 Channels, 7 in.—9¼ lb.....	10.12	2.11	72.1
7	Latticed Angle	4 Angles, 3 by 3 by ½ in..... Single lattice, ¼ by 2¼ in.....	8.44	3.73	40.7
8	Starred Angle	4 Angles, 3 by 3 by ½ in..... 1 Plate, ½ by 6½ in..... 2 Plates, ½ by 3½ in.....	13.27	1.40	108.6

TABLE 3b.—SCHEDULE OF FIRE TESTS
Unprotected Cast Iron and Pipe Columns

Test No.	SECTION	DETAILS	Nom- inal Area, Sq. In.	Effe- ctive Length, l, In.	Least Radi- us of Gyra- tion, r, In.	$\frac{1}{r}$
9	Round Cast Iron, Horizontally cast	Ends restrained	14.73	152	2.23	68.2
10	Round Cast Iron, Horizontally cast	Ends not restrained	14.73	152	2.23	68.2
10-A	Round Cast Iron, Vertically cast	Ends not restrained	14.45	150 1/4	2.38	63.2
11	Round Cast Iron, Horizontally cast, Concrete filled	Ends not restrained Concrete, 1:2:4 Portland cement Joliet sand Joliet gravel	14.73	152	2.23	68.2
12	Steel Pipe, Concrete filled	Ends not restrained Concrete, 1:1 1/2:3 Portland cement Cambridge sand Westfield blue trap	Steel 6.93 Con- crete 38.7	149	2.34	63.9
13	Reinforced Steel Pipe, Concrete filled	Ends not restrained 4 angles inside pipe, 3 1/2 in. 3 1/2 in. 3/4 in. Concrete same as for No. 12	Steel 18.36 Con- crete 40.1	152 3/4	2.24	68.2

TABLE 3c.—SCHEDULE OF FIRE TESTS
Columns Partly Protected by Concrete

Test No.	SECTION	PROTECTION	
		Mixture	Material
14	Rolled H	1:2:4	Portland cement Joliet sand Joliet gravel
15	Rolled H	1:2:4	Portland cement Plum Island sand Rockport granite
16	Plate and Angle	1:2:4	Portland cement Long Island sand New York trap
17	Plate and Angle	1:1 1/2:4 1/2	Portland cement Long Island sand Hard coal cinders
18	Latticed Channel	1:2:4	Portland cement Long Island sand New York trap
19	Z-bar and Plate	1:3:5	Portland cement Fox River sand Chicago limestone
20	I-beam and Channel	1:3:5	Portland cement Long Island sand New York trap
21	I-beam and Channel	1:3:5	Portland cement Long Island sand New York trap
22	Latticed Angle	1:2:4	Portland cement Fox River sand Chicago limestone

NOTE: The horizontal ties, consisting of $\frac{1}{2}$ -inch bars or No. 5 wire, are bent around or wired to the vertical $\frac{1}{2}$ -inch bars, and are spaced 18 inches apart vertically.

TABLE 3d.—SCHEDULE OF FIRE TESTS
Columns Protected by Plaster on Metal Lath

Test No.	SECTION	*PROTECTION
23	Plate and Angle	Two 2-coat layers of Portland cement plaster on No. 24 expanded metal lath, each layer 1 in. thick, with a 1/4-in. air space between layers
24	Plate and Channel	Two 2-coat layers of Portland cement plaster on woven wire lath, 3/8-in. mesh, each layer 7/8 in. thick, with a 1/4-in. air space between layers
25	Z-bar and Plate	One 2-coat layer of Portland cement plaster, 1 in. thick, on No. 24 expanded metal lath
26	Latticed Angle	One 2-coat layer of Portland cement plaster, 1 1/2 in. thick, on No. 24 expanded metal lath
27	Round Cast Iron Column Section same as No. 9	One 2-coat layer of Portland cement plaster, 1 1/2 in. thick, on high ribbed expanded metal lath with 1/2-in. broken air space

*The plaster consisted of 1 part Portland cement, 1/10 part hydrated lime, 2 1/2 parts coarse lake sand. Hair was used in the first coat; the second coat was trowelled smooth.

The proportion of the plaster was 1 part Portland cement, $\frac{1}{10}$ part hydrated lime, and $2\frac{1}{2}$ parts lake sand of medium grade of coarseness, all materials being measured by loose volume. The sand had an average moisture content of 3 per cent.

The thickness of the layers was measured from the inside of the lath. For Nos. 23, 24, and 25 a layer thickness of 1 inch was desired and was attained in Nos. 23 and 25. In No. 24 the layer thickness averaged $\frac{7}{8}$ of an inch. In No. 26 an attempt was made to place a 2-inch thick layer, but this proved impracticable with the given section using two body coats, an average thickness of $1\frac{1}{8}$ inches being finally attained. In No. 27 an effort was made to make a solid covering by pushing the plaster through the lath against the outside of the column. This succeeded only partly, a broken air space averaging $\frac{1}{2}$ inch in depth remaining between the covering and the column.

For all plaster on metal-lath protections the extreme edges of the bracket angles near the top of the column were covered by a single layer 1 inch thick.

The coverings were finished by troweling and floating the second coat to a smooth surface.

(d) COLUMNS PROTECTED BY CONCRETE

The tests with concrete protections are described in Table 3e.

Six combinations of fine and coarse concrete aggregates as used in building construction in four large industrial centers are included. The aggregate combinations and districts represented are (1) Rockport granite with Plum Island sand for the Boston (Mass.) district; (2) Chicago limestone with Fox River sand and Joliet gravel with Joliet sand for the Chicago (Ill.) district; (3) Cleveland sandstone with Pelee Island sand for the Cleveland (Ohio) district; (4) New York trap rock with Long Island sand and hard-coal cinders with Long Island sand for the New York (N. Y.) district. The districts were chosen so as to obtain representation for the main groups of rocks that are used as concrete aggregate. The aggregates are further described in paragraph 6 of Section V (p. 61).

The proportions of the mixtures used were 1 : 2 : 4 and 1 : 3 : 5 for the stone and gravel concrete, and for the cinder concrete, 1 : $1\frac{1}{2}$: $4\frac{1}{2}$ and 1 : 2 : 5. The cinders were used unscreened as received, except that pieces larger than 1 inch were crushed to smaller size.

TABLE 3e.—SCHEDULE OF FIRE TESTS

Columns Protected by Concrete

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	Material
28	Rolled H	2	1:2:4	Portland cement Fox River sand Chicago limestone
28 A	Rolled H	2	1:2:4	Portland cement Fox River sand Chicago limestone
29	Rolled H	2	1:2:4	Portland cement Long Island sand New York trap
30	Rolled H	2	1:2:4	Portland cement Joliet sand Joliet gravel
31	Rolled H	2	1:2:4	Portland cement Peele Island sand Cleveland sandstone
32	Rolled H	2	1:2:5	Portland cement Long Island sand Hard coal cinders
32 A	Z-bar and Plate	2	1:2:5	Portland cement Long Island sand Hard coal cinders
33	Rolled H	4	1:2:4	Portland cement Fox River sand Chicago limestone
33 A	Rolled H	4	1:2:4	Portland cement Fox River sand Chicago limestone

NOTE: Ties where used are of No. 5, B. & S. gage, steel wire, wound spirally on a pitch of 8 in.

TABLE 3e.—SCHEDULE OF FIRE TESTS**Columns Protected by Concrete—Continued**

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	Material
34	Rolled H	4	1:2:4	Portland cement Plum Island sand Rockport granite
34A	Rolled H	4	1:2:4	Portland cement Plum Island sand Rockport granite
35	Rolled H	4	1:3:5	Portland cement Fox River sand Chicago limestone
36	Plate and Angle	2	1:2:4	Portland cement Long Island sand New York trap
37	Plate and Angle	4	1:2:4	Portland cement Long Island sand New York trap
38	Plate and Channel	2	1:2:4	Portland cement Joliet sand Joliet gravel
39	Plate and Channel	4	1:2:4	Portland cement Meramec River sand Meramec River gravel

NOTE: Wire ties are of No. 5, B. & S. gage, steel wire, wound spirally on a pitch of 8 in.

TABLE 3a.—SCHEDULE OF FIRE TESTS
Columns Protected by Concrete—Concluded

Test No.	SECTION	PROTECTION		
		Thickness. In.	Mixture	Material
40	Latticed Channel	2	1:2:4	Portland cement Long Island sand New York trap
41	Z-bar and Plate	4	1:3:5	Portland cement Fox River sand Chicago limestone
42	Z-bar and Plate	4	1:3:5	Portland cement Fox River sand Chicago limestone
43	I-beam and Channel	2	1:2:4	Portland cement Pele Island sand Cleveland sandstone
44	I-beam and Channel	2	1:3:5	Portland cement Pele Island sand Cleveland sandstone
45	Starred Angle	2	1:2:4	Portland cement Meramec River sand Meramec River gravel
46	Latticed Angle	2-in. outside rivets. 3 1/8-in. outside angles.	1:2:4	Portland cement Long Island sand New York trap
47	Round Cast Iron	2	1:2:5	Portland cement Long Island sand Hard coal cinders

NOTE: Wire ties where used are of No. 5, B. & S. gage, steel wire wound spirally on a pitch of 8 in.

Ties consisting of No. 5 (B. and S. gage) bright basic steel wire were wound spirally around the structural section on a vertical pitch of 8 inches. The tie was omitted in Test No. 28A and 33A in order to determine what effect, if any, it has on the effectiveness of the covering. In No. 46 it was omitted because the latticed section was deemed to afford sufficient support for the outside covering. In No. 47 an attempt was made to place the covering with the tie wire supported on 1-inch T bars. This proved impracticable on account of the obstruction it made to the flow of the concrete, this covering being, therefore, placed without tie.

For the square coverings the thickness was measured, respectively, from the face of the flange and from its extreme edge. For the round protections, test Nos. 37 and 40, the thickness of covering on the face of the flange was greater than the given nominal thickness and that on the flange edge smaller than the given thickness, each by about $\frac{5}{8}$ of an inch.

For the concrete, as well as all other full protections, the thickness of covering on the extreme edges of the bracket angles near the top of the column was 1 inch.

(c) COLUMNS PROTECTED BY HOLLOW CLAY TILE

The tests with hollow clay-tile protections are given in Table 3f.

Five kinds of tile from as many producing regions were used for the coverings. These include two varieties of surface-clay tile, one of shale, and two of semifire clay. They are further described in paragraph 11 of Section V (p. 73).

The tile was set in mortar consisting of 1 part, by loose volume, of Portland cement, 1 part of lime putty, and 4 parts of fine beach or bank sand. The sand had an average moisture content of 4 per cent. The thickness of mortar between the tile and the flanges of the test columns varied between the different protections from $\frac{1}{2}$ to $1\frac{1}{8}$ inches. The thickness of horizontal joints between the tile courses averaged $\frac{3}{8}$ of an inch where no wire mesh was used and $\frac{5}{8}$ of an inch where used. The vertical mortar joints varied in thickness from $\frac{1}{4}$ to $\frac{3}{4}$ of an inch, depending on the design of the covering.

The upper tile courses were set out sufficiently to allow the extreme edges of the bracket angles to be covered by a 1-inch thickness of tile and mortar.

Two forms of mechanical ties for the tile were used. One consisted of a No. 12 (B. and S. gage) iron wire, tied around the outside of each course, and the other of strips of $\frac{3}{8}$ -inch wire mesh (diameter of wire, 0.046 inch) laid in the horizontal joints and lapped at the corners.

TABLE 31.—SCHEDULE OF FIRE TESTS**Columns Protected by Hollow Clay Tile**

Test No.	SECTION	Thick- ness of Tile, in.	Kind of Tile and Method of Application	Filling
48	Rolled H	2	Semi-fire clay, New Jersey district 3/4-in. mortar on flanges Outside wire ties	No filling
49	Rolled H	4	Semi-fire clay, New Jersey district 3/4-in. mortar on flanges	No filling
50	Plate and Angle	2	Surface clay, Boston district 1/4-in. mortar on flanges	1:3:5 concrete Portland cement Plum Island sand Rockport granite
50-A	Plate and Angle	2	Same as No. 50	Same as No. 50
51	Plate and Angle	4	Surface clay, Boston district 1 1/8-in. mortar on flanges Outside wire ties	1:3:5 concrete Portland cement Plum Island sand Rockport granite
51-A	Plate and Angle	4	Same as No. 51	Same as No. 51
52	Plate and Channel	2	Ohio shale 3/4-in. mortar on flanges Outside wire ties	1:2:5 concrete Portland cement Long Island sand Hard coal cinders
53	Plate and Channel	4	Ohio shale 1-in. mortar on flanges Outside wire ties	1:2:5 concrete Portland cement Long Island sand Hard coal cinders

TABLE 3f.—SCHEDULE OF FIRE TESTS
Columns Protected by Hollow Clay Tile—Continued

Test No.	SECTION	Thick-ness of Tile, in.	Kind of Tile and Method of Application	Filling
54	Latticed Channel	2	Ohio semi-fire clay Outside wire ties	1:3:5 concrete, Portland cement Long Island sand New York trap
55	Z-bar and Plate	2	Ohio semi-fire clay Outside wire ties	1:3:5 concrete, Portland cement Fox River sand Chicago limestone
56	Z-bar and Plate	4	Ohio semi-fire clay 3/8-in. wire mesh in horizontal joints	1:3:5 concrete, Portland cement Fox River sand Chicago limestone
57	I-beam and Channel	4	Surface clay, Chicago district Outside wire ties	1:3:5 concrete, Portland cement Fox River sand Chicago limestone
58	I-beam and Channel	Two 2-in.	Surface clay, Chicago district 1/2-in. mortar on flanges 3/8-in. wire mesh in horizontal joints	Hollow clay tile, 2 by 12 by 6 in. at ends
59	I-beam and Channel	Two 2-in.	Surface clay, Chicago district 1/2-in. mortar on flanges Outside wire ties	Hollow clay tile, 2 by 12 by 6 in. at ends 3 by 12 by 6 in. at sides

TABLE 3f.—SCHEDULE OF FIRE TESTS
Columns Protected by Hollow Clay Tile—Concluded

Test No.	SECTION	Thick- ness of Tile, In.	Kind of Tile and Method of Application	Filling
60	Latticed Angle	2	Ohio semi-fire clay 1/4-in. mortar between fill and tile Outside wire ties	1:2:4: concrete Portland cement Long Island sand New York trap Fill placed before tile was set
61	Latticed Angle	2	Ohio semi-fire clay Outside wire ties	No filling
62	Round Cast Iron	2	Porous semi-fire clay, New Jersey district 3/4-in. mortar Outside wire ties	No filling
63	Round Cast Iron	2	Same as No. 62	No filling
76	Rolled H	2	Ohio shale: Ohio semi-fire clay; semi-fire clay, New Jersey district 3/4-in. mortar on flanges 3/8-in. wire mesh in hori- zontal joints Tile covered with a 2-coat, 3/4-in. layer of 1:3 gypsum plaster	1:3:5: concrete Portland cement Fox River sand Chicago limestone
77	Plate and Angle	4	Semi-fire clay, New Jersey; surface clay, Chicago; surface clay, Boston dis- trict 1 1/2-in. mortar on flanges 3/8-in. wire mesh in hori- zontal joints Tile covered with a 2-coat, 3/4-in. layer of 1:2 1/2 lime plaster	1:3:5 concrete Portland cement Fox River sand Chicago limestone

NOTE: The mortar used in setting the tile consisted of 1 part Portland cement, 1 part lime putty and 4 parts fine beach or bank sand.

The filling inside of the tile, where used, consisted of concrete or hollow clay tile. The concrete was placed after the tile was set, except in case of No. 60, where the fill was placed and allowed to harden before setting the tile.

All protections were tested in the unplastered condition except those of Nos. 76 and 77. The former was plastered with a two-coat layer of gypsum plaster $\frac{3}{4}$ inch thick. The first coat of about $\frac{5}{8}$ inch thickness consisted of 1 volume part neat, fibered, calcined gypsum and 3 parts fine lake sand, and the finish coat, 1 volume part neat, unfibered, calcined gypsum and 2 parts hydrated lime. No. 77 was plastered with lime plaster $\frac{5}{8}$ of an inch in thickness, the first coat consisting of 1 volume part slaked-lime putty and $2\frac{1}{2}$ parts of fine lake sand, the finish coat being the same as for No. 76.

(f) COLUMNS PROTECTED BY GYPSUM BLOCK

The tests with gypsum block protections scheduled in Table 39 consist of two with 2-inch and three with 4-inch solid block. The material was supplied from two factories, one located in the middle-western section of the country and the other in the eastern section. The proportion of the mortar used for setting the block was 1 part, by volume, of neat, unfibered, calcined gypsum, and 3 parts, by volume, of fine lake sand. The latter as used had a moisture content of about 3 per cent.

In Nos. 64 and 65 the blocks were tied with No. 22 corrugated, galvanized-iron strips, $\frac{3}{4}$ of an inch wide by 6 inches long, placed in the horizontal joints and across all vertical joints, one over each joint in the 2-inch covering and two over each joint in the 4-inch covering. In Nos. 66, 67, and 67A strips of woven wire (0.046-inch wire diameter) of $\frac{3}{8}$ -inch mesh were laid in the horizontal joints over all vertical joints. The size of the strips for the 2-inch protection was 2 by $14\frac{1}{2}$ inches and for the 4-inch protections, $3\frac{1}{2}$ by $16\frac{1}{2}$ inches, the strips being laid so the outer edges were $\frac{1}{2}$ inch from the surface of the covering. The thickness of horizontal joints averaged $\frac{1}{2}$ inch between the 2-inch blocks, and $\frac{3}{4}$ inch between the 4-inch blocks. The vertical joints were about $\frac{3}{8}$ of an inch thick.

The space between the blocks and column flanges was filled with mortar as the blocks were set. The remaining space inside of the blocks was filled with gypsum block set in mortar in the

TABLE 3g.—SCHEDULE OF FIRE TESTS
Columns Protected by Gypsum Block

Test No.	SECTION	Thick- ness of Block, In.	Kind of Gypsum and Method of Application	Filling
64	Rolled H	4	Western gypsum (solid) 3/4-in. mortar on flanges Corrugated wall ties in horizontal joints	Hollow Western gypsum block
65	Plate and Channel	2	Western gypsum (solid) 1-in. mortar on flanges Corrugated wall ties in horizontal joints	Solid Western gypsum block
66	Latticed Channel	2	Eastern gypsum (solid) 3/4-in. mortar on lattice 3/8-in. wire mesh in horizon- tal joints	1:1:4, Calcined gypsum Fine lake sand Broken gypsum block
67	Rolled H	4	Eastern gypsum (solid) 3/4-in. mortar on flanges 3/8-in. wire mesh in horizon- tal joints	1:1:4, Calcined gypsum Fine lake sand Broken gypsum block
67-A	Rolled H	4	Same as No. 67	Same as No. 67

NOTE: The gypsum blocks were set in mortar consisting of 1 part by volume of neat unfibered calcined gypsum and 3 parts fine lake sand.

case of Nos. 64 and 65. For the other gypsum protections the filling material consisted of 1 part, by volume, of neat, unfibred, calcined gypsum, 1 part fine lake sand, and 4 parts gypsum block broken to maximum size of 2 inches, the whole being mixed to wet consistency.

The methods of tying the blocks and filling within them conform with the recommendations of the manufacturers by whom they were supplied.

(g) COLUMNS PROTECTED BY BRICK

The two tests with brick protection are described in Table 3h.

The proportion of the mortar was the same as that used for the clay-tile protections. In placing the brick no metal ties were used, the brick being in each case set so as to obtain the best bond possible with the given design of covering. The thickness of the horizontal joints averaged $\frac{1}{2}$ inch and that of the vertical joints varied from $\frac{1}{4}$ to $1\frac{1}{4}$ inches.

TABLE 3h.—SCHEDULE OF FIRE TESTS
Columns Protected by Brick

Test No.	SECTION	Thick- ness of Brick, In.	Kind of Brick	Filling
68	Rolled H	2 3/4	Chicago common brick set on edge and end 3/8-in. mortar on flanges	Chicago common brick
69	Rolled H	3 3/4	Chicago common brick laid flat 1/2-in. mortar on flanges	Chicago common brick

NOTE: The mortar consisted of 1 part Portland cement, 1 part lime putty and 4 parts bank sand.

(h) REINFORCED-CONCRETE COLUMNS

These are scheduled in Table 3i and details of design are given in Figs. 8 and 9 (p. 26-27).

A thickness of 2 inches of the concrete next to the surface of the column is taken as a protective covering for the concrete and steel within it and is not included in the given effective areas.

TABLE 3i.—SCHEDULE OF FIRE TESTS**Reinforced Concrete Columns****Effective Length, 12 ft. 8 in.**

Test No.	SECTION	Mix	Material	*Effective Area, Sq. In.	Reinforcement, Percent of Effective Area
70	Square Vertically Reinforced	1:2:4	Portland cement Fox River sand Chicago limestone	Concrete, 140 Steel, 4.00	Vertical, 2.78 Lateral, 0.14
71	Square Vertically Reinforced	1:2:4	Portland cement Long Island sand New York trap	Same as No. 70	Same as No. 70
72	Round Vertically Reinforced	1:2:4	Portland cement Fox River sand Chicago limestone	Concrete, 127 Steel, 6.00	Vertical, 4.52 Lateral, 0.13
73	Round Vertically Reinforced	1:2:4	Portland cement Long Island sand New York trap	Same as No. 72	Same as No. 72
74	Hooped Reinforced	1:2:4	Portland cement Fox River sand Chicago limestone	Concrete, 129 Steel, 3.38	Vertical, 2.54 Lateral, 0.99
75	Hooped Reinforced	1:2:4	Portland cement Long Island sand New York trap	Same as No. 74	Same as No. 74

*A depth of 2 in. all around deducted from the gross section for fire protection.

(i) TIMBER COLUMNS

The schedule of timber columns is given in Table 3j and details of columns and bearings are shown in Fig. 10. The tests include two species of timber, each tested with two types of bearing details.

No. 78 was protected by a single layer of plaster on metal lath, the details of application being the same as for the protections listed in Table 3d. No. 80 was covered with gypsum wall board $\frac{3}{8}$ inch thick, nailed to the timber at the corners with No. 4 lathing nails on 2-inch centers, and finished with kalsomine. The other timber columns were tested unprotected.

TABLE 3j.—SCHEDULE OF FIRE TESTS

Timber Columns

Effective Length, 12 ft. 8½ in.

Test No.	SPECIES	SECTION	Protection of Column and Cap	Bearing Details
78	Longleaf pine		One 2-coat layer of Portland cement plaster, 1 in. thick, on woven wire lath, $\frac{3}{8}$ -in. mesh, $\frac{1}{2}$ -in. air space. Plaster consisted of 1 part Portland cement, 1/10 part hydrated lime, 2½ parts coarse lake sand	Cast iron cap and pintle
79	Longleaf pine		Unprotected	Cast iron cap and pintle
80	Longleaf pine		One thickness of $\frac{3}{8}$ -in. gypsum wall board with metal corner beads, nailed to column	Steel plate cap and 8 in. long timber strut.
81	Longleaf pine		Unprotected	Steel plate cap and 6 in long timber strut
82	Douglas fir		Unprotected	Cast iron cap and pintle
83	Douglas fir		Unprotected	Steel plate cap and 18 in. long timber strut

3. SCHEDULE OF FIRE AND WATER TESTS

This series was introduced in order to determine the effect on coverings and columns of the impact and sudden cooling produced by hose streams applied to them when in a highly heated condition.

In order to introduce all of the materials used in the fire-test series with a minimum number of tests, two or three kinds of material of a given class were applied in each test.

(a) COLUMNS PROTECTED BY CONCRETE

The fire and water tests with concrete protections are scheduled in Table 4a. Three kinds of concrete were applied to each column. In this and succeeding tables the concrete is distinguished by the name of the coarse aggregate, the sand used with each being the same as in the combinations given above in paragraph 2d (p.37).

The metal tie in the covering is the same as for the corresponding protections in the fire-test series. In No. 102 the tie is omitted.

(b) COLUMNS PROTECTED BY HOLLOW CLAY TILE

These coverings are also placed in three sections (Table 4b) with one of the varieties of tile used in the protections of the fire-test series in each section. The proportions of the mortar and size of metal ties are the same as for the corresponding protections of the fire-test series.

(c) COLUMNS PROTECTED BY GYPSUM BLOCK

The two kinds of gypsum with the method of application peculiar to each, described in paragraph 2f of this section are employed in the two tests (Table 4c). The filling for both consists of a 1:1:4 mixture by volume parts of calcined gypsum, fine lake sand, and broken gypsum block, mixed to wet consistency.

(d) PLASTER ON METAL-LATH PROTECTION

The protection of this type included in the fire and water series is described in Table 4d. The metal lath, proportion of the plaster, and method of application are the same as for the corresponding protections of the fire-test series.

(e) REINFORCED-CONCRETE COLUMNS

One column of each type is included in this series. Details of concrete and reinforcement are given in Table 4e.

(f) UNPROTECTED CAST-IRON COLUMNS

Two duplicate columns are listed in Table 4f. They are vertically cast and have bearing details as shown in Fig. 6 (c) (p. 22).

TABLE 4a.—SCHEDULE OF FIRE AND WATER TESTS
Columns Protected by Concrete

Test No.	SECTION	PROTECTION		
		Thickness, In.	Mixture	*Kind of Concrete
101	Rolled H	2	1:2:4	Chicago limestone New York trap Joliet gravel
102	Rolled H	2	1:2:4	New York trap Joliet gravel Chicago limestone
103	Plate and Angle	4	1:2:4	New York trap New England granite Chicago limestone
104	Plate and Angle	2	1:2:5 1:2:4 1:2:4	Hard coal cinders Cleveland sandstone New York trap

*Three kinds of concrete were used on each column, placed in three vertical sections in the order named, from top to bottom of column.

NOTE: Ties where used are of No. 5, B. & S. gage, steel wire, wound spirally on a pitch of 8 in.

TABLE 4b.—SCHEDULE OF FIRE AND WATER TESTS

Columns Protected by Hollow Clay Tile

Test No.	SECTION	Thick-ness of Tile, In.	*Kind of Tile and Method of Application	Filling
105	Plate and Angle	2	Surface clay, Boston district; semi-fire clay, N. J. district; Ohio shale $\frac{7}{8}$ -in. mortar on flanges Outside wire ties	No filling
106	Plate and Angle	2	Ohio semi-fire clay; surface clay, Chicago district; Ohio semi-fire clay $\frac{7}{8}$ -in. mortar on flanges Outside wire ties on upper half $\frac{7}{8}$ -in. wire mesh in horizontal joints in lower half	1:3:5 concrete, Portland cement Long Island sand Chicago limestone
107	Plate and Channel	4	Ohio shale; semi-fire clay, N. J. district; surface clay, Boston district 1-in. mortar on flanges Outside wire ties on upper half $\frac{7}{8}$ -in. wire mesh in horizontal joints in lower half	No filling

*Three kinds of tile were used on each column, placed in three vertical sections in the order named from top to bottom of column.

TABLE 4c.—SCHEDULE OF FIRE AND WATER TESTS

Columns Protected by Gypsum Block

Test No.	SECTION	Thick-ness of Block, In.	*Kind of Gypsum and Method of Application	Filling
108	Rolled H	2	Western gypsum (solid); Eastern gypsum (solid) $\frac{3}{4}$ -in. mortar on flanges Corrugated wall ties in horizontal joints in upper half $\frac{7}{8}$ -in. wire mesh in horizontal joints in lower half	1:1:4 Calcined gypsum Fine lake sand Broken gypsum block
109	Rolled H	4	Eastern gypsum (solid); Western gypsum (solid) $\frac{3}{4}$ -in. mortar on flanges $\frac{7}{8}$ -in. wire mesh in horizontal joints in upper half Corrugated wall ties in horizontal joints in lower half	1:1:4 Calcined gypsum Fine lake sand Broken gypsum block

*Two kinds of block were used on each column, placed in two vertical sections in the order named from top to bottom.

TABLE 4d.—SCHEDULE OF FIRE AND WATER TESTS
Column Protected by Plaster on Metal Lath

Test No.	SECTION	PROTECTION
110		Two 2-coat layers of Portland cement plaster; inner layer $\frac{3}{8}$ in. thick, on woven wire lath; outer layer 1 in. thick, on expanded metal lath. $\frac{3}{4}$ -in. air space between layers. Proportion of plaster, 1 part Portland cement, $\frac{1}{10}$ part hydrated lime, $2\frac{1}{2}$ parts coarse lake sand

TABLE 4e.—SCHEDULE OF FIRE AND WATER TESTS
Reinforced Concrete Columns
Effective Length, 12 ft. 8 in.

Test No.	SECTION	Mixture	*Kind of Concrete	**Effective Area, Sq. In.	Reinforcement, Percent of Effective Area
111		1:2:4	Chicago limestone Meramec R. gravel Chicago limestone	Concrete, 140 Steel, 4.00	Vertical, 2.78 Lateral, 0.14
112		1:2:4	Chicago limestone Meramec R. gravel Joliet gravel	Concrete, 127 Steel, 6.00	Vertical, 4.52 Lateral, 0.13
113		1:2:4	New York trap Meramec R. gravel Rockport granite	Concrete, 129 Steel, 3.38	Vertical, 2.54 Lateral, 0.99

*Three kinds of concrete were used in each column, placed in three sections in the order named, from top to bottom of column.

**A depth of 2 in. all around deducted from the gross section for fire protection.

TABLE 4f.—SCHEDULE OF FIRE AND WATER TESTS**Unprotected Cast Iron Columns****Effective Length, 12 ft. 6¼ in.**

Test No.	SECTION	DETAILS	Nominal Area, Sq. In.	Least Radius of Gyration, r In.	$\frac{1}{r}$
114	Round Cast Iron, Vertically cast		14.45	2.38	63.2
115	Round Cast Iron, Vertically cast		14.45	2.38	63.2

IV. PLACING OF COVERINGS AND CONCRETE COLUMNS

The work was planned so as to reproduce as nearly as possible the conditions obtaining in building construction in point of methods of application and workmanship. This was done to make the results of the tests applicable without undue allowance for differences that otherwise might be deemed to exist between the test sample and a similarly constructed member in a building.

1. CONCRETE PROTECTIONS AND COLUMNS**(a) FORMS AND STAGING**

The wood forms were made of 1½-inch yellow-pine planks, with clamps spaced about 2 feet apart vertically. The round columns and coverings were cast in metal forms made of No. 12 (0.1094 inch thick) sheet steel, the forms being made into halves which were bolted together through angles riveted on their edges.

The form was supported within a staging extending to the top of the test column proper, the floor of the staging being used as a platform from which the concrete was placed and on which subsequently the form for the column head or column-head protection was erected. A view of the staging with column forms in place is shown in Fig. 11.

FIG. 11.—Forms and staging for placing concrete

FIG. 12.—Concrete mixer

(b) METHOD OF PROPORTIONING

The proportions were based on volume parts of the materials as taken from the bins, except that the Portland cement was measured in the original package, one bag, containing 94 pounds of cement, being taken to be 1 cubic foot. The sand and stone were measured in deep, steel wheelbarrows in 2 and 3 cubic-foot portions, the volume for each being determined by striking off the top with a board cut to the required shape (Fig. 12). In some tests, where the mixtures appeared lean in sand, two or three shovelfuls of sand were substituted for an equal amount of stone.

(c) MIXING AND PLACING

The concrete was mixed in a motor-driven batch mixer having a capacity of 6 cubic feet of mixed concrete. The materials were charged into the mixer in the following general order, subject to minor variations introduced by the different men in charge of the mixing: (1) 2 cubic feet coarse aggregate, (2) 2 or 3 cubic feet of sand, (3) 1 bag Portland cement, (4) 2 or 3 cubic feet of coarse aggregate, (5) water. Before admitting the water the materials were mixed dry for a period varying from $\frac{1}{4}$ to $\frac{1}{2}$ minute, the total time of mixing being limited to $1\frac{1}{2}$ minutes as a maximum and 1 minute as minimum. The water was measured by means of a gage glass and scale, the former connecting with a vertical measuring tank placed above the mixer (Fig. 12).

The concrete was discharged into wheelbarrows which were raised to the platform of the staging for discharge into the forms. The concrete was spaded along the inside of the form and the latter was tapped with a hand hammer to assist in obtaining a good concrete surface.

To obtain workmanship comparable with that on field-placed concrete, several experienced men connected with local construction companies were at different times placed in charge of the mixing and placing, about one-half of the total number of concrete coverings and columns being thus placed. The methods thus introduced were followed in the mixing and placing of the concrete for the remaining columns.

2. PLASTER ON METAL-LATH PROTECTIONS

(a) PLACING OF LATH

For test No. 23 the lath for both the outer and the inner layer was supported on round bars held upright by iron clips made from $\frac{1}{8}$ by 1 inch flat bars and secured to the structural-steel section. This method proved very cumbersome, and the lath for the other steel columns was supported on $\frac{3}{8}$ by $\frac{3}{4}$ inch channels held in vertical position by wire ties. The high-ribbed lath was supported directly on the metal.

The lath was placed around the column in horizontal courses with laps of about 3 inches. Horizontal joints between sheets had generally shorter laps. All laps were wired with No. 18 wire ties placed 3 to 6 inches apart vertically and one on the middle of each side of the horizontal joints.

(b) APPLYING THE PLASTER

The plaster coats were of the maximum thickness practicable with the given mixture of plaster. The first coat of a layer was allowed to set two or three days before applying the second coat.

The lathing and plastering were done by experienced men obtained through a local plastering contractor.

3. HOLLOW CLAY-TILE AND BRICK PROTECTIONS

(a) PROPORTIONING OF MORTAR

The mortar was proportioned by volume parts of loose materials. The proportion used was 1 part Portland cement, 1 part stiff lime putty (slaked lump lime), and 4 parts fine bank or lake sand.

(b) PLACING OF TILE AND BRICK

The coverings were detailed in advance, and tile of the required size was supplied when possible. All cutting of tile and brick was done on the job with the hammer or trowel. A view of tile and gypsum-block protections under construction is shown in Fig. 13.

The work was done on a contract basis by a mason contractor, and it is thought that the workmanship obtained approximates that secured in good building practice.

(a) PLACING OF CONCRETE FILLING

Where concrete filling between the tile and column was used, the tile was held in place by clamping both ways every 2 feet, boards being placed along the protection inside of the clamps. The filling was placed from the platform of the staging shown in

FIG. 13.—Placing of clay-tile and gypsum-block protections

Fig. 11, the whole space being filled in one continuous operation. The clamps effectively held the tile against the pressure of the wet fill, and very few mortar joints were broken from this cause.

4. GYPSUM-BLOCK PROTECTIONS

(a) PROPORTIONING OF MORTAR

The proportion of mortar used, 1 : 3, neat, unfibred, calcined gypsum and fine lake sand, was the richest mixture that would work satisfactorily under the trowel, the materials being measured by loose volume.

(b) PLACING OF BLOCK

The blocks were cut from standard-size partition blocks with a hand saw, the resulting pieces being generally all used either in the covering or filling. The mortar joints in the covering and between the blocks and flanges of the test column were well filled.

(c) PLACING OF FILLING

Where wet filling was used the dry materials were first turned three times by hand and then mixed with water in small batches. The filling was placed as the blocks were laid up, two courses being generally filled at one time.

The placing and filling of the gypsum coverings were done by a fireproofing contractor employing men experienced in handling the given material.

5. METHOD OF STORAGE

Normal air storage was used for all columns and for the greater number of auxiliary test specimens.

The test columns were stored in the testing room, the temperature of which during the summer months was about the same as that of the outside air, and varied during the winter months from 5 to 25° C.

The auxiliary test specimens, consisting of 8 by 16 inch concrete cylinders and 2-inch mortar cubes made from the material of the coverings as placed, were arranged in tiers and stored near the test columns. In some laboratory tests with mortar, the briquets and cubes were stored for periods in damp closet or water as stated in Section V (Table 28, Appendix D, p. 356).

V. AUXILIARY TESTS OF MATERIALS

The results of the auxiliary tests give information on the physical, chemical, and thermal properties of the materials used in the columns and coverings. Where generally accepted standards in the form of specifications exist some comparable measure of quality is thereby afforded. For the majority of the materials no general specifications have as yet been developed, and their representative character must be determined by the extent of their use and the methods employed to obtain material of average quality.

The large number of tests of concrete, mortar, and plaster give important information on their properties and variability as made under conditions approximating those obtaining in building construction.

1. TESTS OF STRUCTURAL, BAR, AND WIRE STEEL

Results of tension tests are given in Tables 5 and 6 (Appendix D).¹ The specimens of structural steel, about 7 inches long and $\frac{3}{8}$ to $\frac{1}{2}$ inch wide, were cut before test from the upper, enlarged portion of the column section by drilling and sawing. They were finished to uniform width over a gage length of 2 inches. Tests of bar steel were made on the full sections of the bars used except as noted in Table 5 (p. 336).

Specimens for hardness tests were taken where tension specimens were difficult to obtain, and results of tests are given in Table 7 (p. 338).

Chemical analyses of structural and rivet steel are given in Table 8 (p. 339).

The column steel was of the grade generally used for structural purposes. Of the 77 structural-steel columns used in the tests, 36 were donated by the manufacturers and 22 were bought from the same sources, with no specifications as to quality of material. The steel for the remaining columns, which were purchased at a later date, was specified to conform with the specifications for structural steel for buildings of the American Society for Testing Materials. Correspondence with the manufacturers indicates that all of the steel was made by the open-hearth process and that,

¹ The tabular matter for this section is placed in Appendix D (p. 334-364).

with the exception above noted, the specifications followed were manufacturer's standard specifications for steel for railway bridges or for medium open-hearth steel, tensile strength 55 000 to 70 000 pounds per square inch. A few of the test results are above or below the specification limits by 10 per cent or less, a variation that may be allowable, considering that most of the specimens secured were smaller than the standard size.

The metal for the reinforcing bars was specified to conform with specifications for billet-steel concrete reinforcing bars of the American Society for Testing Materials, structural grade. The spiral hooping was of hard-drawn wire of high tensile strength (87 400 pounds per square inch).

2. TESTS OF CAST IRON

Results of transverse and tension tests are given in Table 9. The specimens of the horizontally cast columns were cut from the projecting flanges in the upper 3 feet or head of the column. Those representative of the metal in the vertically cast-iron pipe columns 10A, 114, and 115 were cut from a duplicate column. Results of chemical analyses of the iron in this column are given in Table 10 (p. 339).

The iron for the horizontally cast columns was of gray foundry pig with admixture of machinery casting scrap. The mixture used for the cast-iron pipe column and caps was the same as that regularly used in the manufacture of cast-iron water pipe. The following analyses of the mix were furnished by the makers:

	Hori- zontally cast columns	Vertically cast—			Hori- zontally cast columns	Vertically cast—	
		Columns	Caps			Columns	Caps
Silicon.....	2.04	1.60	1.70	Phosphorus.....	.714		
Manganese.....	.45	.34	.32	Sulphur.....	.108	.081	.060
Combined carbon..	.72						

For the iron in the horizontally cast columns, the analyses and test results indicate conformity with accepted specifications for gray-iron castings of medium weight. For the vertically cast columns, tests of specimens cut from one end of the duplicate column gave transverse and tension values about equal to the specification limits and those of specimens cut from the other end gave values lower by about 15 per cent.

3. TESTS OF PORTLAND CEMENT

Tests of the Portland cement used in the columns and coverings were made by the Washington and Pittsburgh laboratories of the Bureau of Standards and by the R. W. Hunt Co. The results are given in Tables 11 to 13 (Appendix D). Sample Nos. 1 to 5 and B-1 and B-2, Table 11; H-1 to H-6, Table 12; and H-1 to H-3, Table 13, were all from the mill shipment received in April, 1916. Sample Nos. H-7 and H-8, Table 12, were of a later purchase of the same brand as the original mill shipment. Sample No. 12, Table 11, was of the Portland cement used in filling the pipe column of test No. 12. All samples were individual sack samples taken from the portions of the shipment used at the given time.

The time of setting was determined with the Gillmore needles in all tests. Other details of the test conformed with the specifications published by the American Society for Testing Materials, 1916. As based on average results, the tests indicate conformity with the given specifications within the pertaining limits and tolerances, excepting the test of sample No. 12. The latter gave low results in points of fineness and seven-day tensile strength.

4. TESTS OF SAND

The chemical and physical properties of the concrete and finer sands are given in Tables 14 to 17 (pp. 342-344).

The specific gravity was determined by measuring the volume of benzene displaced by a given dry weight of sand in a Le Chatelier apparatus.

The weight per cubic foot is that of the loose dry sand poured into the measure without shaking or bumping and leveled off even with its top.

The weight of the dry contents of a cubic foot of the sand as used on the work was somewhat less than that given in the tables due to the moisture contained which caused it to assume greater bulk than in the dry condition, the difference being about 10 per cent with coarse sand of 3 per cent moisture content. With the finer sands the decrease in weight was about 25 per cent as caused by a moisture content of 4 per cent.

The percentage of computed voids was taken equal to the following relation of values:

$$100 \left(\frac{\text{Weight per cubic foot (dry)}}{1 - \text{specific grav.} \times \text{weight of cubic foot of water}} \right).$$

The granular analysis gives percentages, by weight, passing the given sieve openings.

5. TESTS OF COARSE CONCRETE AGGREGATES

Chemical analyses and physical properties of the broken stone, gravel, and cinder used for concrete aggregate are given in Tables 18 and 19 (p. 344).

The physical properties were determined according to the methods given for tests of sand, except that apparent specific gravity was determined in water in a graduated vessel on the saturated aggregate and was taken as the ratio of the dry weight to that of the water displaced.

6. SOURCE AND CLASSIFICATION OF CONCRETE AGGREGATES

The sands and coarse aggregates used for concrete were analyzed for mineral composition and their chief constituents are given in Table 20 (p. 345). Further information on their mineral composition, geological origin, geographic location, and the method of preparing them for commercial use are given in the following descriptions:

(a) FOX RIVER SAND

This sand was obtained from a sand and gravel deposit about 3 miles east of Elgin, Ill. After crushing of the oversized gravel the whole is washed and screened to size. The grade is known in the Chicago market as coarse "torpedo" sand, the general maximum size of grains being $\frac{1}{4}$ of an inch. It is clean and very sharp, only 15 per cent of the grains being rounded.

The sand is of glacial origin, and its principal mineral constituents are calcite and dolomite, 44 per cent; quartz and chert, 39 per cent. Ferromagnesian, chiefly biotite, pyroxene, and hornblende, constitute 4 per cent. No free clay matter was present, although about 2 per cent of hydrous aluminum silicates were contained in the calcite and dolomite.

(b) JOLIET SAND

This is a clean, sharp, glacial sand obtained from a sand and gravel deposit at Rockdale, Ill., the method of preparation and

grading being the same as for Fox River sand. The chief mineral constituents are calcite and dolomite, 47 per cent; quartz, 42 per cent; feldspar, chiefly orthoclase (KAlSi_3O_8), 7 per cent; ferromagnesian, chiefly as biotite, mica, and hornblende, 3 per cent.

(c) **MERAMEC RIVER SAND**

This sand is obtained incidentally to screening and washing of gravel dredged from the bed or banks of the Meramec River at Drake, Mo. It consists almost wholly of quartz and chert grains formed from sandy chert by rolling in the stream bed. A small amount of calcite (1 per cent) is present as a coating on some of the grains. The quartz grains are fully rounded, while the chert is partly rounded or subangular.

(d) **LONG ISLAND SAND**

This sand was obtained from banks near Roslyn, Long Island, N. Y., and was screened and washed. It is a glacial sand formed from the crystalline rocks of New England. The principal constituents are quartz and feldspar, with minor amounts of ferromagnesian, slate, magnetite, and cinders. The cinders are not an original constituent of the sand. The sand is clean, sharp, and angular, the grains showing little rounding.

(e) **PELEE ISLAND SAND**

This sand was dredged by boat from the bottom of Lake Erie, at Fish Point, Pelee Island, Ontario. It is of glacial origin with sharp and angular grains that show almost no rounding. Of the mineral constituents, calcite and dolomite constitute 32 per cent; chert and quartz, 51 per cent. Clay is present to the extent of 4 per cent, of which about one-half is free clay matter and the rest shale.

(f) **PLUM ISLAND SAND**

This sand was dug on a beach at Plum Island on the Massachusetts coast and was not screened or washed. It is of glacial origin although much modified by ocean waves and currents. About 50 per cent of the grains have been worn to a subangular condition and about 3 per cent are well rounded. It is fine-grained and contains no free clay material, the mineral composition being otherwise closely identical with that of the Long Island sand.

(g) CHICAGO LIMESTONE

The stone, which was crushed to nominal $\frac{3}{4}$ -inch size was quarried at Gary, Ind., from a ledge of the Niagara formation. It is a true dolomite (calcium magnesium carbonate), carrying a moderate amount of clayey impurities.

(h) JOLIET GRAVEL

This is obtained from the same source as Joliet sand. It is a glacial gravel, consisting largely of limestone pebbles, of which 20 per cent are rounded and 60 per cent subangular or partly rounded. Mineralogically the composition is 86 per cent dolomite, containing about 5 per cent clayey impurities; 5 per cent quartz in the form of sandstone and quartzite; 2 per cent quartz, 5 per cent feldspar, and 2 per cent ferromagnesian silicates as pebbles of granite, gabbro, and other basic igneous rocks.

(i) MERAMEC RIVER GRAVEL

The pebbles consist almost entirely of chert, an amorphous form of silica containing a variable amount of water in chemical combination. In the sample examined, 85 per cent of the pebbles show no trace of an earlier structure, 4 per cent are sandstone replacements, 9 per cent are limestone replacements, and 2 per cent have once been shale. The pebbles are colored brown with limonite ($2\text{Fe}_2\text{O}_3 \cdot 3\text{H}_2\text{O}$).

(j) NEW YORK TRAP ROCK

This was quarried at Haverstraw, N. Y., from the northern end of the sill of diabase which extends along the western shore of the Hudson River from this point to Staten Island, forming the Palisades of the Hudson. It is an intrusive igneous rock of finely crystalline texture. Of the minerals present, feldspar, chiefly plagioclase ($\text{CaAl}_2\text{Si}_2\text{O}_8$), with some albite feldspar ($\text{NaAlSi}_3\text{O}_8$), constitute 71 per cent; ferromagnesian as augite, 19 per cent, and as olivine, 7 per cent. In addition to 2 per cent of magnetite (Fe_3O_4), there are present scattered crystals of quartz (SiO_2) and apatite [$\text{Ca}_5\text{F}(\text{PO}_4)_3$].

(k) ROCKPORT GRANITE

This stone was quarried at Rockport, Mass., and is sometimes known as Cape Ann granite. It is an intrusive igneous rock that crystallized under great pressure from a fused condition.

The mineral composition is 59 per cent feldspar, chiefly orthoclase (KAlSi_3O_8) with some albite ($\text{NaAlSi}_3\text{O}_8$); 35 per cent quartz; 5 per cent ferromagnesian silicate, as hornblende [$(\text{Fe}, \text{Mg})\text{SiO}_3$].

The feldspars show slight alteration to kaolin and chlorite. The quartz contains many tiny cavities filled with gases and water occluded when the rock was formed.

(l) CLEVELAND SANDSTONE

The stone was quarried at Amherst, Ohio, and geologically is known as Berea sandstone or Berea grit. It is pure sandstone, 98 per cent consisting of subangular grains of quartz.

(m) HARD-COAL CINDERS

This is an anthracite cinder representative of the product used for cinder concrete in New York, N. Y. It consists largely of a porous, fused mass of basic silicates, apparently high in lime and magnesia and low in iron. There is present about 10 per cent of unburned coal and 5 per cent unfused ash.

7. TESTS OF CONCRETE

The quality of the concrete secured is thought to represent that of field-placed concrete as nearly as it can be conveniently duplicated under laboratory conditions. The method used for proportioning the dry materials was more accurate and the time of mixing longer than now generally obtain on construction work. The range of consistency resulting from the method employed for mixing the concrete is deemed to be representative of current field conditions. Where concrete is properly placed in large building members its properties are probably more uniform than as given by tests of the relatively smaller cylinders, being for a given mixture and consistency, more nearly the average values obtained in the tests.

(a) TEST SPECIMENS

Four cylinders of 8-inch diameter and 16-inch length were molded of the concrete mixed for each concrete covering, filling, or column. Cylinders were also taken of the concrete in the head protections of some of the columns where the method of mixing had been modified so as to give information on the effect of a lower water content and of a longer mixing period.

The molds were of cast iron with machined base plates. After the concrete had set and before removal of the molds, the cylinders were capped with a plastic mixture of Portland cement and calcined gypsum applied even with the top of the molds by means of a finished iron plate which remained in place until the mixture had set. In molding, care was taken to obtain the same consistency of concrete in the cylinders as in the column. Except for spading with a thin tool near the surface, the concrete was not puddled or tamped in the mold.

FIG. 14.—Concrete cylinder after test

(b) PER CENT WATER IN CONCRETE MIXTURE

The water introduced in mixing was measured at the mixer, and, for all but the first 15 columns covered, that present in the aggregates was determined from samples taken on the day the concrete was mixed. The dry weight in pounds per cubic foot of the aggregates as used was determined in a number of separate tests with each aggregate in which the same methods of filling the measures were used as were employed in proportioning materials for the concrete. The net weight per bag of Portland cement was found to be very nearly 94 pounds. The water content of the

mixture is the sum of the water introduced, and that contained in the aggregates and in the tables and diagrams is expressed as a percentage of the total dry weight.

FIG. 15.—Average and range of compressive strength of 1:2:4 and 1:2:5 concrete

While the above method gives a fair comparison in point of consistency of concrete made from the same aggregate, the comparison between those of different aggregates is less direct, due to differences in specific gravity and absorption.

FIG. 16.—Average and range of compressive strength of 1:3:5 concrete

(c) TESTING OF CONCRETE CYLINDERS

Two cylinders of each set were tested in compression at the age of four weeks and two at the time the corresponding column was tested. In the case of the latter, the compressive load was applied by increments of about 10 000 pounds, and the deformation measured over a 10-inch gage length by means of an Ames dial gage mounted in a pivoted frame (Fig. 14). The results of tests on concrete mixed for the protections and columns are given in Table 21 (p. 346) and those of the concrete in some of the head protections in Table 22 (p. 350).

(d) COMPRESSIVE STRENGTH OF CONCRETE

The diagrams in Figs. 15 and 16 give the average and range of compressive strength of the cylinders taken from the concrete mixed for the columns, their covering and filling. In Figs. 17 and 18 the general effect of consistency on compressive strength is

FIG. 17.—Effect of consistency on compressive strength of concrete. Average age, 28 days

FIG. 18.—Effect of consistency on compressive strength of concrete. Average age, 490 days

shown. Herein are also included the tests (Table 22) made to study the effect of a lower water content. Fig. 19 gives a comparison of the strength of concrete mixed for 1 and for 2 minutes.

FIG. 19.—Effect of time of mixing on compressive strength of concrete

The mixtures having a water content of 9 to 11 per cent were of quaking or mushy consistency, the drier ones being discharged from the mixer with considerable difficulty. The higher water percentages (12 to 15) represent varying degrees of fluid consistency. The restricted space in which some of the concrete was placed made fluid or semifluid consistencies necessary in order to secure proper placement, although beyond the point where water collects above the concrete after discharge little is gained in this particular by the excessive use of water. For a given consistency the cinder and, to less extent, the sandstone, required a larger amount of water than the other aggregates. due to greater absorption.

For the concrete in the columns and coverings (Table 21) the amount of water introduced was determined by the man in charge of the mixing and placing of concrete for the given column, the water for the first batch being added by increments until the desired consistency was attained. For the concrete in the head

protections (Table 22) the amount of mixing water necessary for any desired total moisture content was generally predetermined, knowing the dry weight of the aggregates employed and their moisture content at the given time.

(e) MODULUS OF ELASTICITY OF CONCRETE

A comparison of values for 450, 650, and 850 pounds per square inch is given in Fig. 20 as obtained from Table 21. The modulus of elasticity is taken as the ratio of the given unit stress to the corresponding total unit deformation. In Fig. 21 the variation with ultimate compressive strength is shown, and in Fig. 22 the effect of water content of the mixture. The plots are based on values of modulus of elasticity given in Tables 21 and 22 for stress of 650 pounds per square inch.

In all tests the age of the concrete was over one year and the values of modulus of elasticity found are on the average higher than those generally taken to obtain for concrete at lower age.

FIG. 20.—Modulus of elasticity of concrete at 450 to 850 lbs. per sq. in.

FIG. 21.—Modulus of elasticity of 1:2:4 concrete at 650 lbs. per sq. in. Variation with ultimate compressive strength

FIG. 22.—Effect of consistency on modulus of elasticity of concrete at 650 lbs. per sq. in.

8. TESTS OF LIME

Chemical analyses and physical properties of quicklime and hydrated lime are given in Table 23. The analyses disclosed a high calcium lime. The quicklime passed standard specifications but the hydrated product failed in point of CO² content, fineness, and soundness. The latter was used only as a minor component in the Portland-cement plaster protections.

9. TESTS OF CALCINED GYPSUM

Analyses and tests of eastern and western gypsum are given in Table 24 (p. 351).

10. TESTS OF MORTAR, PLASTER, AND FILLING

Results of compression tests on the mortar and plaster used in the coverings are given in Tables 25 to 27 and a further comparison of average and range of compressive strengths is given in Fig. 23. The specimens for these tests were taken of the materials as mixed by the workmen. They were stored in air without artificial drying or curing.

FIG. 23.—Average and range of compressive strength of mortar and plaster

The results of a series of mortar tests made by the Pittsburgh laboratory of the Bureau of Standards are given in Table 28 (p. 356). The same materials, proportions, and average percentage of water were used as for the mortar and plaster of the column coverings.

Table 29 (p. 354) gives results of compression tests on cylinders molded of the mixture with which gypsum-block protections were filled.

11. TESTS OF HOLLOW CLAY TILE

(a) CLASSIFICATION AND DESCRIPTION

The characteristics of the hollow tile employed as column covering are given in Table 30 (p. 357).

The straight partition tiles, Nos. A to E, were nonporous; that is, burnt without sawdust or other filling. The curved tiles, Nos. F, G, and H, in point of method of manufacture, were porous.

(b) POROSITY AND ABSORPTION

The porosity of the burnt clay (Table 31) was obtained by drying samples of tile to constant weight at 112° C and immersing in hot water under vacuum of 24 inches of mercury for about four hours, the porosity being based on the ratio of volume of water absorbed to that of the test specimen. The weight of water absorbed is also expressed in Table 31 (p. 358) as a percentage of the dry weight.

(c) COMPRESSIVE AND TRANSVERSE STRENGTH

Table 31 gives results of compression tests of all types of hollow tile used in the column coverings. The curved tile was tested on end and the partition tile was tested both on end and on edge. By the latter method the load was sustained by the outer shells only and the average maximum unit loads developed are in all cases lower than for the tests made with the tile on end.

The transverse tests (Table 32) were made with center load and supports 10 inches apart. Failure generally occurred on inclined planes extending from points on the lower surface upward toward the loading line. In calculating the maximum outer fiber stress and horizontal shear, the moment of inertia and other properties of the section were obtained, using the measured dimensions of each test specimen.

(d) TEMPERATURES OF VITRIFICATION AND FUSION

In determining temperatures of vitrification a number of pieces of each kind of tile were dried and weighed as for the porosity

tests, and heated to 800° C in a muffle furnace, after which samples were withdrawn at temperature intervals of 30° C, allowed to cool slowly, and tested for porosity. The temperature of vitrification was taken as the point or region of minimum porosity.

In preparing samples for the fusion test, fragments of a number of tiles from the same clay were broken up and a sample obtained by quartering. This was ground and again sampled, the final sample being ground to pass through an 80-mesh sieve and made into cones $\frac{7}{8}$ of an inch high. These were fired in a small pot furnace between standard cones, the temperature of the furnace being determined with platinum thermocouples. Softening was indicated when the tip of the test cone went over. Fusion was taken as occurring at the stage when the tip of the cone reached the level of its base.

The temperatures of vitrification and fusion determined for the hollow tile and brick are given in Table 33 (p. 361).

12. TESTS OF BRICK

Results of tests giving the porosity, absorption, compressive and transverse strength of the brick used in the brick protections are given in Tables 34 to 36 (p. 361-362).

The brick was made in the Chicago district of calcareous surface clay and burnt to medium hardness, the color being generally light red. The average dimensions in inches were $2\frac{1}{4}$ by $3\frac{3}{4}$ by 8.

13. TESTS OF GYPSUM BLOCK

Results of porosity, compression, and transverse tests of gypsum block are given in Tables 37 to 39 (p. 363-364).

The porosity tests were made on pieces of block dried for 24 hours at 60° C and saturated in kerosene under vacuum, the method being in other respects the same as given above for clay tile.

In calculating modulus of rupture the outside dimensions of the block were used, the scoring and imprints being neglected.

14. TESTS OF GYPSUM WALL BOARD

Results of transverse tests on gypsum wall board, in the dry condition and after saturation in water, are given in Table 40. The strength of the board was limited by the tensile strength of the paper facing, the latter developing greater resistance when the strain was parallel with the grain of the paper.

VI. DESCRIPTION OF FURNACE AND RELATED EQUIPMENT

The furnace and accessory equipment used in the tests were specially designed for the purpose and consist of a carriage and traveling crane for handling the test columns, a furnace in which the columns are subjected to fire, a ram with restraining frame for applying load, and hydrant with means for applying water in the fire and water tests.

1. BUILDING

The tests were made at Underwriters' Laboratories, Chicago, Ill., in a building designed for work of this character, a partial plan of which is given in Fig. 24 and a sectional elevation through the center in Fig. 25.

The central portion of the building, wherein the furnace for testing columns is located, is in one story about 37 feet high, an intermediate operating floor being located in the outer bays. Sliding skylights in the roof of the building above the furnace provide means for ventilation. The operator's station on the upper floor is inclosed within fire-resistive partitions, with openings protected by wired glass.

2. APPARATUS FOR HANDLING COLUMNS

A carriage traveling on the lower flange of the I beams placed above the storage bays for the columns is used for transferring the latter to the traveling crane. The column to be moved is lifted on threaded rods passing through the top plate of the column and plates supported on a truck contained within the carriage. This truck carries the column from the traveling crane to its location in the furnace, on I beams (*a*, Fig. 25) spanning the space above the furnace (Fig. 24).

The column is finally attached to the head of the loading ram and lowered into position.

3. LOADING APPARATUS

(a) LOADING RAM

The special hydropneumatic ram used for loading the columns was designed to maintain a constant load during the test and to develop characteristic deformation at the point of failure.

The main ram is 36 inches in diameter and is connected with a smaller lifting ram supported on its upper head. The operating

FIG. 24.—Plan of testing room

medium is water maintained at the required pressure by an air compressor which discharges into steel pressure tanks connected with the ram cylinder through a 6-inch main. A pump is also provided to maintain the water at any given level.

Provision is also made for applying load by pumping water directly into the main cylinder. The gate valve in the main leading to the air-pressure tanks is in this case closed and regulation of pressure obtained with a spring relief valve.

(b) CONTROLLING DEVICES

A gate valve is placed in the pipe connection between the pressure tanks and the main cylinder, and also an automatic cut-off valve which shuts off the pressure tanks and releases the pressure in the main cylinder after a predetermined downward movement of the plunger of the ram has taken place. For the present tests the valve was set to begin shutting off pressure after the plunger had gone down 2 inches, the valve releasing the pressure after an additional $1\frac{1}{4}$ -inch downward movement. This caused maximum center deflection of the column at failure of about 15 inches. As a further precaution a pressure of not less than 400 pounds per square inch was maintained in the lifting cylinder.

The portion of the main cylinder chamber below the piston is connected with the outside air and also with a closed discharge chamber, with the latter through perforations in the cylinder lining. A depth of water is maintained in this part of the cylinder to act as a cushion in case the travel of the ram piston, following a sudden column failure, should extend to the end of the cylinder.

To obtain an approximate record of the load sustained by the column during test and during the period of failure, a steam-engine indicator was mounted on the control board (s, Fig. 27) with its pressure chamber connected with the main cylinder of the ram and the drum actuated by a cord attached to the ram plunger. The resulting card diagram indicated on axes at right angles, the pressures in the main cylinder and the corresponding positions of the ram plunger.

For convenience in operating, the valves for admitting and releasing pressure to all parts of the loading system are located at the control board (Fig. 27). Here are also located the load and pressure indicating gages and the automatic and manually oper-

FIG. 25.—Elevation of testing machine

ated controls for the starting switches of the air compressor and water pump. The main features of the installation are shown in Fig. 25, minor piping and details being omitted.

(c) **RESTRAINING FRAME**

The ram is supported from the upper transverse members of the frame, the lower set being embedded in the foundation pit. Each set consists of two pairs of 24-inch I beams with cover plates. Two vertical tension bars on each side carry the reactions from the applied column loads. The lower end of the ram is supported laterally in an intermediate cross frame.

(d) **BEARING DETAILS**

The ends of the test column are bolted to rolled-steel plates 2 inches thick, which are in turn bolted to a cast-steel foundation plate at the bottom and to the lower flanged head of the ram at the top. Between these and the column plates are placed adjustable bearings consisting of skew disks, the middle two of which can be turned to obtain even bearing.

(e) **CAPACITY AND CALIBRATION**

The machine has a load capacity of 545 000 pounds and has been calibrated by the Bureau of Standards, using a comparison bar whose load-deformation relation was determined in the 2 300 000 pound Emery testing machine at Washington.

The accuracy of loading within the range used in this series of tests can generally be taken to be within 1 per cent of the applied load, with a possible extreme variation of 2 per cent. The errors incurred are due to variation in the ram friction and in the indication of the test gages used for determining the load. The latter were calibrated in a dead-weight gage tester before or after each test.

4. TESTING FURNACE

(a) **COMBUSTION CHAMBER**

The chamber is 7 feet square and 12 feet high, exclusive of two shallow pits to receive falling material and carry off water during the fire-stream tests. The sides are formed by two stationary brick walls and two movable brick walls suspended in structural-steel frames from overhead beams by trolleys (Fig. 25). These walls are pulled open by means of a motor-driven hoist. The top of the furnace consists of heavy fire-clay blocks supported

by steel I beams and hangers. Some of the blocks are removable to permit installation of the test column. Concrete made of Portland cement and crushed fire brick and reinforced with rods and wire mesh was later substituted for fire clay in making additional roof blocks. The bottom of the chamber is formed by the protection provided for the lower bearing plate and the supporting members of the restraining frame.

The products of combustion are carried off through four flues extending from the furnace through the roof of the building. The furnace walls are provided with mica-glazed observation holes, so arranged near the top, middle, and bottom that all parts of the test column can be observed.

A general view of the furnace with a preliminary test column in place is given in Fig. 26.

(b) BURNERS

The furnace is heated by means of four primary blast burners extending in at the corners at the bottom of the chamber, and are all arranged to discharge in an inclined direction upward and toward the adjacent corner. The burners are supplied with gas and air through tubes connecting with mains located in a shallow pit under the floor outside of the furnace. Gas is supplied to the furnace through a 6-inch pipe connected to the city service mains. Air is supplied by an electrically driven blower.

The primary burners are operated with a smaller proportion of air to gas than is required for full combustion. The additional air needed to complete combustion and distribute the fire is supplied by secondary air inlets extending through the walls of the furnace at regular intervals toward the top, each inlet being provided with a regulating valve (Fig. 25). The air is so directed as to avoid impingement on the test column, and to set up a whirling motion of the furnace gases as an aid in securing uniform temperature distribution.

(c) OPERATING DETAILS

Means for regulating the gas and air supplied to the burners and the draft from the furnace chamber are located on the main floor. An emergency cut-off valve with electrical control is placed in the gas main leading to the furnace, with switches for operating it set at several points in the building.

A Hoskins direct-reading millivolt-temperature indicator with connections to four chromel-alumel furnace thermocouples is placed near the furnace for use in regulating its temperature.

FIG. 26.—General view of testing machine

- A—Gate to main cylinder.
- B—Air outlet from auxiliary tank.
- C—Water outlet from auxiliary tank.
- D—Water outlet from main tanks.
- E—Air outlet from main tanks.
- F—Water pump switch.
- G—Air compressor switch.
- H—Automatic pressure regulator.
- I—Water inlet to auxiliary cylinder.
- J—Water outlet from auxiliary cylinder.
- K—Air inlet to auxiliary tank.
- L—Water inlet to auxiliary tank.
- M—Water pump pressure.
- N—Water inlet to main tanks.
- O—Air inlet to main tanks.
- P—Air pump pressure.
- Q—Auxiliary cylinder pressure.
- R—Main cylinder pressure.
- S—Pressure indicator.
- T—Spring relief valve.
- U—Auxiliary tank pressure.
- W—Main tank pressure.

FIG. 27.—Control board

FIG. 28.—Fire-stream apparatus

FIG. 20.—Temperature-measuring instruments

5. FIRE-STREAM APPARATUS

The pressure head for the hose stream used in the fire and water tests is supplied by air pressure in a tank of 4500 gallons capacity. The water is applied through a standard Underwriter play pipe with nozzle $1\frac{1}{8}$ inches in diameter (Fig. 28). It is connected with the hydrant through about 20 feet of $2\frac{5}{8}$ -inch cotton, rubber-lined hose. A throttling valve is placed at the outlet to the hose and a pressure gage connected with the base of the nozzle by means of which the desired pressure during water application is maintained.

The water was applied to three sides of the column, the nozzle being moved back and forth on one side of the furnace and kept at a distance of about 20 feet from the center of the column (Fig. 24).

VII. TEMPERATURE MEASUREMENTS

All temperature measurements were made by the thermoelectric method, platinum-platinum rhodium being used in the thermocouples placed in the furnace and base metal in those on the test columns. The electromotive force was measured with indicating and recording potentiometers, and the corresponding temperature obtained from comparison calibrations with standard thermocouples whose temperature-emf relations were known.

The temperatures are expressed on the Centigrade scale. The Fahrenheit equivalents are given at the right of the time-temperature plots. A centigrade and Fahrenheit conversion table is given in Appendix F (p. 375).

1. INSTRUMENTS

A view of the instruments used is given in Fig. 29, wherein an indicating potentiometer or indicator is shown at *A*, a recording potentiometer at *B*, dial and push-button switches at *C* and *D*, respectively, and bus-bar distributing board at *E*.

(a) INDICATING POTENTIOMETER

The potentiometer consists essentially of a means for securing a known variable electromotive force and balancing it against the electromotive force to be measured.

A wiring diagram of a simple form of this instrument is shown in Fig. 30. Current from a battery flows through the resistance

ABCDE, of which the portion *BCD* is a slide wire. A scale is attached to the slider, giving the potential drop between points on the slide wire and the point *B* for a given value of the battery current. The latter is adjusted to the given value by opposing the electromotive force of a standard cell to the drop of potential over the resistance *AB* and varying the resistance *R* until the galvanometer included in the circuit shows no deflection. In a similar manner, an unknown electromotive force as that of a thermocouple, is balanced against the potential drop over some portion *BC* of the slide wire and its magnitude read from the attached scale.

Corrections for variation in the temperature of the cold end of the thermocouple require in general the addition of a small electromotive force to the observed one. With the potentiometer used

FIG. 30.—Wiring diagram of indicating potentiometer

in these tests, this could be accomplished mechanically at the time of making the observation by setting off the corresponding electromotive force on the cold-junction compensator, which has the same effect as moving the point *B* up the scale the same amount.

The potentiometer indicator used in these tests is equipped with two scales, one reading from 0 to 16 and the other from 0 to 80 millivolts. The accuracy of this particular instrument was found to be within one-fourth of 1 per cent of the total range at all points on the scale.

In the above method of measurement the indication is independent of the resistance of the thermocouple and lead wires, and no errors are introduced by variations in resistance caused by temperature changes in them.

(b) RECORDING POTENTIOMETER

The wiring diagram of this instrument is an exact equivalent of that of the indicating potentiometer (Fig. 30), the electromotive force of the thermocouple being automatically balanced and recorded. It has scales of the same range as the indicating instrument, and a 13-point selector switch provides connections for a maximum of 12 thermocouples. On the remaining switch point the battery current is automatically adjusted to the correct value. Records are made at intervals of one minute. Where the temperature differences between the different couples connected are not too large, records can be made every half minute.

(c) ACCESSORIES

The accessories consist of a bus-bar board that was used for connecting any desired set of thermocouples with the recording potentiometer, a 12-point double-pole, double-throw, push-button switch, by means of which the couples connected with the recorder could be read on the indicator, and a 24-point double-pole dial switch for connecting thermocouples directly with the indicator.

2. FURNACE TEMPERATURES

Because of the high temperatures attained in column tests of long duration it was deemed advisable to use rare metal (Pt-90 Pt 10 Rh) thermocouples for the measurement of furnace temperatures.

(a) LOCATION OF FURNACE THERMOCOUPLES

In general, four couples were used to measure furnace temperatures, placed two on each of two levels at 3 feet and 9 feet above the fireproofing about the base of the column (Fig. 31). The couples on the 3-foot level passed through the furnace walls near the northwest and southeast corners; those at the 9-foot level near the northeast and southwest corners. Laterally the junctions of the furnace pyrometers were placed 15 inches from the nearest point on the column. The above symmetrical distribution is such that the center of gravity of the couple locations is identical with that of the portion of the furnace chamber above the fireproofing line at the base of the column.

The curves on the time-temperature plots (Appendix B) corresponding to the respective furnace couples are designated as *L-NW*, *L-SE*, *U-NE*, and *U-SW*, the letter preceding the dash indicating the level (*L*, lower; *U*, upper) on which the corresponding couple was placed; those following it, the corner nearest to which the couple entered the furnace.

The same system of designation is extended to include the few cases where couples were used in locations other than those mentioned. An exception is found in test No. 20, in the case of curves L_4 and U_4 . The corresponding couples measured furnace temperatures at points $1\frac{1}{2}$ inches from the column face, 3 feet and 9 feet, respectively, above the fireproofing line. They were made of No. 16 iron and constantan wires, with the junctions exposed directly to the furnace atmosphere.

(b) THERMOCOUPLE MOUNTINGS

Details of the thermocouple mounting are shown in Fig. 32. The use of rare-metal couples necessitated porcelain protecting tubes, which were covered with woven-asbestos sleeving to prevent breakage from sudden temperature changes. The alundum tube, with its winding of asbestos cord covered the outer 2 feet of the porcelain tube and served as a protecting sleeve where the mounting passed through the furnace walls.

The porcelain protecting tubes differed somewhat in wall thickness, as furnished by three different makers. The maximum and the minimum wall thickness were $\frac{5}{32}$ and $\frac{1}{16}$ of an inch, respectively. About 85 per cent of the total number of tubes used had a wall thickness of very nearly $\frac{1}{8}$ of an inch.

Practically all of the woven asbestos sleeving was bought in two lots, both of the same nominal size. The first lot, comprising approximately $\frac{3}{4}$ of the sleeving used, ran about 15 feet per pound, the second lot about 24 feet per pound, as unwound from the spool.

(c) CONNECTIONS TO INSTRUMENTS

Permanent leads of No. 18 copper fixture wire were installed in a conduit leading from the case housing the recording potentiometer to the base of the furnace wall, from whence branches extended up to the thermocouple points. Hubbell receptacles were provided at the couple outlets (Fig. 31), which received the Hubbell plugs terminating the short leads attached to the cold ends of the furnace couples (Fig. 32). At the instruments the leads led to the bus-bar board, where each of the four furnace couples was connected to three recorder points, and also through the push-button switch with the indicator.

The temperatures of the cold junctions in the heads of the furnace pyrometers were measured by thermocouples formed of one of the copper leads in each head and No. 18 constantan wires

FIG. 32.—Detail of furnace thermocouple mounting

(Fig. 32), which were installed in the same conduit as the copper leads, all wires extending to the case housing the recorder, from whence copper wires led through the dial switch to the indicator.

3. COLUMN TEMPERATURES

The base-metal thermocouples used for measuring column temperatures were of No. 16 (B. & S. gage) iron, No. 16 and No. 18 constantan, and No. 18 nickel wires, in the combinations, iron-constantan and nickel-constantan, 90 per cent of the total number of couples used being of the iron-constantan combination. As supplied, a portion of the wire carried a thin waterproofed asbestos covering, while the remainder was bare. As used on the column all wire was covered with closely fitting asbestos sleeving.

(a) ATTACHMENT OF THERMOCOUPLES TO COLUMN

A preliminary test consisting in subjecting to heat in a gas-fired furnace two concrete-covered lengths of wrought pipe, with thermocouples placed on the metal and in the covering, indicated that the proposed method of measuring column temperatures gave, in general, reliable results. Two methods of attaching the couples to the pipe section were tried out and appreciable differences of indication found to be caused by the given variation in this detail. By the one method, which will be termed "peening" the ends of the individual couple wires were placed in closely-fitting holes drilled in the metal and secured in place by driving in the metal around the holes, using a slotted punch placed in turn over each wire. Couples attached by the other method, and referred to as "insulated," had fused or twisted junctions that were laid against a piece of mica about 0.01 inch thick placed between it and the metal.

Since it was apparent that the peened couples gave more nearly the true temperature of the metal, the couples on the columns prepared subsequent to the preliminary test were attached by peening, except that one or two insulated couples were added at certain important locations as a check on the adjacent peened couples.

The couples placed before the preliminary test was made were all insulated and include those under the concrete covering or filling of all partly protected columns, test Nos. 14 to 22, inclusive, and the couples of the concrete-protected column of test No. 28 and those on the vertical bars of the reinforced-concrete column of test No. 73.

To decrease the danger of injury to the peened junctions in placing the coverings and in testing the columns, the region immediately surrounding the point of attachment was covered to a thickness of about $\frac{1}{8}$ of an inch with asbestos wool loosely placed under a shield of sheet iron 1 inch square and 0.015 inch thick. The couple wires, protected by the applied sleeving, were held firmly in place under a strap-iron clip screwed to the column near the junction. The same clip also held the shield over the asbestos wool in place. Similar clips, placed about 18 inches apart, held the couple wires on the column up to 1 foot below the top bearing, where they were all led out through a $1\frac{1}{4}$ -inch pipe (Fig. 31).

The asbestos wool and shield were omitted over the insulated junctions, since it was thought that they would not be subjected to as large strains as the junctions that were peened.

As it was very difficult to attach couple junctions to the inside of hollow steel or cast-iron columns by the peening method, the junctions were laid against the metal and protected with asbestos wool, as described above.

In placing couples in the column coverings or in locations other than on the metal, suitable means were used for supporting them in place, the methods varying with the conditions presented by the different types of columns and coverings. In all cases the details were so arranged as to reduce to a minimum heat interchange between the couple junction and its support as well as between the junction and the couple wires. The latter was accomplished by placing a sufficient length of the wires immediately leading from the junction in a plane where, with the contemplated fire exposure, the same temperatures would normally obtain as at the junction.

(b) LOCATION OF COLUMN THERMOCOUPLES

In all tests, except in those of timber columns, the thermocouples were spaced vertically in four general locations, *B*, *N*, *M*, and *T*, 1 foot 6 inches, 4 feet 6 inches, 7 feet 6 inches, and 10 feet 6 inches, respectively, above the fireproofing line at the base of the column (Fig. 31). The lateral locations are numbered 1, 2, 3, 4, etc., and marked by small solid circles on the sectional location diagrams of the time-temperature plots in Appendix B. The letter and the number designating, respectively, the vertical and horizontal position, as combined in the curve designations, define completely the location of the couple.

Additional couples, designated on the plots (Appendix B) as H_2 and H_3 , were placed on some of the structural-steel and cast-iron columns at the base and on the edge of the beam bracket near the top (Fig. 31).

In general the letter *i* following the numbers in the column couple designations (Appendix B) is to be interpreted as "*insulated*," referring to the method of attachment. Exceptions occur only in test Nos. 3 and 4 (Fig. 90) in the case of the couples attached on the inside of the column.

Letters indicating points of the compass are, in general, included in the designations of the couples that are not on the side of the column on which the general vertical distribution occurs. In the latter case, the couples are designated by the location letter and number only. Exceptions to the above are designations for the insulated couples in test No. 27 (Fig. 98), No. 47 (Fig. 101), and Nos. 62 and 63 (Fig. 126), which are located near the couple on the same level that has a direction letter in its designation.

The couples whose curves on the plots for the timber column tests (Figs. 140 and 141) carry the designation *T* were located 13 inches below the cap bearing, the lateral positions being evident from the location diagrams, as are also the locations of the couples on the metal of the caps and pintles. The other vertical positions were the same as in the other tests.

In tests Nos. 10 and 11 (Fig. 92) and No. 13 (Fig. 93) the prime mark (') on one couple designation in each test indicates that the given couple had no asbestos and sheet-iron protection over it.

In test No. 21 (Fig. 96) the prime marks on two curves indicate couples whose leads for 19 inches immediately above the junction were protected by strips of asbestos board 1 inch wide and $\frac{1}{4}$ inch thick, the object being to determine what influence possible heat interchange between the leads and the junction would have on the indication of the couple.

(c) CONNECTIONS TO INSTRUMENTS

The wires of the column thermocouples led from the outlet in the column head above the furnace roof to the junction box placed to one side of the latter (Fig. 31), where they were put under binding posts on a set of connecting blocks. Duplex leads of insulated iron and constantan wire were permanently

installed from the junction box to the bus-bar board in the case housing the recording potentiometer, from whence copper leads extended to the dial switch. The column couples could accordingly be connected with either the indicator or the recorder.

4. METHOD OF TAKING AND REDUCING OBSERVATIONS

In most tests the rate of temperature change was not more rapid, but that readings of both furnace and column thermocouples could be taken on the indicating potentiometer. Since it was found that the observations taken with it were more readily reduced to final form than the recorder records, the temperature determinations as given in the tables and plots are in general based on indicator readings, although the recorder was connected with the furnace couples in practically all tests, and its records served as a general check on temperatures during the test, and were also used in making occasional interpolations between indicator readings in plotting the results.

In taking observations with the indicating potentiometer the electromotive forces of the furnace or the column couples were read in succession in a chosen order, then repeated in reverse order, with equal time intervals (10 seconds or less) between consecutive readings. Mean values were taken of the direct and reverse readings on the respective couples, which, on the assumption of linear temperature change, are equivalent to simultaneous readings on all couples. The readings of the copper-constantan couples at the cold junctions of the furnace pyrometers, were reduced to the corresponding platinum-platinum rhodium electromotive forces, and added to the indications of their respective furnace couples. In the case of the iron-constantan column couples the cold junction correction, which was due to temperature differences between the junctions at the bus-bar board and the zero of the calibration plot, were set off directly on the compensator. For the nickel-constantan column couples a further correction was applied due to the electromotive force introduced by the couple formed in connecting the nickel couple wire to the iron wire leading from the junction box above the furnace to the instruments.

After the proper corrections to the observed electromotive forces had been applied the corresponding temperatures were obtained from the calibration plots.

5. CALIBRATION OF THERMOCOUPLES**(a) FURNACE THERMOCOUPLES**

The furnace couples, in addition to being compared several times during the series of tests with a standard couple, were frequently submitted to a homogeneity test. The latter is, in effect, a determination of the electromotive force, at frequent points along the couple wire, against standard wire of the same kind. This test was necessary, since the electromotive force of a nonhomogeneous couple—that is, one whose thermoelectric properties vary from point to point along the wire—depends not only on the temperature of the junction, but also on the magnitude and position of the temperature gradients along the length of the couple, which, in general, were not the same as placed in the column-testing furnace as in the furnace used for calibrations.

Changes in calibration were found to be very small as long as the protecting tubes served their intended purpose of excluding the furnace gases. When a protecting tube was broken, or the couple wires had in any manner been exposed to the furnace gases, a homogeneity test was made. When such test showed no electromotive force greater than the equivalent of 15°C at 1000°C , the couple was used again without further treatment. When this value was exceeded, the wire was either brought sufficiently near to its original (uncontaminated) condition by annealing for several hours at 1500°C by passing an electric current through it, or the bad portion of the wire was removed and good wire substituted.

An accuracy within 1 per cent of the indicated temperatures was attained in the great majority of tests. In the case of individual couples in a few tests the homogeneity determination indicated possibility of errors as high as 5 per cent, although the actual errors incurred were probably considerably smaller, since the nonhomogeneous portion of the couple was not necessarily in the region of maximum temperature gradient.

(b) COLUMN THERMOCOUPLES

Practically all of the wire used in constructing the column couples was bought in two lots, one of which was purchased in 1914, the other in 1917.

A preliminary study of the variations in the thermoelectric properties among a number of samples taken from the wire in

the first lot indicated that to secure the desired accuracy calibration of individual couples made from this wire would be necessary and that differences between the indications of individual couples were approximately proportional to the temperature.

The latter result showed that a good estimate could be made of the calibration of a given couple by comparing it at one temperature with a standard, and this practice was, in general, followed for all the couples made from the wire in the lot first purchased. Tests on about 50 representative couples made from the wire in the second lot showed sufficiently small variations among the individual couples as to permit the use of one temperature-emf curve for all the couples made from this lot.

The results of the large number of tests made in studying the wire used in constructing column couples indicate that the accuracy was within 2 per cent of the indicated temperatures.

VIII. DEFORMATION MEASUREMENTS

1. GENERAL OUTLINE

Included under this head are means for measuring (1) the unit compression and expansion over a definite gage length, (2) the total depression or expansion of the column measured at a point above its heated portion, (3) the lateral center deflection.

FIG. 33.—Diagram showing method used for measuring deformation

The general method used for measuring unit deformation and center deflection is shown in diagram in Fig. 33 and a view of the furnace with attachments in place is given in Fig. 34. Fine nickel-chromium alloy wires were connected to opposite sides of the test column at points symmetrical with its center, using for each a short length of heavier wire and a threaded insert. The

FIG. 34.—Testing furnace with furnace couples, column, and deformer in place

other ends of the wires were weighted and passed over small flanged wheels located in the end posts. The movement of the wires was measured at an intermediate station on each side of the furnace.

(a) ATTACHMENT AND PROTECTION OF WIRES

A detail of the insert used for attaching the wire to the column is given in Fig. 35, as are also details of the water-cooled insulating tube. The hole for the insert was prepared before the column was covered or placed in the furnace and a $1\frac{1}{4}$ -inch pipe with collar was bolted around it to provide a clear space for the wire. The pipe projected outside of the covering to afford support for the insulating tube. With the column in place in the furnace, the insert, with wire attached and held in a suitably formed tool, was threaded into the column flange and turned so that the wire was left supported on the upper ledge of the insert.

The insulating tube was designed to protect the wires from the heat of the furnace, as otherwise they would break under the tension to which they were subjected. The circulating water was contained in the annular space between the 2 and 3 inch wrought pipes, the outer pipe being further protected by fire-clay tubing. A 5-inch wrought pipe, with the annular space between it and the 3-inch pipe filled with finely crushed fire brick, was substituted for the fire-clay tubing in the later tests of the series.

Outside of the furnace the wires were shielded by sheet-metal tubes.

2. UNIT COMPRESSION AND EXPANSION

(a) MICROMETERS AND MOUNTING

The vertical movement of the wires was measured by means of microscopes set in micrometer slides (Fig. 36). The latter were mounted one on each end of nickel-steel bars supported near the middle. The supports were provided with pivots and slides to secure angular, vertical, and horizontal adjustment of the bar as a whole. The micrometer heads are graduated to 0.005 mm and can be read to the nearest 0.001 mm, the total range of the slide being 50 mm.

(b) METHOD OF TAKING AND REDUCING OBSERVATIONS

Readings were taken at the same time on both sides of the furnace, the lower micrometer, upper micrometer, and again, the

FIG. 35.—Detail of insulating tube and insert

FIG. 36.—Apparatus for measuring deformation

lower micrometer being read in the given order at equal intervals. The compression or expansion in the gage length was obtained by multiplying the movement of the upper wire relative to the lower wire at the point of observation by the ratio the distance from the column to the wire support in the end post has to the distance from the latter point to the microscope (Fig. 33). The unit deformation is the average of the values of total deformation obtained as given above, divided by the distance between gage points (94 cm).

3. CENTER DEFLECTION

The lower gage point was located near the center of the heated length of the column, and the center lateral deflection was measured by means of polished nickel scales, readings being taken with reference to points on the wires and their reflection in the scales.

The east-and-west deflection scales were placed perpendicular to the wires, and the deflection of the column was obtained by multiplying the observed movement at the scale by the ratio of distances, as given in Fig. 33, of the column and of the scale, respectively, from the wire supports in the end posts, taking the average of values obtained on the two sides.

The north-and-south deflection scales were placed parallel with the wires, small flat disks being attached to the latter in front of the scales, with reference to which readings were taken. The deflection in the north or south direction is the average of the movements observed at the two posts, assuming the expansion of the measuring wires on both sides of the column to be equal.

4. TOTAL DEPRESSION OR EXPANSION

The total vertical depression or expansion of the column before failure, as indicated by the movement of the head of the loading ram, was determined for the columns in the fire and water test series, the timber columns, and a few subsequent tests of structural-steel and cast-iron columns. For this purpose a single No. 30 wire on one side of the furnace only was attached to the bearing below the head of the ram plunger, the details of attachment and measurement of its movement being the same as described in paragraph 2 of this section.

5. CALIBRATION AND ACCURACY

Calibrations of the instruments and appliances used for measuring unit deformation and study of the test results, indicate possible maximum errors of 20 parts in 100 000 (0.0002), which correspond to errors in instrument readings of about 0.008 inch (0.2 mm). These errors were caused mainly by the change in sag of the wires due to moisture in the insulating tubes and also by movement of the column as a whole in the time interval required by a set of micrometer readings. The errors due to irregularities in micrometer screws and slides and in parallelism of slides and of microscopes as mounted on the supporting bar were relatively small.

In determining the total expansion or depression of the column by measurement of the movement of the loading ram a large error was incurred in applying the load due to deflection of supports. During the subsequent fire exposure under constant load the supports appear to have remained fairly rigid, the maximum errors for this period being probably within 0.01 inch (0.25 mm) for the steel and cast-iron columns and 0.04 inch (1 mm) for the timber columns.

The center deflection in the east-and-west direction was determined with an accuracy of about 0.04 inch (1 mm) and in the north-and-south direction within about 0.08 inch (2 mm). The principal source of error in the latter case was unequal expansion of the wires due to unequal temperatures within the insulating tubes.

IX. METHOD OF TESTING

The columns in the fire-test series were subjected to a constant working load and fire exposure increasing according to a predetermined time-temperature relation until failure occurred or until they had withstood the test eight hours, or more.

In the fire and water tests the working load was maintained constant and the column exposed to fire for a predetermined period when water at given pressures was applied by means of a hose stream.

1. LOADING FORMULAS AND APPLIED LOADS

(a) WORKING LOADS

The formulas used in calculating the working loads to be placed on the columns were among those in most general use at the time the method of procedure for these tests was determined. They are given in Table 41, where are also given the properties of the columns and the calculated and applied working loads.

The applied loads, which include weights of the column head and bearing blocks (2500 pounds), exceed the calculated working loads

TABLE 41.—COMPUTED AND APPLIED WORKING LOADS

SECTION	Formula	Nominal Area, Sq. In.	l/r	Computed Load		Applied Load	
				Lb. per Sq. In.	Total, Lb.	Total, Lb.	Per cent of Computed Load
Rolled H.	$16000-70 \frac{l}{r}$	10.17	75.6	10710	108900	119500	109.7
Plate and Angle.	do	13.00	111.8	8170	106200	116000	109.2
Plate and Channel.	do	8.76	64.7	11470	100500	111000	110.5
Latticed Channel.	do	7.78	44.0	12920	100500	111000	110.5
Z-bar and Plate.	do	9.32	81.7	10280	95900	105000	109.5
I-beam and Channel.	do	10.12	72.1	10950	110800	122000	110.1
Latticed Angle.	do	8.44	40.7	13160	111000	122500	110.4
Starred Angle.	do	13.27	108.5	8435	111900	124000	110.7
Round Cast Iron (Vertically Cast).	$10000-60 \frac{l}{r}$	14.45	63.2	6210	89700	98500	109.8
Round Cast Iron (Horizontally Cast).	do	14.73	68.2	5910	87100	95500	109.6
Round Cast Iron (Concrete filled).	do	14.73	68.2	5910	87100	95500	109.6
			$\frac{l}{d}$				
Steel Pipe (Concrete filled).	As (13500-140 l/d) +Ac (1000-11 l/d).	Steel 6.93 Concrete 38.74	19.6	Steel 10750 Concrete 784	104900	114500	109.2
Reinforced Steel Pipe (Starred angles embedded in the concrete filling).	do	Steel 18.36 Concrete 40.07	17.7	Steel 11060 Concrete 805	234700	236000	100.6
Square Vertically Reinforced Concrete.	$450 (Ac+15 As)$	Concrete 140 Steel 4.00	12.7	Concrete 450 Steel 6750	90000	101000	112.2
Round Vertically Reinforced Concrete.	$450 (Ac+15 As)$	Concrete 127 Steel 6.00	11.7	Concrete 450 Steel 6750	97500	107500	110.3
Hooped Reinforced Concrete.	$650 (Ac+15 As)$	Concrete 129 Steel 3.38	11.7	Concrete 650 Steel 9750	117000	129000	110.3
Timber.	$1000 (1 - \frac{1}{80d})$	129.4	13.4	833	107700	118500	110.0

l—Effective length, inches.
r—Least radius of gyration, inches.
d—Diameter or side, inches.

As—Area of vertical steel, sq. inches.
Ac—Area of concrete, sq. inches.

by about 10 per cent for most of the columns. The principal part of the excess was incurred in the first tests of the series which were made before the calibration of the loading ram was completed. It was decided to maintain the same loads for the subsequent tests as representative of a moderate condition of overload.

No allowance was made for the load-carrying capacity of the covering or filling in any of the tests except in case of the pipe columns where the loads were calculated according to the loading formula in use by the manufacturer who supplied the columns.

(b) **LOADING TO FAILURE**

In case the column withstood the 8-hour fire test it was immediately loaded to failure under full fire exposure.

In the fire and water test series, three protected structural steel, the two unprotected cast-iron, and the three reinforced-concrete columns were loaded to failure after they had cooled. Four protected structural-steel columns after subsection to fire and water application, were loaded to a little over twice the applied working load and reserved for use in further tests.

2. FIRE EXPOSURE

(a) **CHARACTER OF THE FIRE**

The fuel used was carbureted water gas from the city service mains. It was admitted at the primary burners with insufficient air for immediate complete combustion, in order to avoid excessive temperatures at the bottom of the furnace and allow further combustion to take place at the secondary air inlets. The gas was generally burned with sufficient air to prevent deposit of soot on the test column or furnace walls.

(b) **PRELIMINARY PANEL TESTS**

During the first half hour it is not in general possible to regulate the temperature of a large testing furnace to correspond closely with predetermined temperatures; also, the determination of temperature is complicated by the lag of the furnace pyrometers with the rapid temperature rise occurring during this period. To obtain information on the effect of varying the rate of temperature rise, two panels built up of the five kinds of hollow clay tile used as protection in the column tests, were subjected to fire on one side for one-half hour and allowed to cool in place. The

Panel No. 1

Panel No. 2

Black lines trace fine cracks developed in fire test

FIG. 38.—*Preliminary tile panels after test*

average furnace temperatures obtaining in the two tests are given in Fig. 37, and in Fig. 38 the condition of the panels after test is shown. Most of the tile developed fine cracks which extended to the inside of the inner shell and in a few cases through the cross webs to the outer shell. Cracks along the cross webs near the inner shells were common and some inner shells were loose. While in one test, the indicated furnace temperature rise in the first 10 minutes was over twice as rapid as in the other, the condition of the two panels after test was nearly the same.

(c) TIME-TEMPERATURE CURVE

The temperature of the furnace was regulated to conform as nearly as practicable with a predetermined time-temperature reference curve shown in broken lines in Fig. 39. Here is also shown the furnace temperature attained, the curve being obtained by

FIG. 37.—Furnace temperatures, preliminary panel tests

FIG. 39.—Comparison of furnace temperatures used in fire tests

averaging the results from the full number of tests. Three curves giving the average indicated furnace temperatures attained in previous fire tests are also given.²

On the reference curve of the column tests a furnace temperature of 832° C (1550° F) obtains at the end of 30 minutes, and 927° C (1700° F) at the end of the first hour. From this point the temperature increases at a gradually varying rate up to 2 hours, when the temperature is 1010° C (1850° F). Subsequent to this the rate of rise is constant, 1101° C (2000° F) being attained at the end of 4 hours, and 1260° C (2300° F) at the end of 8 hours. The average indicated furnace temperature was somewhat below the reference curve in the first hour and above it by a maximum of 25° C during the subsequent periods.

(d) INFLUENCE OF PYROMETER MOUNTING ON INDICATED TEMPERATURES

With the type of mounting it was deemed necessary to use in the column tests (Fig. 32) the pyrometers did not indicate the full measure of the temperature effects within the furnace chamber, due mainly to the heat insulating and radiating properties of the mounting.

(1) TEMPERATURE LAG.—The retardation due to the heat insulation given the thermocouple by the mounting was particularly marked during the first part of the test period, when, with the obtaining rapid temperature rise, the indicated temperatures were considerably lower than those acting on the mounting on account of the time required to produce temperature equilibrium throughout the latter. This effect will be termed "lag," and a measure of its extent can be obtained by application of the principles governing heat interchange.

When the rate of temperature rise is not too rapid and the heat interchange is chiefly due to conduction, within a given range, the temperature lag is very nearly proportional to the rate of rise of indicated temperature, or,

$$U - \theta = \lambda \frac{d\theta}{dt} \quad (1)$$

where θ is the temperature indicated by the pyrometer, U is the

² cf. Woolson and Miller, Paper No. 272, International Association for Testing Materials, 1915; floor test Nos. 55 to 74. Humphrey, U. S. G. S. Bulletin No. 370; panel test Nos. 1 to 30. British Fire Prevention Committee, Journal No. 6, 1911; average from eight full-protection, Class B, floor tests.

FIG. 40.—Lag correction for furnace pyrometers

temperature the pyrometer would indicate if the furnace conditions at the given instant were maintained constant a sufficiently long time, $\frac{d\theta}{dt}$ is the rate of rise of indicated temperature, and λ is a constant for the pyrometer considered within a given range in U . To determine λ , the pyrometer was plunged into a gas-fired furnace whose temperature was maintained as nearly constant as possible and readings taken until the indicated temperatures showed little or no change. The experiment was begun as soon after the gas was lighted as was consistent with maintaining steady temperature in order that the furnace conditions might approximate those present during the first part of a column test. If U , for the conditions of the experiment, be assumed constant, from equation (1)

$$\lambda = \frac{t}{\log_e (U - \theta)} + C, \quad (2)$$

where t is the time after the pyrometer was introduced into the furnace and C is the constant of integration, which latter, in the method applied, is eliminated by subtraction.

The results of two determinations are given in Fig. 40 (a), λ being obtained from the slope of the line connecting the experimental points, multiplied by 0.4343, the modulus of the common system of logarithms. As applied for correcting the average curve of indicated temperature of the column tests, a lower value than the average experimental result was used to allow for impaired and thinner insulation that was present in a number of tests, a value of λ of 2.4 minutes being used up to indicated temperatures of about 800° C. Determinations made at higher temperatures gave lower results, values of about 2 minutes obtaining for mountings with unimpaired insulation tested at 950 to 1000° C.

In Fig. 40 (b) the lower curve is the average curve of indicated furnace temperature of the column tests for the first 20-minute period. Corrections for lag were deduced for the indicated temperatures given by the curve in the interval 5 to 20 minutes by substituting the given value of λ in equation (1), the rate of rise, $\frac{d\theta}{dt}$, being obtained from tangents drawn to the average curve at points $\frac{1}{2}$ to 1 minute apart. As given by the difference between the original and corrected curves, the lag decreases from a little over 100° C at 5 minutes to 14° C at 20 minutes. From there on

FIG. 41.—Effect of radiation on indication of furnace pyrometers

the decrease is gradual up to the uniform slope after two hours (Fig. 39), where the lag correction is about 1°C . For the furnace temperatures of the individual tests the corrections would be much more irregular than those on the average curve on account of irregularities of slope, although, in general, after the first hour they would be relatively small and within the limit of error applicable to furnace-temperature measurements. The furnace temperatures as given on the plots in Figs. 90 to 145 are indicated temperatures, as it was not deemed practicable to apply corrections for either lag or radiation effects to results of individual tests.

(2) RADIATION EFFECTS.—In a gas-fired furnace where the source of heat is combustion within the chamber the inside of the walls of the latter will be at lower temperature than the furnace contents. The indications of a pyrometer introduced into the chamber to measure the temperature of its contents will be influenced by radiant interchange of heat with the inclosure, the extent of the effect being dependent mainly upon the size of the pyrometer and the temperature difference between the inclosure and the gaseous contents. To obtain a measure of the extent to which the indications of the furnace pyrometers were thus affected, an unprotected thermocouple of fine platinum—platinum-rhodium wires 0.004 inch ($1/10$ mm) in diameter was supported $1\frac{1}{2}$ inches outside of the closed end of a porcelain tube on similar wires of larger diameter that were passed through and sealed into the end of the tube, through which they led to the outside of the furnace. Two pyrometers of the same design as those used in the column tests (Fig. 32) were placed about 3 inches away from the unprotected couple and several runs made in a furnace having walls of the same material and of about the same thickness as those of the column furnace. Readings of the unprotected and the protected couples were taken at intervals of one-half minute or less. The indicated temperatures obtained in a run extending to 8 hours are plotted in Fig. 41. Those given by the unprotected couple were considerably higher than those of the protected couples, because the temperature of the fine wire was influenced to a much smaller extent by radiant heat interchange with the inside of the furnace inclosure than that of the protected couples, and the temperatures indicated by it approximate those of the furnace contents as closely as can be attained without excessive refinements.

The difference in indication due to radiation, as given by the lower curve in Fig. 41, was obtained by correcting the average indication of the protected couples for lag and subtracting the corrected value from that of the unprotected couple. While the variations in the resulting difference of indication are partly due to local and general variations in furnace conditions and minor changes in calibration of the unprotected couple, the general decrease from about 150° C during the first half hour to less than 50° C at the end of 8 hours can be ascribed mainly to decrease in temperature difference between the furnace contents and the inside surface of the furnace chamber.

The average difference due to radiation was applied to the results plotted in Fig. 40 (b), and the estimated indication of the unprotected fine-wire couple is given by the upper curve.

Since the indication of all pyrometers is influenced by lag and radiation effects, the comparison of furnace exposures afforded by the indicated temperatures given in Fig. 39 is limited by differences in these particulars incident with the types of pyrometers used.

3. FIRE AND WATER TEST PROCEDURE

The duration of the fire periods varied from 22½ minutes to one hour, and that of the subsequent water application from one to five minutes. The length of the maximum fire period was determined by the time within which water is generally applied in building fires, which was estimated as being one hour. The unprotected columns and some of the protected columns were given fire periods of shorter duration which were well within the time to failure of the corresponding columns in the fire-test series. The duration and pressure of the water application were also varied for the different types of protection, the heavier ones being subjected to the most severe test conditions.

In applying the hose stream, the nozzle was moved back and forth on one side of the furnace and maintained at a constant distance from the column, the water being applied in succession over the full height on three of its sides.

4. OBSERVATIONS DURING TEST

Readings for temperature of furnace and of test column were taken at intervals of 2 to 15 minutes, the frequency of the readings

depending on the rate of temperature change. Readings for deformation were also taken at intervals of 2 to 15 minutes.

Notes were taken on the character of the fire and on its effects on the test column at all stages of the test where the latter could be observed.

5. OBSERVATIONS AFTER FAILURE

After the column had cooled notes were taken of its general condition and the covering was partly or wholly removed to determine the extent to which it was damaged. Measurements were taken of the amount and direction of the final buckle.

6. PHOTOGRAPHIC RECORDS

Photographs were taken of all columns after test and also of a number of typical columns before test. The views were generally taken diagonally and from opposite sides, so as to show all faces of the column, only one view being as a rule included in this paper.

X. RESULTS OF FIRE TESTS

1. FIRE-TEST RESULTS IN TABLES AND FIGURES

The results of the fire tests in points of time to failure, load sustained, and relative fire exposure are given in Tables 42*a* to 42*i*. The columns are grouped by classes of protection, and the thickness and material applied in each test are indicated.

In Table 43 (pp. 122-123) are given the period of expansion, the time to failure, and the maximum temperatures attained in the metal.

The period of expansion of a column when loaded and exposed to fire is the period from the beginning of the test to the time when expansion of the column ceases, due to yielding of the heated metal under the applied load.

The time to failure in the fire test extends from the beginning of the test to the time when the column is unable to sustain the applied working load.

The time to failure of all columns in the fire-test series is given in diagram form in Fig. 42 (p. 114), and the period of expansion of the steel, cast-iron, and concrete columns is given in similar manner in Fig. 45 (p. 121).

2. PHOTOGRAPHIC RECORDS

Views before and after test of the columns in the fire-test series are given in Figs. 58 to 82, Appendix A (pp. 216-241).

TABLE 42a.—RESULTS OF FIRE TESTS

Unprotected columns

Test No.	SECTION	Nominal Area, Sq. in.	l/r	Load Sustained During Test		Time to Failure, Hr.—Min.	Furnace Exposure, Percent
				Total Load, Lb.	Unit Load, Lb. per Sq. in.		
1	Rolled H.....	10.17	75.6	119500	11750	0—11¼	95.6
2	Plate and Angle.....	13.00	111.8	116000	8900	0—19¼	97.0
3	Plate and Channel.....	8.76	64.7	111000	12650	0—14	101.4
4	Latticed Channel.....	7.78	44.0	111000	14250	0—11	101.1
5	Z-bar and Plate.....	9.32	81.7	105000	11250	0—14¼	100.0
6	I-beam and Channel.....	10.12	72.1	122000	12050	0—17	89.6
7	Latticed Angle.....	8.44	40.7	122500	14500	0—14	96.3
8	Starred Angle.....	13.27	108.5	124000	9350	0—21½	99.0
9	Round Cast Iron.....	14.73	68.2	95500	6500	0—34¼	101.2
10	Round Cast Iron.....	14.73	68.2	95500	6500	0—34½	101.7
10A	Round Cast Iron.....	14.45	63.2	98500	6800	0—34¼	100.3
11	Round Cast Iron (Concrete filled).....	14.73	68.2	95500		0—45¼	103.1
12	Steel Pipe (Concrete filled).....	Steel 6.93 Concrete 38.74	63.9	114500		0—36	101.9
3	Reinforced Steel Pipe (Starred angles imbedded in concrete filling).....	Steel 18.36 Concrete 40.07	68.2	236000		1—11¼	99.3

NOTE: Details of the steel, cast-iron, and pipe columns listed in Tables 42a to 42g are given in Figs. 1 to 4 (p. 16-19), and Figs. 6 and 7 (p. 22 and 24). Further properties of the sections are given in Tables 3a to 3h (p. 33-47).

TABLE 42b.—RESULTS OF FIRE TESTS

Columns partly protected by concrete

Test No.	SECTION	*Protection		Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
		Mixture	Kind of Concrete				
14	Rolled H.....	1:2:4	Joliet gravel.....	405	119500	1 — 04¼	85.1
15	Rolled H.....	1:2:4	Rockport granite....	407	119500	0 — 48¾	91.5
16	Plate and Angle...	1:2:4	New York trap.....	416	116000	0 — 44¾	94.9
17	Plate and Angle...	1:1½:4½	Hard coal cinders...	408	116000	0 — 41¾	99.6
18	Latticed Channel..	1:2:4	New York trap.....	418	111000	2 — 53	98.1
19	Z-bar and Plate....	1:3:5	Chicago limestone...	414	105000	1 — 07¼	95.8
20	I-beam and Channel.....	1:3:5	New York trap.....	415	122000	1 — 24½	100.2
21	I-beam and Channel.....	1:3:5	New York trap.....	416	122000	1 — 21¾	99.6
22	Latticed Angle....	1:2:4	Chicago limestone...	408	119500	5 — 14	99.2

*Re-entrant portions and interior filled with concrete.

TABLE 42c.—RESULTS OF FIRE TESTS

Columns protected by plaster on metal lath

Test No.	SECTION	Protection	Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
23	Plate and Angle...	Two 2-coat layers of Portland cement plaster on expanded metal lath, each layer 1 in. thick, with a ¼-in. air space between layers	508	*117500	2 — 52	103.1
24	Plate and Channel...	Two 2-coat layers of Portland cement plaster on woven wire lath, each layer ¾ in. thick with a ¼-in. air space between layers	496	111000	2 — 24	101.5
25	Z-bar and Plate...	One 2-coat layer of Portland cement plaster, 1 in. thick, on expanded metal lath	484	105000	1 — 07¾	103.7
26	Latticed Angle....	One 2-coat layer of Portland cement plaster, 1½ in. thick, on expanded metal lath	497	122500	1 — 23½	104.2
27	Round Cast Iron...	One 2-coat layer of Portland cement plaster, 1½ in. thick on high ribbed expanded metal lath with a ½-in. broken air space	498	95500	2 — 53	98.2

* Heavier load used as plate has 1/32 in. greater thickness than nominal.

TABLE 42d.—RESULTS OF FIRE TESTS

Columns protected by concrete

Test No.	SECTION	Protection			Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
		Thick-ness, In.	Mix-ture	Kind of Concrete				
28	Rolled H.....	2	1:2:4	Chicago limestone	437	119500	6 — 33 $\frac{3}{4}$	96.9
28A	Rolled H.....	2	1:2:4	Chicago limestone	438	119500	7 — 09 $\frac{1}{4}$	95.2
29	Rolled H.....	2	1:2:4	New York trap	435	119500	4 — 38 $\frac{1}{2}$	99.5
30	Rolled H.....	2	1:2:4	Joliet gravel	439	119500	7 — 16	99.2
31	Rolled H.....	2	1:2:4	Cleveland sandstone	500	119500	4 — 11 $\frac{1}{2}$	99.4
32	Rolled H.....	2	1:2:5	Hard coal cinders	503	119500	3 — 44	101.3
32A	Z-bar and Plate	2	1:2:5	Hard coal cinders	497	105000	4 — 02	100.5
33	Rolled H.....	4	1:2:4	Chicago limestone	450	119500 ‡431000	8 — 08	99.2
33A	Rolled H.....	4	1:2:4	Chicago limestone	455	119500 ‡405000	8 — 07 $\frac{1}{4}$	98.8
34	Rolled H.....	4	1:2:4	Rockport granite	452	119500	7 — 58	98.6
34A	Rolled H.....	4	1:2:4	Rockport granite	454	119500	7 — 23	102.5
35	Rolled H.....	4	1:3:5	Chicago limestone	504	119500 ‡348000	8 — 07	101.3
36	Plate and Angle....	2	1:2:4	New York trap	445	116000	3 — 53 $\frac{1}{4}$	100.3
37	Plate and Angle....	4 round	1:2:4	New York trap	503	116000	7 — 34 $\frac{1}{2}$	99.9
38	Plate and Channel..	2	1:2:4	Joliet gravel	449	*116500	5 — 28 $\frac{3}{4}$	100.2
39	Plate and Channel..	4	1:2:4	Meramec R. gravel	436	*116500	3 — 41 $\frac{1}{4}$	98.3
40	Latticed Channel ..	2 round	1:2:4	New York trap	501	111000	7 — 57	99.1
41	Z-bar and Plate....	4	1:3:5	Chicago limestone	451	105000 ‡332000	8 — 24 $\frac{1}{2}$	99.6
42	Z-bar and Plate....	4	1:3:5	Chicago limestone	453	105000 ‡333000	8 — 11 $\frac{1}{2}$	98.5
43	I-beam and Channel	2	1:2:4	Cleveland sandstone	456	122000	4 — 11	98.7
44	I-beam and Channel	2	1:3:5	Cleveland sandstone	458	122000	3 — 04 $\frac{1}{4}$	100.6
45	Starred Angle.....	2 round	1:2:4	Meramec R. gravel	447	124000	1 — 47	99.7
46	Latticed Angle.....	‡2	1:2:4	New York trap	451	122500	6 — 43 $\frac{1}{2}$	99.0
47	Round Cast Iron...	2	1:2:5	Hard coal cinders	446	95500	2 — 48 $\frac{3}{4}$	99.6

* Heavier load used as plates have 1/32 in. greater thickness than nominal.

† 2-in. outside rivets, 3 $\frac{1}{4}$ -in. outside angles.

‡ Load necessary to cause failure of column. After 8 hr. the load was increased until failure occurred.

Fire Tests of Building Columns

III

TABLE 42e.—RESULTS OF FIRE TESTS

Columns protected by hollow clay tile

Test No.	SECTION	Protection		Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure Percent
		Thick-ness of Tile, In.	Kind of Tile, Filling, and Method of Tying				
48	Rolled H.....	2	New Jersey semi-fire clay..... No filling Outside wire ties	496	119500	1 — 50	100.9
49	Rolled H.....	4	Same as No. 48.....	497	119500	1 — 40	99.9
50	Plate and Angle...	2	Surface clay, Boston.. Granite concrete fill No ties	494	116000	1 — 05¼	99.8
50A	Plate and Angle...	2	Same as No. 50.....	505	*117500	1 — 59½	101.8
51	Plate and Angle...	4	Surface clay, Boston Granite concrete fill Outside wire ties	487	116000	2 — 17¼	101.8
51A	Plate and Angle...	4	Same as No. 51.....	507	116000	2 — 55½	100.5
52	Plate and Channel..	2	Ohio shale..... Cinder concrete fill Outside wire ties	513	111000	1 — 40¼	100.2
53	Plate and Channel..	4	Same as No. 52.....	495	111000	1 — 22¼	103.0
54	Latticed Channel..	2	Ohio semi-fire clay.... Trap concrete fill Outside wire ties	489	111000	3 — 17¼	101.1
55	Z-bar and Plate....	2	Ohio semi-fire clay.... Limestone concrete fill Outside wire ties	485	105000	3 — 46¼	99.5
56	Z-bar and Plate....	4	Ohio semi-fire clay.... Limestone concrete fill Wire mesh in joints	491	105000	3 — 33½	100.3
57	I-beam and Channel.....	4	Surface clay, Chicago.. Limestone concrete fill Outside wire ties	491	122000	3 — 23	100.0
58	I-beam and Channel.....	Two 2	Surface clay, Chicago.. Hollow tile fill Wire mesh in joints	502	122000	4 — 35¼	101.7
59	I-beam and Channel.....	Two 2	Surface clay, Chicago.. Hollow tile fill Outside wire ties	490	122000	1 — 33¼	101.2
60	Latticed Angle....	2	Ohio semi-fire clay.... Trap concrete fill, placed before tile was set Outside wire ties	487	122500	3 — 09½	100.4
61	Latticed Angle....	2	Ohio semi-fire clay.... No filling Outside wire ties	501	122500	0 — 50¼	100.3
62	Round Cast Iron...	2 round	Porous semi-fire clay, New Jersey..... No filling Outside wire ties	483	95500	4 — 11½	101.3
63	Round Cast Iron...	2 round	Same as No. 62.....	493	95500	2 — 57½	100.6
76	Rolled H.....	2	Ohio shale; Ohio semi-fire clay; semi-fire clay, New Jersey.... Limestone concrete fill Wire mesh in joints Tile covered with ¾-in. layer of gypsum plaster	42	119500	4 — 25½	98.1
77	Plate and Angle	4	Semi-fire clay, N. J.; surface clay, Chicago; surface clay, Boston.. Limestone concrete fill Wire mesh in joints Tile covered with ¾-in. layer of lime plaster	45	116000	4 — 42¼	97.3

* Heavier load used as plate has 1/32 in. greater thickness than nominal.

TABLE 42f.—RESULTS OF FIRE TESTS

Columns protected by gypsum block

Test No.	SECTION	Thickness of Block, In.	Protection	Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
			Kind of Block, Filling and Method of Tying				
64	Rolled H..	4	Western gypsum (solid) Hollow gypsum block fill Wall ties in joints	502	119500	4 — 43¼	104.5
65	Plate and Channel	2	Western gypsum (solid) Solid gypsum block fill Wall ties in joints	505	111000	2 — 21½	104.5
66	Latticed Channel	2	Eastern gypsum (solid) Poured gypsum fill Wire mesh in joints	495	111000	2 — 36	101.0
67	Rolled H..	4	Same as No. 66	492	119500	5 — 31½	101.2
67A	Rolled H..	4	Same as No. 66	491	119500	6 — 24½	99.7

TABLE 42g.—RESULTS OF FIRE TESTS

Columns protected by brick

Test No.	SECTION	Thickness of Brick, In.	Kind of Brick and Filling	Age of Covering, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
68	Rolled H..	2¼	Chicago common brick set on edge and end Brick fill	498	119500	1 — 40¼	104.0
69	Rolled H..	3¼	Chicago common brick laid flat Brick fill	502	119500	7 — 13¼	101.2

TABLE 42h.—RESULTS OF FIRE TESTS

Reinforced concrete columns

Effective length, 12 ft., 8 in.

Test No.	SECTION	Outside Dimensions and Reinforcement	Concrete		Age, Days	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
			Mixture	Kind.				
70	Square Vertically Reinforced	16-in. square..... Four 1-in. sq. bars	1:2:4	Chicago limestone	433	101000 ‡294000	8 — 40½	101.1
71	Square Vertically Reinforced	Same as No. 70....	1:2:4	New York trap	450	101000	7 — 22¾	99.2
72	Round Vertically Reinforced	17-in. diameter.... Six 1-in. sq. bars	1:2:4	Chicago limestone	520	107500 ‡250000	8 — 04½	102.1
73	Round Vertically Reinforced	Same as No. 72....	1:2:4	New York trap	442	107500	7 — 57½	98.5
74	Hoopd Reinforced	17-in. diameter.... Six ¾-in. sq. bars ¼-in. ϕ spiral on 1½-in pitch	1:2:4	Chicago limestone	522	129000 ‡243000	8 — 06½	99.5
75	Hoopd Reinforced	Same as No. 74....	1:2:4	New York trap	460	129000 ‡163000	8 — 01¾	100.7

‡Load necessary to cause failure of column. After 8 hr. the load was increased until failure occurred.

TABLE 42i.—RESULTS OF FIRE TESTS

Timber columns

Nominal net section, 11⅝ by 11⅝ in. Effective length, 12 ft., 8½ in.

Test No.	Species	Bearing Details	Protection of Column and Cap	Load Sustained During Test, Lb.	Time to Failure, Hr.—Min.	Furnace Exposure, Percent
78	Longleaf pine.	Cast iron cap and pintle	One 2-coat layer of Portland cement plaster 1 in. thick on woven wire lath, ¾-in. air space. Age, 30 days	118500	2 — 15¾	97.6
79	Longleaf pine.	Cast iron cap and pintle	Unprotected.....	118500	0 — 50	105.2
80	Longleaf pine.	Steel plate cap and timber strut	One thickness of ¾-in. gypsum wall board with metal corner beads nailed to column	118500	1 — 13	103.5
81	Longleaf pine.	Steel plate cap and timber strut	Unprotected.....	118500	0 — 35	113.0
82	Douglas fir....	Cast iron cap and pintle	Unprotected.....	118500	0 — 45¼	106.6
83	Douglas fir....	Steel plate cap and timber strut	Unprotected.....	118500	0 — 38½	106.1

NOTE: Details of design of reinforced-concrete and timber columns are given in Figs. 8 to 10 (p. 26-29), and further details of protection are given in Tables 3f and 3j (p. 48-49).

Since a large portion of the effects shown on the photographs of columns after test is due to the deformation and deflection taking place at failure, they should not be considered as representing the condition of the column or its covering immediately before this point was reached. The general condition of the test columns near the end of the fire period is indicated in their respective test logs.

3. FURNACE TEMPERATURES

The temperatures of the furnace in the fire tests, as indicated by thermocouples located on two levels at symmetrical points within the chamber, are given by the upper curves in Figs. 90 to 141, Appendix B (pp. 249-302).

(a) VARIATIONS FROM AVERAGE CURVE

To obtain a measure of the variation in furnace exposure between the different tests, the area under the average furnace curve given in Fig. 39, was calculated up to the end of successive intervals and compared with the area under the average furnace curve for each test up to a point near failure. The comparisons are given in Tables 42*a* to 42*i* as percentages of the area under the average curve of all tests.

4. COLUMN TEMPERATURES

The temperatures attained in the test columns and their coverings are given by the lower curves in Figs. 90 to 141, Appendix B. The arrows on the plots indicate the time of failure for each test.

(a) TEMPERATURE VARIATION OVER LENGTH OF COLUMN

The highest temperature generally obtained over the middle half of the column, the loss of heat at the ends due to conduction tending to maintain a lower temperature in the adjacent portions of the column. The difference was seldom very large, and in some tests, due to local variations in furnace temperature or in column protection, the highest temperature was indicated by one of the end couples.

(b) TEMPERATURE VARIATION OVER CROSS SECTION

The temperature variation laterally across the section was quite marked in all protected-column tests, the highest temperature in the metal obtaining almost invariably at the outer edges. In Fig. 43 this variation is given for some typical columns near failure.

FIG. 43.—Temperature variation over cross-section of typical columns

The temperature indicated by the couple in the covering varied with its distance from the surface and the character of the covering material.

(c) DEHYDRATION POINTS

In the tests with concrete or gypsum protections and reinforced-concrete columns, temperatures in the column of about 100° C obtained over a considerable length of time, due to evaporation of water in the concrete and gypsum. To less extent this effect was present in tests with plaster on metal-lath and concrete-filled hollow clay-tile protections.

5. LONGITUDINAL DEFORMATION AND AVERAGE TEMPERATURE

The unit longitudinal deformation measured over a 37-inch gage length located immediately above the midheight of the column (Fig. 33), together with the computed average effective temperature over the same length, are plotted for a number of fire tests in Figs. 146 to 171, Appendix C (pp. 307-333).

(a) COMPUTATION OF AVERAGE EFFECTIVE TEMPERATURE

The relative location of gage points and thermocouple points and the temperature variation between the latter assumed in calculating the average effective temperature in the gage length, are shown in Fig. 44 for the case of the rolled H section. It can

FIG. 44.—Assumed temperature variation between thermocouple points

be readily shown with reference to the diagram at (a) that the average temperature in the gage length of the edge couples, N , M , and T , located in the (3) position, is given by

$$0.115 T + 0.143 N + 0.742 M = \text{Average (3)},$$

where T , N , and M are the temperatures indicated by the respective thermocouples at a given time. Similarly with reference to the diagram at (b) it can be shown that the average temperature in the cross section at a given level, say at the M position, is expressed by,

$$0.17 (1) + 0.38 (2) + 0.45 (3) = \text{Average } M,$$

(1), (2), and (3) being the temperatures indicated by thermocouples in the respective positions in the cross section. The average effective temperature in the gage length (Av. $G L$) can then be obtained from the relation,

$$\text{Average } G L = \left(\frac{\text{Average } 3}{M \ 3} \right) \text{Average } M.$$

For the other structural-steel sections the average temperature at the *M* or *N* level, was obtained by taking the sum of parts of temperature readings of couples in the (1), (2), and (3) positions as given below, the derivation being similar to that for the rolled H section:

Plate and angle 0.33 (1) + 0.38 (2) + 0.29 (3)

Plate and channel 0.20 (1) + 0.27 (2) + 0.53 (3)

Latticed channel 0.27 (1) + 0.365 (2) + 0.365 (3)

Z bar and plate 0.441 (1) + 0.232 (2) + 0.327 (3)

I beam and channel 0.223 (1) + 0.294 (2) + 0.483 (3)

Latticed angle, test No. 26 0.484 (1) + 0.35 (2) + 0.166 (3)

Latticed angle, test No. 60 0.513 (2) + 0.487 (3)

Starred angle, 0.416 (1) + 0.584 (2)

The average temperature in the vertical plane was obtained from the same expression as that given for the rolled H section in all cases except that for the plate and channel section, due to a slight difference in the relative positions of couples and gage points, the average temperature was given by—

$$0.122 \ T + 0.742 \ M + 0.136 \ N.$$

In the case of the cast-iron and pipe columns the average temperatures plotted pertain to the outer surface of the metal. Readings obtained on the inside of a number of the unfilled cast-iron columns indicate temperatures generally lower by 5 to 10° C. For the reinforced-concrete columns, Figs. 169 to 171, the temperatures plotted are those of the vertical reinforcing bars.

The maximum error involved in computing average temperatures by the above method is estimated to be within 20° C for tests where the covering material remained in place until near the time of failure. Where parts of the covering fell off, as for tests plotted in Figs. 163, 165, and 166, the resulting local heating and irregular temperature distribution introduced much larger errors, as evidenced by the discrepancy between the computed temperature and the corresponding unit deformation. For such tests the limit of error is probably as high as 50° C.

(b) DEFORMATION UNDER HEAT AND LOAD

On application of working load to the column before starting the fire test, a compressive deformation resulted, which in a few tests increased slightly during the first part of the fire period, caused either by shrinkage stresses set up by dehydration of the covering or by increased load on the metal portion of the column resulting from cracking of the covering. As the temperature increased the steel, cast-iron, and reinforced-concrete columns expanded up to a point where the yielding due to the load became equal to or greater than the thermal expansion. (See curves, pp. 308-333).

The unit expansion per degree C during this period varied generally between 0.000010 and 0.000014 for the steel and cast-iron columns, the average of the calculated values for each of these two types of columns being very nearly 0.0000125. These were taken from room temperature up to a point where the rate of expansion began to decrease rapidly due to yielding of the metal. The total observed unit expansion up to the point where expansion ceased varied in the tests of steel columns from 0.0044 to 0.0066, the average being 0.0054. For the cast-iron columns the variation was from 0.0060 to 0.0071, with an average of 0.0064. The lower values are due mainly to local heating which caused the metal to yield over a relatively short portion of the gage length before full expansion had been attained elsewhere. The columns under the lower unit loads generally attained higher expansion than those more heavily loaded.

In the tests of reinforced-concrete columns unit expansions of 0.0000095 to 0.0000102 per degree C obtained, with values of maximum unit expansions of 0.0023 to 0.0046. The point of maximum expansion was less sharply defined than for the steel and cast-iron columns, and the expansion attained was lower.

The compressive deformation taking place subsequent to maximum expansion varied in rate and duration with the type of column and the load-carrying capacity of the covering material. In some tests the total compression before failure more than equaled the previous expansion.

In the tests of timber columns slight expansions were noted during the first few minutes of the fire period, subsequent to which the movement was one of progressive depression, the principal deformation occurring at the bearings on the steel or cast-iron cap introduced near the top of the column.

(c) PERIOD OF EXPANSION

The period of expansion and time to failure of all columns in the fire-test series are given in Table 43 (pp. 122-123). A comparison is given between the length of the portions of the test period preceding and following the point of maximum expansion, as also the maximum temperatures obtaining at the latter point and at failure.

The period of expansion of the steel, cast-iron, and concrete columns is shown in chart form in Fig. 45.

(d) MAXIMUM COLUMN TEMPERATURES

These are given in Table 43 for the point of maximum expansion and the time of failure, the temperatures being obtained from the time-temperature curves extended where necessary to the failure point in the given test.

The edge temperature for the structural-steel columns is the maximum temperature indicated by the couple located nearest to the edge of the section on any of the four regular thermocouple levels. For the cast-iron columns the temperatures given pertain to the outer surface of the metal and for the reinforced-concrete columns to the vertical reinforcing bars. The edge temperatures given for the timber columns were measured on the metal cap at the edge of the column bearing.

The maximum average temperatures for the sections of the columns having the highest edge temperatures were computed by the method given in paragraph 5*a* above, the limits of error involved being somewhat higher than for the plotted results, as each determination is based on fewer couple readings. In the case of the unprotected and partly protected columns, gypsum-block protections, and a few of the concrete and hollow clay-tile protections the average temperatures are not given, since the rapid temperature rise and irregular distribution obtaining near the end of these tests made the computed results unreliable.

At the point of maximum expansion the average over the sections having the maximum temperature varies for tests of structural steel from 484 to 593° C (903 to 1099° F), with an average of 530° C (986° F). Similarly at the time of failure in the fire tests the computed values vary for the given structural-steel columns from 570 to 837° C (1058 to 1601° F), the average being 668° C (1234° F). The high temperatures obtaining at failure in a number of the tests indicate that at this point the covering carried

FIG. 45.—Expansion period of steel, cast iron, and concrete columns in fire-test series

TABLE 43.—TIME TO FAILURE, PERIOD OF EXPANSION AND MAXIMUM COLUMN TEMPERATURES

Test No.	Period of Expansion Hr.—Min.	Time to Failure, Hr.—Min.	Difference, Per cent of Expansion Period	Maximum Observed Temperature in Metal, Deg. C.					
				At End of Expansion Period			At Failure		
				Couple Level	Edge	Average	Couple Level	Edge	Average
1	0—08	0—11½	40.6	B	530	-----	B	624	-----
2	0—17	0—19½	13.2	N	633	-----	N	668	-----
3	0—11	0—14	27.3	M	557	-----	M	626	-----
4	0—09	0—11	22.2	M	510	-----	M	578	-----
5	0—11	0—14½	29.5	M	620	-----	M	670	-----
6	0—13	0—17	30.8	B	623	-----	B	660	-----
7	0—11	0—14	27.3	M	547	-----	M	622	-----
8	0—19	0—21½	13.1	B	575	-----	B	620	-----
9	0—24	0—34½	42.7	M	584	-----	M	694	-----
10	0—23	0—34½	50.0	B	638	-----	B	745	-----
10A	0—22	0—34½	55.7	N	626	Measured on surface of column	N	718	Measured on surface of column
11	0—27	0—45½	67.6	B	570		N	758	
12	0—14	0—36	157.0	N	502		B	758	
13	0—52	1—11½	37.9	M	781	-----	M	872	-----
14	0—45	1—04½	42.8	M	623	-----	M	810	-----
15	0—40	0—49½	20.6	N	670	-----	N	810	-----
16	0—40	0—44½	11.2	M	671	-----	M	717	-----
17	0—40	0—41½	4.4	N	700	-----	N	730	-----
18	2—00	2—53	44.1	N	672	-----	N	788	-----
19	1—00	1—07½	12.1	M	831	-----	N	860	-----
20	1—10	1—24½	20.7	N	853	-----	M	909	-----
21	1—10	1—21½	16.8	M	820	-----	N	858	-----
22	1—40	5—14	214.0	M	504	-----	N	790	-----
23	2—30	2—52	14.7	N	573	556	M	650	635
24	2—00	2—24	20.0	N	504	498	N	614	609
25	0—57	1—07½	18.9	N	504	522	N	653	610
26	1—10	1—23½	19.3	N	512	501	N	606	598
27	2—11	2—58	35.9	T	607	-----	N	735	-----
28	3—30	6—33½	87.5	M	590	545	N	750	716
28A	3—31	7—09½	103.5	N	515	484	N	723	690
29	3—15	4—38½	42.8	N	565	525	N	735	693
30	3—45	7—16	93.8	N	570	535	N	732	702
31	2—46	4—11½	51.5	N	533	504	N	732	685
32	2—32	3—44	47.4	N	547	503	B	722	656
32A	2—50	4—02	42.4	M	576	486	N	736	656
33	7—50	*8—08	-----	B	546	533	B	*554	*541
33A	7—50	*8—07½	-----	M	536	522	M	*547	*532
34	6—34	7—58	21.3	N	598	574	N	718	690
34A	5—50	7—23	26.6	N	571	542	N	732	700
35	7—40	*8—07	-----	N	539	527	N	*558	*546
36	3—10	3—53½	22.8	B	603	571	B	695	664
37	6—10	7—34½	22.8	B	545	525	B	664	644
38	3—10	5—28½	73.0	N	531	516	N	696	683
39	3—20	3—41½	10.6	M	739	-----	M	794	-----
40	5—01	7—57	58.5	B	574	536	M	881	837
41	7—30	*8—24½	-----	N	527	496	N	*561	*535
42	8—00	*8—11½	-----	B	553	526	B	*560	*535
43	2—50	4—11	47.6	N	601	553	N	769	724
44	2—23	3—04½	28.8	N	592	520	N	727	677
45	1—41	1—47	5.9	B	519	510	B	575	570
46	4—55	6—43½	36.8	N	543	-----	N	695	-----
47	2—03	2—48½	37.2	N	568	-----	N	710	-----
48	1—32	1—50	19.6	M	542	537	M	647	636
49	1—21	1—40	23.4	M	561	-----	M	654	-----
50	0—55	1—05½	19.5	N	682	-----	N	775	-----
50A	1—44	2—59½	14.9	B	695	-----	B	778	-----
51	2—11	2—55½	4.8	N	617	546	N	670	601
51A	2—40	2—17½	9.7	N	600	549	N	678	622
52	1—32	1—40½	9.5	N	520	497	N	593	570
53	1—17	1—22½	6.8	N	580	535	N	659	603
54	2—20	3—17½	49.9	B	543	493	B	820	764
55	2—51	3—46½	32.6	M	684	527	M	859	692
56	2—30	3—33½	42.3	M	643	564	M	795	732
57	2—15	3—23	50.4	N	749	593	T	894	755
58	4—00	4—35½	14.9	T	624	523	T	714	591
59	1—25	1—33½	10.3	-----	-----	-----	-----	-----	-----
60	1—47	3—09½	77.0	N	561	532	N	750	737
61	0—45	0—50½	11.7	N	585	571	N	638	625
62	3—00	4—11½	39.7	M	584	-----	M	760	-----
63	2—20	2—57½	26.8	N	598	-----	N	730	-----

*Column loaded to failure after 3-hr. fire exposure.

TABLE 43.—TIME TO FAILURE, PERIOD OF EXPANSION, AND MAXIMUM COLUMN TEMPERATURES—Concluded

Test No.	Period of Expansion Hr.—Min.	Time to Failure, Hr.—Min.	Difference, Percent of Expansion Period	Maximum Observed Temperature in Metal, Deg. C.					
				At End of Expansion Period			At Failure		
				Couple Level	Edge	Average	Couple Level	Edge	Average
76	3—45	4—25½	18.0	N	553	524	N	663	619
77	4—10	4—42½	12.9	N	600	558	B	690	652
64	4—32	4—43½	4.1	T	513		T	868	
65	2—20	2—21½	1.1	N	479		N	490	
66	2—32	2—36	2.6	T	463		T	491	
67	5—01	5—31½	10.1	N	496				
67A	5—45	6—24½	11.4				M	980	
68	1—10	1—40½	43.9	N	606	516	N	754	877
69	5—30	7—13½	31.3	N	550	553	N	706	716
70	5—00	*8—40½		N	642		N	*797	
71	2—40	7—22½	176.6	T	468	Measured on vertical reinforcing bars	B	942	Measured on vertical reinforcing bars
72	5—00	*8—04½		T	475		T	*623	
73	4—10	7—57½	91.0	N	528		N	845	
74	5—50	*8—06½		B	493		N	*589	
75	4—50	*8—01½		B	552		B	*788	
78		2—15½					H	360	
79		0—50					H	510	
80		1—13					H	564	
81		0—35					H	432	
82		0—45½					H	510	
83		0—33½					H	544	

* Column loaded to failure after 8-hr. fire exposure. H. temperature measured on metal cap at edge of timber column bearing.

a large proportion of the applied load. The same effect appears to have influenced to much smaller extent the temperatures attained at the point of maximum expansion.

The surface temperatures at failure and at maximum expansion in the tests of cast-iron columns were on the average about 70° C (126° F) higher than the average over the section for structural-steel columns at the given stages of the test.

6. TOTAL VERTICAL DEFORMATION

(a) BEFORE FAILURE

Measurement of the movement of the head of the loading ram was made for a number of columns as described in Sec. VIII, paragraph 4, and the results are plotted in Figs. 46 and 47. An approximate measure of the total expansion and depression was also obtained from the card of the indicator (Sec. VI, par. 3b) mounted on the control board.

The total expansion of the steel columns varied from ⅞ to ⅓ inch and of the cast-iron columns from 1 to ½ inch, the lower values being due to local heating caused by impairment of the covering over a short length, which induced failure while other

FIG. 46.—Total expansion, test Nos. 10A, 76, 77, 110, 114, and 115

portions of the column were at much lower temperature. The expansion of the reinforced-concrete columns was about one-half that of the cast-iron columns.

The depression of the top of the timber columns is given in Fig. 47. These deformations were due mainly to crushing of the wood at the metal cap, the heat conducted by it into the bearing causing a large reduction in the compressive strength of the wood in contact with it.

(b) AT FAILURE

With the setting used for the cut-off valve of the cylinder, the resulting downward movement at failure varied between $2\frac{1}{2}$ and $3\frac{1}{4}$ inches. As given on the indicator card, from $\frac{1}{4}$ to $2\frac{1}{2}$ inches of this travel was made under nearly full pressure. The indicated pressure in the cylinder at the end of the travel varied from $\frac{1}{4}$ to $\frac{1}{2}$ of the original pressure in tests with steel columns and some of the cast-iron columns. In the case of the cast-iron columns that broke before the end of the travel and of the reinforced-concrete columns that failed under working load, the pressure indicated immediately before the valve cut off was less than $\frac{1}{4}$ of the original pressure.

7. LATERAL DEFLECTION

(a) BEFORE FAILURE

The center deflections observed, where large enough to have any bearing on the manner of failure of the column, are noted in the respective test logs. In most tests, decided deflection did not begin until after the point of maximum expansion was passed. The deflection observed at the last reading before failure, varied for different tests from less than $\frac{1}{8}$ to $2\frac{1}{2}$ inches. In almost all cases the direction of the deflection before failure was the same as that of the final buckle. Initial general bends up to $\frac{3}{16}$ of an inch appear to have had no influence on the direction of the deflection of the column either before or at failure.

The maximum measured deflections of the reinforced-concrete columns before the end of the 8-hour fire period varied from less than $\frac{1}{8}$ to $\frac{1}{4}$ of an inch.

(b) DEFLECTION AT FAILURE

The maximum deflections at failure of the steel columns as measured after they were taken out of the furnace, varied from

FIG. 47.—Depression of top of timber columns

less than 1 to $15\frac{3}{8}$ inches. The smaller deflections obtained in tests where the failure was due to local buckling of individual section members or by direct crushing. The sections that generally failed locally were the plate and channel section and the latticed sections, although the other sections developed this type of failure when exposed to local heating. The large deflections were coincident with more nearly uniform heating and failure with deflection of the column as a whole. The direction of the deflection conformed quite uniformly with the line of least rigidity of the section. The strains at failure caused the thinner plates and section members to buckle between rivet points. The rivets were seldom found to have sheared.

The cast-iron columns usually failed by breaking at one or more points. The curvature of the pieces indicated that deflections of over 9 inches had been sustained before fracture occurred. In a few cases loading was discontinued after the column failed to sustain working load and before fracture occurred, with resultant deflections of 3 to 5 inches.

The failure of the reinforced-concrete columns was due to local crushing and shearing and developed little deflection of the column as a whole.

The failure of the timber columns with the steel plate cap and timber strut bearing was accompanied by lateral slipping of the cap on its bearings due to the softening of the wood immediately adjacent to them. In the case of the timber columns with cast-iron cap and pintle bearing, the immediate cause of failure was fracture of the cap. In both cases the deformation in the wood at the cap, before slipping or fracture of the latter, was so large as to constitute a near equivalent of the ensuing failure.

8. LOG OF FIRE TESTS

The log of the tests includes visual observations during the test, the time of maximum expansion, the time of failure, and effects at failure. The latter was usually accompanied by general or local buckling, the point of maximum lateral deflection being in or near the most highly heated region of the column.

The references after each test give in order the figure number of the corresponding column views, time-temperature curves, the deformation and average temperature curves.

(a) UNPROTECTED COLUMNS

TEST NO. 1. ROLLED H.—7 mins.—trace of color on flange edges. 8 mins.—maximum expansion. Lateral deflection at middle of column, $\frac{1}{8}$ in. to west, increasing to $\frac{3}{4}$ in. at 10½ mins. 11¼ mins.—failure with buckling to west, maximum at 4½ ft. above base. (Figs. 58, 90, and 146.)

TEST NO. 2. PLATE AND ANGLE.—6 mins.—trace of color on flange edges increasing to cherry red at 15 mins. 17 mins.—maximum expansion. 16 mins.—deflection $\frac{1}{8}$ in. to east, increasing to $\frac{1}{4}$ in. at 18½ mins. 19¼ mins.—failure with buckling to east, maximum at 5½ ft. above base. (Figs. 58, 90, and 146.)

TEST NO. 3. PLATE AND CHANNEL.—10 mins.—trace of color on flanges. 11 mins.—maximum expansion. 14 mins.—failure with buckling to west, maximum at 7½ ft. above base. Column low cherry red in color. (Figs. 58, 90, and 146.)

TEST NO. 4. LATTICED CHANNEL.—9 mins.—maximum expansion. 10 mins.—trace of color on flange edges and web. 11 mins.—failure with buckling to east, maximum at 3½ ft. above base. (Figs. 58, 90, and 146.)

TEST NO. 5. Z BAR AND PLATE.—9 mins.—deflection of $\frac{1}{8}$ in. north increasing to $\frac{1}{4}$ in. north at 11½ mins. 10 mins.—dull red color noted. 11 mins.—maximum expansion. 14¼ mins.—failure with buckling to north, maximum at 6 ft. above base. (Figs. 58, 91, and 146.)

TEST NO. 6. I BEAM AND CHANNEL.—8 mins.—no deflection. 10 mins.—trace of color on flanges. 13 mins.—maximum expansion. 16 mins.—deflection of $\frac{3}{4}$ in. north. 17 mins.—failure with buckling to north, maximum at 5½ ft. above base. (Figs. 59 and 91.)

TEST NO. 7. LATTICED ANGLE.—10 mins.—trace of color noted in lattice. 11 mins.—maximum expansion. 14 mins.—failure with local buckling in each angle from 8 to 10 ft. above base. (Figs. 59 and 91.)

TEST NO. 8. STARRED ANGLE.—13½ mins.—deflection of $\frac{1}{8}$ in. northeast, increasing to $\frac{5}{8}$ in. north at 18½ mins. 15 mins.—traces of color increasing to red at 18 mins. 19 mins.—maximum expansion. 21½ mins.—failure with buckling to north, maximum at 5 ft. above base. (Figs. 59 and 91.)

TEST NO. 9. ROUND CAST IRON, ENDS RESTRAINED.—18 mins.—traces of color increasing to bright red at 32 min. 24 mins.—maximum expansion. 26 mins.—deflection of $\frac{3}{8}$ in. west, increasing to 1 in. west at 31 mins. 34¼ mins.—failure, buckling to west and breaking at 1½, 5, and 11 ft. above base. Thinnest metal on southeast side. (Figs. 60, 92, and 147.)

TEST NO. 10. ROUND CAST IRON.—20 mins.—dull red color increasing to cherry red at 28 mins. 23 mins.—maximum expansion. 26 mins.—deflection of $\frac{1}{4}$ in. north-west, increasing to $\frac{1}{2}$ in. west at 31 mins. 34½ mins.—failure, buckling to southwest and breaking about 5 ft. above base. Thinnest metal on northeast side. (Figs. 60, 92, and 147.)

TEST NO. 10A. ROUND CAST IRON.—26 mins.—no color noted. Deflections not measured. 22 mins.—maximum expansion. 34¼ mins.—failure, buckling to north and breaking in two about 5 ft. above base. Column glowing red. (Figs. 60, 92, and 147.)

TEST NO. 11. ROUND CAST IRON, CONCRETE FILLED.—33 mins.—traces of color increasing to dull red at 39 mins. 27 mins.—maximum expansion. 31 mins.—deflection of $\frac{3}{8}$ in. west, increasing to 1¼ in. west at 41 mins. 45¼ mins.—failure, buckling to west and breaking in two about 5 ft. above base. Thinnest metal on north side of section. (Figs. 60, 92, and 147.)

TEST NO. 12. STEEL PIPE, CONCRETE FILLED.—14 mins.—maximum expansion. 21 mins.—traces of color increasing to cherry red at 32 mins. 12½ mins.—deflection of ¼ in. northeast, increasing to ¾ in. northeast at 31 mins. 36 mins.—failure, buckling to northeast, maximum at 6½ ft. above base. (Figs. 60, 93, and 147.)

TEST NO. 13. REINFORCED STEEL PIPE, CONCRETE FILLED.—15 mins.—traces of color increasing to bright red at 38 mins. 52 mins.—maximum expansion. 4 mins.—slight deflection to northwest noted, increasing to ¾ in. at 29 mins., 1⅝ in., at 1 hour, and 2⅝ in. northwest at 1 hr., 8 mins. 1 hr. 11¼ mins.—failure, buckling to northwest, maximum at 6 ft. above base. (Figs. 60 and 93.)

(b) PARTLY PROTECTED COLUMNS

TEST NO. 14. ROLLED H, 1:2:4 JOLIET GRAVEL CONCRETE.—No cracks or spalling noted in concrete before failure. 45 mins.—maximum expansion. 25 mins.—slight deflection to west noted, increasing to ¼ in. at 46 mins. 1 hr. 4¼ mins.—failure, buckling to west, maximum at 7 ft. above base. Loud report at failure probably due to failure of concrete in compression.

After failure.—Concrete generally loose, due to large deflection at failure; no marked heat or dehydration effects. Two pieces of concrete filling 3¾ by 7⅞ by 10 in. tested on end three weeks after the fire test gave ultimate compressive strengths of 1440 and 1800 lb. per sq. in., respectively. (Figs. 61 and 94.)

TEST NO. 15. ROLLED H, 1:2:4 GRANITE CONCRETE.—25 min.—fine horizontal cracks appeared in concrete on west at center. 30 mins.—cracks on west opening up and extending, column otherwise unaffected; steel dull red. 40 mins.—maximum expansion. No spalling before failure. 24 mins.—slight deflection to west noted, increasing to ¼ in. at 36 mins. 48¼ mins.—failure, buckling to west, maximum at 7½ ft. above base.

After failure.—Concrete generally loosened, this probably occurring at failure. Piece of filling tested 9 days after the fire test had ultimate compressive strength of 960 lb. per sq. in. (Figs. 61 and 94.)

TEST NO. 16. PLATE AND ANGLE, 1:2:4 TRAP CONCRETE.—40 mins.—steel flanges dull red. Maximum expansion. No cracks or spalling noted in concrete before failure. 36 mins.—deflection of ⅓ in. to east. 44½ mins.—failure, buckling to the east, maximum at 5½ ft. above base.

After failure.—Concrete cracked and loose in several places. (Figs. 61 and 94.)

TEST NO. 17. PLATE AND ANGLE, 1:1½:4½ CINDER CONCRETE.—38 mins.—flanges of angles dull red. 40 mins.—maximum expansion. 41 mins.—no cracks or spalling of concrete. 41¾ mins.—failure, buckling to east, maximum at 5 ft. above base.

After failure.—Concrete cracked at points of maximum flexure and had fallen out to a depth of 1 in. in places. (Figs. 61 and 94.)

TEST NO. 18. LATTICED CHANNEL, 1:2:4 TRAP CONCRETE.—15 mins.—traces of color showing in concrete and all surfaces glowing at 30 min. 2 hrs.—maximum expansion. 2 hrs. 45 mins.—several vertical cracks ⅝ in. wide and about 18 in. long on east and west sides, mostly in lower half; concrete bulged out ½ to ¾ in. at cracks. 2 hrs. 53 mins.—failure, with local buckling of channels about 3½ ft. above base.

After failure.—Concrete in channels cracked along flanges in lower half and quite crumbly; concrete in upper half quite firm. A piece of concrete, filling the space between the channels near the middle of the column, 5½ by 9½ by 14 in. was tested on end 34 days after the fire test and developed an ultimate compressive strength of 577 lb. per sq. in. (Figs. 61 and 95.)

TEST NO. 19. Z BAR AND PLATE, 1:3:5 LIMESTONE CONCRETE.—20 mins.—traces of color in steel increasing to cherry red at 40 min. 50 mins.—concrete flowing. Very little cracking of concrete before failure. 1 hr.—maximum expansion. 35 mins.—deflection of $\frac{1}{4}$ in. north, increasing to $1\frac{1}{8}$ in. at 1 hr. 4 mins. 1 hr. $7\frac{1}{4}$ mins.—failure, buckling to north, maximum at 7 ft. above base.

After failure.—Concrete generally loose from steel on north and south sides, standing out to $\frac{7}{8}$ in. in places. A few horizontal cracks in the concrete were present on the east and west sides. A piece of filling $11\frac{1}{4}$ in. long tested 14 days after the fire test developed ultimate compressive strength of 730 lbs. per sq. in. (Figs. 62 and 96.)

TEST NO. 20. I BEAM AND CHANNEL, 1:3:5 TRAP CONCRETE.—34 mins.—trace of color on steel flanges. 36 mins.—color in concrete. 40 mins.—flanges dull red. 60 mins.—concrete and exposed steel bright red. 1 hr. 8 mins.—no cracks or spalling of concrete. 1 hr. 10 mins.—maximum expansion. 1 hr. $24\frac{1}{2}$ mins.—failure, buckling to south, maximum at 6 ft. above base.

After failure.—Concrete remained in place except where crushed at middle, top, and bottom. Rivet heads probably helped to hold it. (Figs. 62 and 96.)

TEST NO. 21. I BEAM AND CHANNEL, 1:3:5 TRAP CONCRETE.—38 mins.—traces of color on steel flanges. 1 hr., 10 mins.—maximum expansion. 1 hr. 20 mins.—no cracks or spalling of concrete. Steel flanges dark red, concrete bright red. 1 hr. $21\frac{3}{4}$ mins.—failure, buckling to north, maximum at 6 ft. above base.

After failure.—Concrete remained in place except where crushed at failure. (Figs. 62 and 96.)

TEST NO. 22. LATTICED ANGLE, 1:2:4 LIMESTONE CONCRETE.—25 mins.—concrete glowing on corners; a number of fine cracks noted on all faces. 1 hr. 40 mins.—maximum expansion. 3 hrs.—surface of column glowing at white heat. 4 hrs. 50 mins.—large vertical cracks appeared under brackets near top. Very little cracking or spalling of concrete before failure. 5 hrs. 14 mins.—failure with local buckling of steel and crushing of concrete about 11 ft. above the base. Unprotected brackets probably caused failure at this point by conducting heat into angles.

After failure.—Except at point of failure concrete did not appear greatly damaged. Limestone calcined to depth of 1 in., and one month after test the whole exterior concrete had become loose due to air slaking of the lime. From temperature of steel at 3 hrs. it is probable that the concrete carried most of the load after this time. (Figs. 62 and 95.)

(c) PLASTER ON METAL-LATH PROTECTIONS

TEST NO. 23. PLATE AND ANGLE, TWO 1-IN. LAYERS OF PORTLAND-CEMENT PLASTER ON EXPANDED METAL LATH.— $12\frac{1}{2}$ mins.—distinct thud heard caused by failure of covering due to expansion; plaster cracked and spalled on all faces at top of column near bottom of bracket. 23 mins. to 1 hr. 15 mins.—some six or eight fine vertical and horizontal cracks, 3 to 8 in. long, appeared on east, south, and west sides from 1 to 8 ft. above base. Very little change up to failure, except that all cracks opened up slightly. 2 hrs. 30 mins.—maximum expansion. 2 hrs. 41 mins.—deflection of $\frac{1}{8}$ in. northwest, increasing to 1 in. west at 2 hrs. 51 mins. 2 hrs. 52 mins.—failure, buckling to west, maximum at $7\frac{1}{2}$ ft. above base.

After failure.—Plaster of outer layer dehydrated and both outer and inner layers very crumbly. Both layers fairly well keyed to lath. (Figs. 63, 97, and 148.)

TEST NO. 24. PLATE AND CHANNEL, TWO $\frac{7}{8}$ -IN. LAYERS OF PORTLAND-CEMENT PLASTER ON WOVEN WIRE LATH.— $8\frac{1}{2}$ mins.—distinct thud heard; plaster cracked and spalled on all faces at bottom of bracket exposing lath on corners. 14 mins. to 1 hr. 45 mins.—some 15 fine vertical cracks, 3 to 8 in. long, appeared on all sides near corners. 2 hrs.—maximum expansion. 2 hrs. 15 mins.—very little change except some cracks opened up to $\frac{1}{8}$ in. 2 hrs. 24 mins.—failure, with local buckling about 6 ft. above base.

After failure.—Outer layer of plaster dehydrated; inner layer fairly hard except at failure point. Plaster well keyed to lath. (Figs. 63, 97, and 148.)

TEST NO. 25. Z BAR AND PLATE, ONE 1-IN. LAYER OF PORTLAND-CEMENT PLASTER ON EXPANDED METAL LATH.—10 mins.—slight noise heard; plaster cracked and spalled on all sides under bracket. 48 to 54 mins.—fine vertical cracks noted on all faces near corners, opening up slightly toward end of test. 57 mins.—maximum expansion. 41 mins.—slight deflection to north noted, increasing to $\frac{1}{4}$ in. at 1 hr. 1 min. 1 hr. 7 $\frac{3}{4}$ mins.—failure, buckling to north, maximum at 5 $\frac{1}{2}$ ft. above base.

After failure.—Plaster appeared to be in fairly good condition and was quite hard, except where crushed. Plaster well keyed, covering inner face of lath. (Figs. 63, 98, and 149.)

TEST NO. 26. LATTICED ANGLE, ONE 1 $\frac{1}{8}$ -IN. LAYER OF PORTLAND-CEMENT PLASTER ON EXPANDED METAL LATH.—10 to 17 mins.—cracking and some spalling of plaster on bracket. 20 mins. to 1 hr.—some 15 fine vertical cracks, 6 to 12 in. long, appeared near corners on all sides; also horizontal cracks near bottom. Before failure bracket cracks had opened up to $\frac{1}{2}$ in. and others nearly $\frac{1}{8}$ in. 1 hr. 10 mins.—maximum expansion. 1 hr. 23 $\frac{1}{2}$ mins.—failure, with local buckling about 6 ft. above base.

After failure.—Very little strength in plaster, very crumbly. Keys covered lath on inside. $\frac{3}{4}$ in. air space between plaster and angles. (Figs. 63, 98, and 149.)

TEST NO. 27. ROUND CAST IRON, ONE 1 $\frac{1}{2}$ -IN. LAYER OF PORTLAND-CEMENT PLASTER ON HIGH-RIBBED METAL LATH.—18 mins.—vertical and horizontal cracks under bracket. 1 hr. 40 mins. to 2 hrs. 5 mins.—several fine vertical and horizontal cracks, 3 to 12 in. long, mostly near middle. 2 hrs. 11 mins.—maximum expansion. 2 hrs. 50 mins.—cracks opening up to $\frac{1}{8}$ in. 2 hrs. 30 mins.—slight deflection to southwest noted, increasing to $\frac{1}{2}$ in. at 2 hrs. 51 mins., and 1 $\frac{1}{4}$ in. southwest at 2 hrs. 56 mins. 2 hrs. 58 mins.—failure, buckling to southwest and breaking about 6 ft. above base.

After failure.—Plaster fairly hard where not crushed. Plaster pushed through to iron except at ribs; average thickness of plaster 1 $\frac{1}{2}$ in. (Figs. 63, 98, and 149.)

(d) CONCRETE PROTECTIONS

TEST NO. 28. ROLLED H. 2-IN. 1:2:4 LIMESTONE CONCRETE.—30 to 45 mins.—a few fine vertical cracks at middle and top of east face opening up slightly toward failure. No spalling before failure. 3 hrs. 30 mins.—maximum expansion. 6 hrs. 33 $\frac{3}{4}$ mins.—failure, buckling to west, maximum at 6 $\frac{1}{2}$ ft. above base.

After failure.—Concrete fairly hard although calcined on surface. Flanges exposed at middle and flange edges exposed at several points, this all occurring at failure. Wire tie not broken. (Figs. 64 and 99.)

TEST NO. 28A. ROLLED H. 2-IN. 1:2:4 LIMESTONE CONCRETE. NOT TIED.—2 hrs. 55 mins. to 3 hrs. 23 mins.—several fine vertical cracks 3 to 12 in. long and about 3 in. from corners appeared on east and west faces. No spalling or other effects noted before failure. 3 hrs. 31 mins.—maximum expansion. 5 hrs.—slight deflection to west noted, increasing to $\frac{1}{4}$ in. at 6 hrs. 2 mins., and 1 $\frac{1}{2}$ in. west at 7 hrs. 2 mins. 7 hrs. 9 $\frac{1}{4}$ mins.—failure, buckling to west, maximum 6 $\frac{3}{4}$ ft. above base.

After failure.—Concrete surface fairly firm although discoloration and calcination extend to depth of 1 in. Concrete fell off at failure, exposing about one-half of area of steel flanges. (Figs. 64, 100, and 150.)

TEST NO. 29. ROLLED H. 2-IN. 1:2:4 TRAP CONCRETE.—50 mins. to 3 hrs. 50 mins.—several fine vertical and horizontal cracks appeared on east and west faces, which became larger near failure. No spalling or other effects noted before failure. 3 hrs. 15 mins.—Maximum expansion. 3 hrs. 30 mins.—slight deflection to west noted, increasing to $\frac{1}{4}$ in. at 4 hrs. 2 mins., and 1 $\frac{1}{2}$ in. west at 4 hrs. 32 mins. 4 hrs. 38 $\frac{1}{2}$ mins.—failure, buckling to west, maximum at 5 $\frac{1}{2}$ ft. above base.

After failure.—Surface of concrete reddish in color. No fusion noted. Concrete fell off at failure exposing flange for $1\frac{1}{2}$ ft. on south side and flange edges at other points. Wire tie not broken. (Figs. 64, 101, and 151.)

TEST No. 30. ROLLED H. 2-IN. 1:2:4 JOLIET GRAVEL CONCRETE.—37 mins.—fine cracks noted in brackets. 3 hrs. 4 mins. to 5 hrs. 5 mins.—a number of fine cracks, 2 to 12 in. long, mostly vertical and near edges, appeared on east and west faces opening up to $\frac{1}{8}$ of an in. near end of test in some cases. No spalling before failure. 3 hrs. 45 mins.—maximum expansion. 6 hrs. 40 mins.—slight deflection to east noted, increasing to $\frac{3}{8}$ in. at 7 hrs. 9 mins. 7 hrs. 16 mins.—failure, buckling to east, maximum at $5\frac{1}{2}$ ft. above base.

After failure.—Concrete on surface fairly hard after test, but disintegrated in 30 days due to air slaking. Flanges and flange edges exposed in places due to spalling of concrete at failure. (Figs. 64 and 102.)

TEST No. 31. ROLLED H. 2-IN. 1:2:4 SANDSTONE CONCRETE.—28 mins. to 1 hr. 5 mins.—vertical cracks appeared on east and west faces about 3 in. from edges, running nearly full length of column, opening up in places to $\frac{1}{8}$ in.; also a few fine vertical cracks on north and south faces in lower half. 45 mins.—southeast corner spalled off 3 in. deep from 3 to 6 ft. up, exposing edge of steel. 1 hr. 5 mins. to 3 hrs.—vertical cracks at corners became continuous, opening up to $\frac{1}{8}$ in.; concrete on southeast and southwest corners generally spalled off or loose in lower half, exposing edges of flanges in places. Very little change to failure except for minor cracking and spalling. 2 hrs. 46 mins.—maximum expansion. 2 hrs.—slight center deflection to northwest noted, increasing to $\frac{5}{8}$ in. at 3 hrs. 1 min., and $2\frac{1}{4}$ in. northwest at 4 hrs. 11 mins. 4 hrs. 11½ mins.—failure, buckling to west, maximum at $5\frac{1}{2}$ ft. above base.

After failure.—Concrete on flanges disintegrated and crumbly. Nearer the web the concrete was harder but had apparently lost much strength. (Figs. 64, 103, and 152.)

TEST No. 32. ROLLED H. 2-IN. 1:2:5 CINDER CONCRETE.—3 hrs.—no cracking or spalling noted. 3 hrs. 20 mins. to 3 hrs. 38 mins.—a few small vertical cracks 4 to 6 in. long appeared near corners on east and west faces in lower half extending in length and opening up toward end of test; no spalling before failure. 2 hrs. 32 mins.—maximum expansion. 3 hrs.—slight deflection to east noted, increasing to $\frac{1}{2}$ in. at 3 hrs. 31 mins., and $1\frac{5}{8}$ in. east at 3 hrs. 43 mins. 3 hrs. 44 mins.—failure, buckling to east about 6 ft. above base.

After failure.—Concrete very crumbly to a depth of about 1 in. Quite hard and apparently little affected at column web 5 in. from surface. (Figs. 65, 104, and 153.)

TEST No. 32A. Z BAR AND PLATE. 2-IN. 1:2:5 CINDER CONCRETE.—2 hrs. 50 mins.—maximum expansion. 3 hrs.—no cracking or spalling noted. 3 hrs. 50 mins.—fine vertical cracks developing on all faces mostly near northeast and southwest corners. 3 hrs. 30 mins.—slight deflection to north noted, increasing to $\frac{3}{8}$ in. at 3 hrs. 51 mins., and $1\frac{5}{8}$ in. north at 4 hrs. 1 min. 4 hrs. 2 mins.—failure, buckling to north, maximum at $5\frac{1}{2}$ ft. above base.

After failure.—Concrete crumbly and dehydrated to a depth of about $1\frac{1}{2}$ in. Beyond this it was harder and apparently little affected. (Figs. 65, 104, and 153.)

TEST No. 33. ROLLED H. 4-IN. 1:2:4 LIMESTONE CONCRETE.—2 hrs. to 4 hrs.—fine vertical cracks 6 to 12 in. long on east and west faces, 4 in. from corners, in lower half; also small cracks in bracket; cracks opening slightly toward end of test; no spalling. 7 hrs. 50 mins.—maximum expansion. 8 hrs.—column still supporting working load, with no apparent change; less than $\frac{1}{8}$ in. deflection. 8 hrs. 2 mins.—load increased, with fire going until failure occurred under load of 431 000 lb., with local buckling about 10 ft. above base, at 8 hrs. 8 mins.

After failure.—Outer $\frac{3}{4}$ in. of concrete soft and could be easily knocked off 8 days after test, and was dehydrated for another $1\frac{1}{2}$ in., inside of which it was hard and apparently little affected by the heat. (Figs. 65, 105, and 154.)

TEST No. 33A. ROLLED H. 4-IN. 1:2:4 LIMESTONE CONCRETE. NOT TIED.—30 mins.—slight surface flaking at several points. 34 to 40 mins.—fine cracks noted on east and west sides of bracket. 2 hrs. 10 mins. to 5 hrs. 20 mins.—three or four fine vertical cracks 3 to 24 in. long on east and west faces, 3 in. from corners, in lower half; no spalling or further cracking before failure. 7 hrs. 50 mins.—maximum expansion. 8 hrs.—column still supporting load with no apparent change; less than $\frac{1}{8}$ in. deflection. 8 hrs. 5 mins.—load increased with fire going until failure occurred under load of 405 000 lb. with buckling to the east, maximum 7 ft. above base, at 8 hrs. $7\frac{1}{4}$ min.

After failure.—In lower 8 ft. concrete checked and pitted with small holes of $\frac{1}{8}$ to $\frac{1}{4}$ in. diameter, and hard to depth of over $\frac{1}{8}$ in. Concrete calcined to depth of $1\frac{1}{2}$ in. from surface. Concrete on flanges broke loose at failure due, in part, to absence of wire tie. (Figs. 65 and 106.)

TEST No. 34. ROLLED H. 4-IN. 1:2:4 GRANITE CONCRETE.—32 mins. to 1 hr. 5 mins.—fine vertical cracks about 3 in. from corners mostly on east and south sides, 5 to 9 ft. up; also cracks on all sides in bracket. 1 hr. 28 mins.—cracks opened generally $\frac{1}{8}$ to $\frac{3}{8}$ in. width. 2 hrs. 13 mins.—crack through concrete at southeast corner 6 ft. up. 4 hrs. 7 mins.— $\frac{1}{4}$ -in. crack through northeast corner 6 ft. up. 6 hrs. 34 mins.—maximum expansion. 6 hrs. 40 mins.—very little change except for slight spalling on northwest corner 8 ft. up. 7 hrs. 33 mins.—spalling, northwest corner, 6 by 3 by 3 in. at 8 ft. up; spalling, southeast corner, 6 by 6 by 20 in. $6\frac{1}{2}$ ft. up, exposing steel. 3 hrs.—slight deflection to east noted, increasing to $\frac{3}{8}$ in. at 7 hrs. 31 mins. 7 hrs. 58 mins.—failure, buckling to east, maximum at 7 ft. above base.

After failure.—Incipient fusion of concrete $1\frac{1}{2}$ to 2 in. in depth throughout. Concrete underneath very soft and crumbly. (Figs. 65, 107, and 155.)

TEST No. 34A. ROLLED H. 4-IN. 1:2:4 GRANITE CONCRETE.—55 mins. to 1 hr. 45 mins.—small vertical cracks appeared on all faces 3 to 5 in. from corners, mostly from 5 to 7 ft. up, opening up to $\frac{1}{8}$ in. in some cases; also several cracks in bracket. 3 hrs. 16 mins.—cracks opening up to $\frac{1}{4}$ in., no spalling except small piece at bracket. 4 hrs. 44 mins.—gas shut off for $1\frac{1}{2}$ mins. to cool furnace. 5 hrs. 50 mins.—maximum expansion. 6 hrs. 30 mins.—gas shut off for 2 mins. to clear furnace; small spall noted on southwest corner $7\frac{1}{2}$ ft. up. 7 hrs.—gas shut off for 2 mins.; incipient fusion apparently present. Minor spalling of corners noted. 6 hrs. 30 mins.—slight deflection to west noted, increasing to $\frac{1}{2}$ in. at 7 hrs. 21 mins. 7 hrs. 23 mins.—failure, buckling to west, maximum at 5 ft. above base.

After failure.—Incipient fusion of concrete 1 to $1\frac{1}{2}$ in. in depth over whole surface. (Figs. 65 and 108.)

TEST No. 35. ROLLED H. 4-IN. 1:3:5 LIMESTONE CONCRETE.—56 mins.—fine vertical crack, 12 ins. long, on both faces at southeast corner at bottom, 4 in. from edge. 3 hrs.—no spalling or further cracking noted. 4 to 7 hrs.—furnace filled with heavy flame; impossible to see column. 7 hrs. 40 mins.—maximum expansion. 8 hrs.—column still supporting working load with no apparent change; deflection less than $\frac{1}{8}$ in.; load increased with fire going until failure occurred under load of 348 000 lbs. with local buckling $7\frac{1}{2}$ ft. above base, at 8 hrs. 7 mins.

After failure.—Surface concrete hard and sand fused throughout to depth of $\frac{1}{8}$ in. to $\frac{1}{4}$ in.; limestone completely calcined up to 1 in. from surface, and soft and dehydrated up to $2\frac{1}{2}$ ins. from surface. Concrete was harder farther in, although less so than in 33 and 33A. (Figs. 66, 109, and 156.)

TEST No. 36. PLATE AND ANGLE. 2-IN. 1:2:4 TRAP CONCRETE.—37 mins.—small crack on west face, 5 in. long, running up diagonally from southwest corner, $2\frac{1}{2}$ ft. up. 1 hr. 13 mins.—small spall, 3 by $\frac{1}{4}$ in. on southeast corner, $6\frac{1}{2}$ ft. up. 1 hr. 20 mins. to 3 hrs. 40 mins.—several fine cracks at bracket. 3 hrs. 10 mins.—maximum expansion. 3 hrs. 46 mins.—several vertical cracks 2 in. from corners on east and west faces, 2 ft. to 4 ft. up; no spalling before failure. 3 hrs.—slight deflection to northwest noted, increasing to $\frac{7}{8}$ in. at 3 hrs. 44 mins. 3 hrs. $53\frac{1}{4}$ mins.—failure, buckling to west, maximum at 6 ft. above base.

After failure.—Concrete soft and crumbly to a depth of $1\frac{1}{2}$ to 2 in. (Figs. 66, 103, and 151.)

TEST No. 37. PLATE AND ANGLE. 4-IN. 1:2:4 TRAP CONCRETE, ROUND SECTION.—35 mins. to 1 hr.—small cracks noted on all sides at bracket. 1 hr. 25 mins. to 1 hr. 40 mins.—spalling of southwest and southeast corners at bracket. 4 hrs. 20 mins.—no further cracking or spalling. 6 hrs. 10 mins.—maximum expansion. 6 hrs. 24 mins.—spalling across west face at bracket about 1 in. deep. 7 hrs. 10 mins.—spalling south face at bracket; impossible to observe lower part of column on account of furnace gases. 6 hrs. 30 mins.—slight deflection to west noted, increasing to $\frac{1}{2}$ in. at 7 hrs. and $1\frac{3}{4}$ in. west at 7 hrs. 30 mins. 7 hrs. $34\frac{1}{2}$ mins.—failure, buckling to west, maximum at $5\frac{1}{2}$ ft. above base.

After failure.—Considerable fusion and flowing of concrete to depth of 2 in. and incipient fusion for 1 to $1\frac{1}{2}$ in. farther in, in lower 8 ft. Partial fusion to $\frac{1}{2}$ in. depth above this point. Concrete crumbly for 1 in. inside fused portion; fairly hard farther in. Trap rock extends to surface quite generally in upper portion which was not fused. (Figs. 66, 110, and 157.)

TEST No. 38. PLATE AND CHANNEL. 2-IN. 1:2:4 JOLIET GRAVEL CONCRETE.—45 mins. to 2 hrs. 55 mins.—several fine vertical cracks appeared on east and west faces about $2\frac{1}{2}$ in. from corners, 5 to 8 ft. above base, some opening up to $\frac{1}{8}$ in., also several cracks in bracket. 3 hrs. 10 mins.—maximum expansion. 3 hrs. 26 mins.—horizontal cracks on north face 4 ft. up, $\frac{1}{4}$ in. wide. 4 hrs. 6 mins.—vertical cracks on east and west faces opening up to $\frac{1}{8}$ in.; very few cracks on north and south faces. 5 hrs. 4 mins.—concrete loose on corners 3 ft. up. 5 hrs. 28 mins.—no spalling. 4 hrs.—slight deflection to east noted, increasing to $\frac{1}{2}$ in. at 5 hrs. 21 mins. 5 hrs. $28\frac{3}{4}$ mins.—failure, buckling to east, maximum at $2\frac{1}{2}$ ft. above base.

After failure.—Concrete calcined to depth of $\frac{3}{4}$ in. and very crumbly to depth of 2 in. Inside of this point on web side concrete was quite hard and apparently little affected by the heat. (Figs. 66, 111, and 158.)

TEST No. 39. PLATE AND CHANNEL. 4-IN. 1:2:4 MERAMEC RIVER GRAVEL CONCRETE.—19 to 26 mins.—vertical cracks developed on east and west faces near corners in lower half and at bracket. 26 mins.—2-ft. spall on northwest corner near middle. 28 mins. to 40 mins.—vertical cracks extending upward and opening up. 40 mins.—horizontal crack, 5 ft. up, across north face. 45 mins.—large spall on southwest corner, 7 ft. up. 1 hr. 20 mins.—cracks opening up to $\frac{1}{2}$ in. in some cases. 1 hr. 26 mins.—concrete cracked through northeast corner, this corner spalling off from 4 to 8 ft. up at 2 hrs. 6 mins., exposing edge of steel for 4 ft. 2 hrs. 13 mins.—southwest corner spalled off for 3 ft. near middle exposing steel. 3 hrs.—northwest corner and west side spalled off above middle of column exposing steel on corner for 2 ft. 3 hrs. 20 mins.—maximum expansion. 3 hrs. $41\frac{1}{4}$ mins.—failure with local buckling $6\frac{1}{2}$ to 8 ft. above base.

After failure.—Concrete inside of spalled or cracked portions was quite hard and split with fracture of aggregate. Aggregate also fractured by cracks produced in the fire test. (Figs. 66 and 112.)

TEST NO. 40. LATTICED CHANNEL. 2-IN. 1:2:4 TRAP CONCRETE, ROUND SECTION.—5 hrs. 1 min.—maximum expansion. 6 hrs. 56 mins.—no cracking, spalling, or fusion of concrete. 7 hrs. 25 mins.—a number of vertical cracks on east and west sides in lower half, varying from very fine to one $\frac{1}{8}$ by 16 in. at southeast, 3 ft. up. No spalling or fusion. 7 hrs. 55 mins.—cracks opening up; the crack noted above open $\frac{1}{4}$ in.; no spalling, possibly some fusion. 7 hrs. 57 mins.—failure with local buckling about 5 ft. above base.

After failure.—Fusion of concrete from depth of $\frac{1}{2}$ to $1\frac{1}{2}$ in. near failure point. Very little concrete had run. Concrete outside of steel generally disintegrated; between channels it is harder. (Figs. 67, 113, and 159.)

TEST NO. 41. Z BAR AND PLATE. 4-IN. 1:3:5 LIMESTONE CONCRETE.—10 mins.—fine vertical crack, 2 in. long, on west face near north corner 7 ft. up. 1 hr. to 1 hr. 50 mins.—several vertical cracks, 3 to 12 in. long, appeared on west face near corners in middle section of column; some opening up to $\frac{1}{8}$ in.; no spalling. 2 hrs. 50 mins.—concrete cracked through for 2-in. length on northwest corner, 5 ft. up. 7 hrs. 30 mins.—maximum expansion; lateral deflection at middle of column less than $\frac{1}{8}$ in. 8 hrs.—column still supporting working load with little apparent change; no spalling; load increased with fire going until failure occurred under load of 332 000 lbs., buckling to south, maximum 6 ft. above base, at 8 hrs. 24 $\frac{3}{4}$ mins.

After failure.—Concrete reddish in color and dehydrated to depth of 1 in. from surface. Concrete outside of flanges dehydrated and crumbly. (Figs. 67, 114, and 160.)

TEST NO. 42. Z BAR AND PLATE. 4-IN. 1:3:5 LIMESTONE CONCRETE.—1 hr. 15 mins.—small crack on southwest corner at bracket. 2 hrs. 20 mins.—fine vertical crack on west face 4 in. from northwest corner, 4 ft. up. 8 hrs.—maximum expansion; column still supporting working load with little apparent change; no spalling; deflection less than $\frac{1}{8}$ in.; load increased with fire going until failure occurred under load of 333 000 lb. with buckling to north, maximum 6 ft. above base, at 8 hrs. 11 $\frac{1}{2}$ mins.

After failure.—Concrete affected same as in test No. 41. (Figs. 67, 115, and 161.)

TEST NO. 43. I BEAM AND CHANNEL. 2-IN. 1:2:4 SANDSTONE CONCRETE.—13 mins.—vertical crack, 4 in. long, on southeast corner, $5\frac{1}{2}$ ft. up, causing slight spalling at 20 mins. 21 to 30 mins.—vertical cracks appeared on east and west faces near corners, mostly in middle part of column. 30 mins.—southeast corner spalled off to a height of $5\frac{1}{2}$ ft., exposing edge of steel and ties for 4 ft. 33 to 41 mins.—vertical cracks on east and west faces extended nearly full length of column. 45 mins.—southwest corner spalled off 4 to 10 ft. up. 51 mins.—southeast corner spalled off 6 to 9 ft. up. 55 mins.—cracks opening up to $\frac{1}{4}$ in. in places. 1 hr. 15 mins.—southwest corner spalled off, lower 4 ft. 2 hrs. 50 mins.—maximum expansion. 3 hrs. 5 mins.—concrete checking all over surface. 4 hrs. 11 mins.—failure, buckling to north, maximum 6 ft. above base.

After failure.—Channel flanges and ties on south oxidized. Concrete discolored on surface and very crumbly for depth of 1 in., the rest dehydrated and possessed little strength. Fractures split aggregate, which also seems to have lost much of its strength, crumbling under light hammer blows. (Figs. 67, 116, and 152.)

TEST NO. 44. I BEAM AND CHANNEL. 2-IN. 1:3:5 SANDSTONE CONCRETE.—17 mins.—vertical crack on northeast corner, 2 to 5 ft. up, corner spalling off at 18 mins., also vertical crack on northeast corner, 3 to 7 ft. up, corner spalling off at 25 mins. 22 mins.—vertical cracks on both sides of northwest and southwest corners. 32 mins.—southeast corner spalled off in lower 3 ft. 35 mins.—vertical cracks on all faces about 2 in. from corners running full length of column. Edges of steel exposed for lower 7 ft. on northeast and southeast corners; concrete loose on northwest and southwest corners. 36 $\frac{1}{2}$ mins.—southwest corner spalled from 6 to 7 ft. up. 1 hr. 2 mins.—spall east side near middle for length of 3 ft., exposing flange edges. 1 hr. 23 mins.—

all corners at bracket spalled or cracked. 1 hr. 24 mins.—northwest corner spalled from 6 to 9 ft. up. 1 hr. 35 mins.—southeast flange of channel dull red at middle of column; no other change. 2 hrs. 23 mins.—maximum expansion. 3 hrs. 4¼ mins.—failure, buckling to north, maximum 6 ft. above base.

After failure.—Concrete generally very soft and crumbly. Falls off column readily in large pieces. (Figs. 67 and 116.)

TEST NO. 45. STARRED ANGLE. 2-in. 1:2:4 MERAMEC RIVER GRAVEL CONCRETE, ROUND SECTION.—20 to 30 mins.—a number of deep cracks, mostly vertical, in lower half of column; concrete becoming loose in places. 35 mins.—some spalling near middle of column. 36 mins.—cracks in upper half of column; practically all sides have large cracks. 38 mins.—crack on west, 3 ft. up, open 3 in. exposing steel. 44 mins.—spalling to steel on west. 47 mins.—spalling to steel on north and west near bottom. 55 mins.—wire tie appeared to be holding concrete between angles in place. 56 mins.—edge of steel exposed on west from 1 ft. to 6 ft. up, on north from 1 to 2 ft. up. 1 hr. 7 mins.—steel exposed on south, 1 to 2 ft. up. 1 hr. 10 mins.—large pieces spalled on north side near middle of column. 1 hr. 34 mins.—concrete on lower part of bracket spalled off. 1 hr. 41 mins.—maximum expansion. 1 hr. 47 mins.—failure by local buckling about 3 ft. above base.

After failure.—Coarse aggregate was quite generally broken on fracture planes. Concrete inside of fractures appeared to be in good condition. (Figs. 66 and 112.)

TEST NO. 46. LATTICED ANGLE. 2-in. 1:2:4 TRAP CONCRETE.—1 hr.—diagonal crack, southeast corner at bracket. 2 hrs. 35 mins.—vertical crack, 3 in. long, on east face, 4 in. from southeast corner, 8 ft. up. No spalling before failure. 4 hrs. 55 mins.—maximum expansion. 6 hrs. 43½ mins.—failure with local buckling about 6 ft. above base, each angle deflecting outward about 2½ in.

After failure.—Incipient fusion on surface of concrete with small cracks. Concrete crumbly from dehydration to 1½ in. from surface. It was otherwise apparently in fair condition, except where crushed at point of failure. (Figs. 67 and 117.)

TEST NO. 47. ROUND CAST IRON. 2-in. 1:2:5 CINDER CONCRETE. NOT TIED.—50 mins.—piece of concrete 2 in. diameter, spalled off under bracket on south, exposing bracket; also crack 6 in. long extended down from spall. 2 hrs. 3 mins.—maximum expansion. 2 hrs. 42 mins.—vertical cracks noted on all sides at middle of column. 2 hrs. 47 mins.—concrete fell off from 2 to 10 ft. up, exposing metal. 2 hrs.—slight deflection to south noted, increasing to ¼ in. at 2 hrs. 20 mins. 2 hrs. 48¾ mins.—failure with buckling to south, maximum 6 ft. above base.

After failure.—Column broken up and thickness found to vary from ½ to 1⅞ in., with thinnest metal on north. A short horizontal crack had formed on north side 3 ft. above base. Concrete crumbly for ¾ in. from surface. Inside of this it appears almost unaffected by the fire test. (Figs. 67, 101, and 158.)

(e) HOLLOW CLAY-TILE PROTECTIONS

TEST NO. 48. ROLLED H. 2-IN. SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT. NO FILLING.—7 mins.—a number of vertical joints open about ⅛ in. 12 mins.—vertical cracks in center of north, east, and south sides, 5 to 9 ft. up; cracks in about ⅓ of all vertical joints. 20 mins.—vertical cracks extending and in some cases opening to ¼ in.; cracks on all sides at bracket, and lower edge of tile spalled off on southwest corner at bracket. 30 mins.—north face cracked, 2 to 10 ft. up; vertical cracks on east from 3 to 10½ ft., on south from 5 to 9 ft. up; cracks in some cases open ¾ in. 1 hr. 15 mins.—outer shell of tile spalled off along lower west edge at bracket; little change in tile below; no spalling below bracket before failure. 1 hr. 32 mins.—maximum expansion. 1 hr. 40 mins.—slight deflection to northwest noted, increasing to ⅝ in. at 1 hr. 49 mins. 1 hr. 50 mins.—failure, buckling to west, maximum 8 ft. above base.

After failing.—At failure tile fell off from 5 ft. up to top course of bracket. Lower two or three courses almost intact. (Figs. 68, 118, and 162.)

TEST NO. 49. ROLLED H. 4-IN. SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT. NO FILLING.—3½ to 8½ mins.—a number of vertical cracks in tile and joints in lower 8 ft. opened $\frac{1}{8}$ to $\frac{1}{4}$ in. 12 to 24 mins.—cracks opening $\frac{1}{4}$ to $\frac{5}{8}$ in. maximum; west face bulged out $\frac{3}{4}$ in. from 4 to 8 ft. up. 22 mins.—ties down at 9th and 12th courses. 25 mins.—tile in middle section of column on south and west sides cracked and crushed. 1 hr. 27 mins.—cracks opening up to $\frac{3}{8}$ in. in places; no extension of cracks or spalling. 1 hr. 20 mins.—slight deflection to north noted, increasing to $\frac{1}{4}$ in. at 1 hr. 38 min. 1 hr. 21 mins.—maximum expansion. 1 hr. 40 mins.—failure with compound buckling to northwest 8 ft. above base and to southeast 6 ft. above base.

After failure.—A few tile intact, without cracks; tile generally cracked longitudinally through the shells, not often cracked transversely across webs. This probably accounts for the fact that no spalling of shells occurred until very near failure. (Figs. 68, 118, and 162.)

TEST NO. 50. PLATE AND ANGLE. 2-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL. (TILE 6 IN. WIDE, LAID HORIZONTALLY.)—7 mins.—cracking and spalling on corners, 4 to 6 ft. up. 10 mins.—tile spalled, concrete exposed on south and east for 1 ft. near bottom. 16 mins.—tile bulged out on south face from bottom to 6½ ft. up. 21 mins.—tile fell off on south where bulged, exposing steel. 26 mins.—concrete exposed from 4 to 6½ ft. up on north, east, and west. 32 mins.—cracks opening up at joints in upper half. 55 mins.—maximum expansion. 1 hr. 5¾ mins.—failure, with local buckling about 5½ ft. above base.

After failure.—Tile broke clear from concrete with no splitting of tile at keys. Transverse shearing of long tile at joints of short tile very marked. Tile in all courses fell in whole or part except top bracket course and three lower courses. Concrete in fill where not crushed by compression apparently uninjured. (Figs. 68 and 119.)

TEST NO. 50A. PLATE AND ANGLE. 2-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL. (TILE 6 IN. WIDE, LAID HORIZONTALLY.)—5 mins.—several horizontal joints open $\frac{1}{8}$ in.; vertical crack 12 in. long through tile on east face, 6¾ ft. up; large number of vertical cracks near corners at bracket. 7 mins.—part of one outer shell on each of south and west, near middle spalled. 13 mins.—vertical cracks $\frac{1}{4}$ in. wide opposite joints at all corners. 13 to 20 mins.—outer shell spalled off for 2 ft. at middle on east and west; outer shell bulged out $\frac{1}{2}$ in. in several places. 21 mins.—edge of steel exposed for 6 in. on northeast corner, 6 ft. up. 21 to 39 mins.—tile fell off, exposing concrete for 3 ft. at middle on east and west; buckling of tile increasing on flanges to $\frac{3}{4}$ in. maximum; upper 3 ft. intact, except for $\frac{1}{8}$ -in. cracks at bracket. 1 hr. 14 mins.—tile fell off, exposing concrete in lower 3 ft. on west face; otherwise little change. 1 hr. 24 mins.—mortar joints evidently not very full on north and south, can see through in places. 1 hr. 42 mins.—tile buckled out 1 in. on north, 5 ft. up. 1 hr. 44 mins.—maximum expansion. 1 hr. 59½ mins.—failure, buckling to west, maximum 5¼ ft. above base.

After failure.—All tile fell off at failure except parts of courses in lower foot and in upper 3 feet. Concrete fully fills reentrant portions except where crushed out at point of failure. (Figs. 69, 119, and 163.)

TEST NO. 51. PLATE AND ANGLE. 4-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL.—7 to 17 mins.—considerable cracking and bulging, mostly in middle courses, where some corners were shattered. 14 mins.—vertical cracks in about 50 per cent of tile. 18 to 38 mins.—bulging increasing, vertical cracks opening to $\frac{3}{4}$ in. maximum; minor spalling on southeast and southwest corners in middle courses. 20 mins.—ties on 2d and 8th courses down. 36 mins.—tie on 6th course down. 40 to 52 mins.—outer shell of tile spalled off from 5 to 7 ft. up on south and

west. 1 hr. 22 mins.—bulging of tile increased to $1\frac{1}{2}$ in. maximum. 1 hr. 47 mins.—tile fell on north, to 7 ft. up, exposing mortar and concrete. 1 hr. 30 mins.—slight deflection to west noted, increasing to $\frac{1}{8}$ in. at 2 hr. 2 hrs. 11 mins.—maximum expansion. 2 hrs. 17 $\frac{1}{4}$ mins.—failure, buckling to west, maximum 6 ft. above base.

After failure.—Tile generally shattered. Parts of tile near bottom blackened and partly fused by the heat. Concrete fills web spaces fairly; it is apparently little injured. Mortar joint on flanges not full in places, but voids are partly filled with concrete. (Figs. 69 and 120.)

TEST NO. 51A. PLATE AND ANGLE. 4-IN. SURFACE-CLAY TILE, BOSTON DISTRICT. GRANITE CONCRETE FILL.—3 to 15 mins.—considerable cracking and some bulging at corners. 16 mins.—cracks quite general, open to $\frac{1}{4}$ in. in lower two-thirds, not over $\frac{1}{8}$ in. above. 16 to 44 mins.—cracks increasing and opening up; bulging out of tile at horizontal joints to maximum of 2 in., mostly in middle four courses. 38 mins.—tie broken on 5th course. 1 hr. 6 mins.—cracks and bulges opening up in lower 8 ft.; little change above. 1 hr. 14 mins.—ties dropped on 2d and 3d courses. 1 hr. 20 mins. to 1 hr. 32 mins.—outer shells on east and west sides in middle courses spalling; at end of this period tile had fallen, exposing concrete from 4 to 8 ft. up on east and from 4 to 10 ft. up on west. 1 hr. 44 mins.—all ties down except on three upper courses. 1 hr. 45 mins.—tile buckled away from flanges on north and south 6 ft. up. 2 hrs. 6 mins.—tile spalled on north 6 ft. up, exposing east flange 3 by 4 in. 2 hrs. 23 mins.—tile buckled away from flanges on north and south 8 $\frac{1}{2}$ ft. up. 2 hrs. 26 mins.—no change in upper courses. 2 hrs. 40 mins.—maximum expansion. 2 hrs. 54 mins.—tile fell on south 6 ft. up, exposing steel. 2 hrs. 41 mins.—lateral deflection at center less than $\frac{1}{8}$ in. 2 hrs. 55 $\frac{1}{2}$ mins.—failure with buckling to west, maximum at 5 $\frac{1}{2}$ ft. above base.

After failure.—Upper 2 $\frac{1}{2}$ courses almost intact except for spalling of part of outer shell on west. Tile generally cracked through shells parallel with webs, but some were also cracked through webs parallel with shells. Filling good, no voids. Tile fused near bottom on south and west sides, some pieces had been nearly plastic. Partial fusion extended up to the 10th course. (Figs. 69, 120, and 163.)

TEST NO. 52. PLATE AND CHANNEL. 2-IN. OHIO SHALE TILE. CINDER-CONCRETE FILL.—2 mins.—cracks $\frac{1}{8}$ in. wide on east and west. 8 to 11 mins.—cracks increasing and open to $\frac{1}{4}$ in., some shells buckling, two outer shells spalled, and considerable number of outer shells loose. 16 mins.—cracks open to 1 in. maximum; tie broken on 7th course. 18 mins.—outer shell 7th course on west spalled off. 27 mins.—tie broken on 9th course. 32 mins.—outer shell 9th course on south spalled; a number of loose outer shells held in place by ties. 53 mins.—all tile cracked, but little further spalling. 1 hr. 6 mins.—edge of southeast flange exposed opposite 6th course. 1 hr. 25 mins.—inner shell ready to fall on 7th course, west, edges of flanges probably exposed. 1 hr. 32 mins.—maximum expansion. 1 hr. 40 $\frac{3}{4}$ mins.—failure with local buckling about 6 ft. above base.

After failure.—Concrete fill without voids; mortar joints on north and south sides fairly full. Bracket courses almost intact. (Figs. 70, 121, and 163.)

TEST NO. 53. PLATE AND CHANNEL. 4-IN. OHIO SHALE TILE. CINDER-CONCRETE FILL.—2 to 15 mins.—cracking very pronounced on all faces, cracks opening up to 1 in. maximum at end of this period. 7 to 30 mins.—spalling of outer shells beginning at corners and later across faces; outer shells generally loose. 8 to 23 mins.—bulging out of tile at horizontal joints on east and west, 3 to 9 ft. up, increasing to $1\frac{1}{2}$ in. at end of period. 24 mins.—ties off on 3d, 7th, and 9th courses. 30 to 33 mins.—all outer shells spalled off on west, 2 to 6 ft. up on east, 9 to 10 ft. up. 33 to 37 mins.—tile fell on west, 4th and 5th courses, on east 7th, 8th, and 9th courses. 42 mins.—all ties down except on 1st, 8th, 11th, and 12th courses. 45 mins.—tile bulged out on south, exposing steel from 2 to 5 ft. up. 58 $\frac{1}{2}$ mins.—tile now fallen on west from 3d to 9th

courses, inclusive. 1 hr. 4 mins.—bracket courses nearly intact. 1 hr. 17 mins.—maximum expansion. 1 hr. 22¼ mins.—failure with local buckling about 7½ ft. above base.

After failure.—Cracks through tile webs parallel with faces predominate, although many units cracked through both faces. Mortar joints generally full. Concrete fill good, no voids. (Figs. 70, 121, and 163.)

TEST No. 54. LATTICED CHANNEL. 2-IN. OHIO SEMIFIRE-CLAY TILE. TRAP CONCRETE FILL.—2 to 15 mins.—considerable cracking, both in joints and in tile, cracks opening up generally to ⅜ in. at end of period; outer shells spalling near corners and beginning to bulge out at horizontal joints, mostly in middle courses. 24 mins.—bulging out of outer shells increased to maximum of 2 in. 28 mins.—tie broken on 9th course. 48 mins.—nearly all of outer shells spalled off at bracket on south. 50 mins. to 1 hr. 4 mins.—outer shell of tile spalled on south, 4 to 5 ft. up, north half of east face, 3 to 5 ft. up, and on west 7 to 9 ft. up. 1 hr. 22 mins.—tie broken on 4th course. 1 hr. 41½ mins.—inner shells on east fell 4 to 6 ft. up, exposing steel. 1 hr. 43 mins.—inner shells on north fell 4 to 6 ft. up, exposing steel; outer shell spalled on north 3 to 4 ft. up. 1 hr. 52 mins.—tile fell on west 2 to 4 ft. up, exposing flange edges; on north, 3 to 6 ft. up, exposing part of flanges and lattice. 2 hrs. 30 mins.—all tile off on west, 9th and 10th courses. 2 hrs.—slight deflection to northwest noted, increasing to ⅜ in. at 2 hrs. 41 mins. and to 1 in. west at 3 hrs. 1 min. 2 hrs. 20 mins.—maximum expansion. 3 hrs. 17¼ mins.—failure, buckling to west, maximum 6 ft. above base.

After failure.—Concrete between flanges of channels quite crumbly; that between channel webs hard and apparently little injured except where crushed. Concrete fairly fills space between steel and tile on both flange and web sides. (Figs. 70, 122, and 164.)

TEST No. 55. Z BAR AND PLATE. 2-IN. OHIO SEMIFIRE-CLAY TILE. LIMESTONE CONCRETE FILL.—4 mins.—cracking and bulging out of outer shells. 13 to 23 mins.—outer shells generally cracked to ⅜ in. maximum; bulging out at horizontal joints increasing; some spalling at corners. 34 mins.—parts of shell on west spalled at 4 to 5 ft. and 2 to 3 ft. up. 45 mins.—all cracks opening. 1 hr. 4 mins.—wire tie broken on 3d course. 1 hr. 48 mins.—tile fell on west, 7th to 10th courses, inclusive, also part south. 2 hrs.—outer shell spalled on south at bracket course exposing edge of bracket steel. 2 hrs. 8 mins.—outer shell spalled on north, 7th to 9th course. 2 hrs. 37 mins.—tile bulged out 1¼ in. on south 9 ft. up. 2 hrs. 51 mins.—maximum expansion. 3 hrs. 20 mins.—tile fell on west 11th course. 3 hrs. 23 mins. to 3 hrs. 27 mins.—outer shells spalled on north at bracket and from 2 to 6 ft. up; on east, shells now spalled from 2 to 7 ft. up. 3 hrs. 41 mins.—deflection of ⅜ in. south. 3 hrs. 46¼ mins.—failure, buckling to south, maximum at 8 ft. above base.

After failure.—Tile shells remaining on are firmly held. Concrete filled interior fully, including space between Z bar flanges and tile. Concrete hard where tile remained in place; where exposed concrete was calcined to depth of about ½ in. (Figs. 71, 122, and 164.)

TEST No. 56. Z BAR AND PLATE. 4-IN. OHIO SEMIFIRE-CLAY TILE. LIMESTONE CONCRETE FILL.—5 to 30 mins.—considerable cracking and some spalling of parts of outer shells; spalling principally in upper half near southwest corner, at middle near northwest corner, and on lower courses at northeast corner. 57 mins.—outer shell bulged slightly at some horizontal joints, most on west at 6 and 7 ft. up; no inner shells spalled. 2 hrs. 30 mins.—maximum expansion. 2 hrs. 55 mins.—outer shell spalled on south at bracket; otherwise little cracking or spalling in last 2½ hrs.; wire mesh in horizontal joints holding tile in place quite effectively. 1 hr. 51 mins.—

deflection of $\frac{1}{8}$ in. south; deflection changed to $\frac{1}{8}$ in. northeast at 3 hrs. 1 min., increasing to $\frac{3}{8}$ in. at 3 hrs. 21 mins. 3 hrs. 33½ mins.—failure, buckling to north, maximum at 8½ ft. above base.

After failure.—Wire mesh in all horizontal joints lapping at corners. Concrete fill good, no voids; concrete hard, almost intact. (Figs. 71, 123, and 165.)

TEST NO. 57. I BEAM AND CHANNEL. 4-IN. SURFACE-CLAY TILE, CHICAGO DISTRICT. LIMESTONE-CONCRETE FILL.—3 to 13 mins.—general cracking and bulging of a few outer shells, cracks open to $\frac{1}{8}$ in. 23 mins.—bulging increasing; general spalling of small pieces from corners. 25 mins.—parts of outer shell spalled on east and west 4 to 5 ft. up. 32 mins.—cracks and bulges generally open $\frac{1}{2}$ in. on all sides from 3 to 6 ft. up. 38 mins.—outer shells loose or fallen on west 5 to 7 ft. up and on south 4 to 6 ft. up. 53 mins.—all outer shells spalled on east and south 3 to 6 ft. up, and on north 5 to 6 ft. up; tile in 5th course on south fell, exposing part of flange. 1 hr. 2 mins.—large amount of tile fell on all sides, exposing most of concrete from 3 to 10 ft. up. 2 hrs. 15 mins.—maximum expansion. 2 hrs. 37 mins.—tile down on all sides from 2 to 12 ft. up, exposing concrete. 2 hrs.—slight deflection to north noted, increasing to $\frac{1}{4}$ in. at 2 hrs. 50 mins., and to $\frac{1}{2}$ in. at 3 hrs. 21 mins. 3 hrs. 23 mins.—failure, buckling to north, maximum at 8 ft. above base.

After failure.—A number of tile sheared at plaster key, leaving latter in the concrete. Concrete fill fairly full, except at a few points between tile and channel flange. Concrete calcined to depth of $\frac{1}{2}$ in. where exposed; hard where protected. (Figs. 71 and 123).

TEST NO. 58. I BEAM AND CHANNEL. 2 LAYERS OF 2-IN. SURFACE-CLAY TILE, CHICAGO DISTRICT. TILE FILL. WIRE MESH IN HORIZONTAL JOINTS.—3 to 15 mins.—outer shells and tile spalled at corners quite generally. 16 mins.—outer shell of outer tile bulged out $1\frac{1}{2}$ in. at horizontal joint on east 5 ft. up. 26 mins.—part of inner shell of outer layer of tile at southeast corner 10 ft. up fell; outer shells continuing to spall at corners and on parts of some faces. 44 to 48 mins.—outer shells of outer tile spalled on all sides at bracket, also partly on east at 10 ft. up, and all on west 10 to 11 ft. up; outer shell bulged 1 in. at 10½ ft. up on south. 1 hr. 36 mins.—all outer shells now spalled on east from 9 ft. to top. 1 hr. 57 mins.—outer tile down on south, 10th course, and on east, 9th course. 3 hrs.—little change in 1 hr. 3 hrs. 30 mins.—outer shell of outer tile spalled on south, 5th course. 3 hrs. 43 mins.—bracket steel exposed for 6 in. on northeast. 3 hrs. 45 mins.—all tile down in upper two courses on east; on north and south, outer tile down in upper two courses and inner tile bulging out 1 in.; all of bracket exposed on both sides. 3 hrs. 56 mins.—50 to 75 per cent of outer shells of outer tile now down, most of which occurred before 1 hr. 57 mins.; total outer tile and part of inner tile fallen in a few places as noted above; little spalling now taking place. 4 hrs.—maximum expansion. 4 hrs. 25 mins.—no decided change; no fusion noted, although tile at 4 ft. up concave outward as if beginning to fuse. 3 hrs.—slight center deflection to south noted, increasing to $\frac{1}{4}$ in. at 4 hrs. 31 mins. 4 hrs. 35¾ mins.—failure, with local buckling to south and west about 11 ft. above base.

After failure.—Generally all but outer shell of outer tile in place in lower eight courses. On 9th course outer tile down and almost all tile down in 10th, 11th, and 12th courses. Wire mesh found in all joints except between the 1st and 2d courses, between the 9th and 10th on north and south where the pipes interfered, and between the 11th and 12th where the bracket angles interfered. Some mesh was found between the 10th and 11th courses, although so much tile was down that it was impossible to determine whether all pieces had been placed. Very decided fusion of tile occurred in lower 4 ft. and incipient fusion up to 7 ft. above base, being most pronounced on south side of column. (Figs. 71, 124, and 165.)

TEST NO. 59. I BEAM AND CHANNEL. 2 LAYERS OF 2-IN. SURFACE-CLAY TILE, CHICAGO DISTRICT. TILE FILL. OUTSIDE WIRE TIES.—3 to 14 mins.—considerable cracking of outer shells on all sides. 14 mins.—outer tile bulged out $1\frac{1}{2}$ in. on west, 7 ft. up. 15 to 23 mins.—spalling of two outer shells at corners. 17 mins.—outer tile bulged out $\frac{1}{2}$ in. on north, 4 ft. up, increasing to 2 in. at 27 mins. 26 mins.—nearly all of outer shells of outer tile spalled on west, 6 to 8 ft. up; tie broken on 8th course. 29 mins.—outer shell spalled on east, 7th and 8th courses. 33 mins.—all outer tile 8th and 9th courses down and part of tile in 7th course. 35 mins.—inner tile bulged out $2\frac{1}{2}$ in. on west, 7 ft. up. 36 mins.—outer tile loose on west in 11th and 12th courses, held by wire. 38 mins.—all outer tile down on south from 3 to 6 ft. up, on north from 3 to 8 ft. up. 42 mins.—inner tile on south bulged out $\frac{3}{4}$ in., 5 ft. up. 1 hr. 6 mins.—inner tile down on south, 6 to 9 ft. up, exposing both flanges. 1 hr. 16 mins.—all outer tile and parts of inner tile down except for parts of upper and lower courses. 1 hr. 23 mins.—all tile down on west, 6 to 8 ft. up, exposing channel. 1 hr. 25 mins.—maximum expansion. 1 hr.—slight center deflection to north noted, increasing to $\frac{3}{8}$ in. at 1 hr. 31 mins. 1 hr. 33 $\frac{3}{4}$ mins.—failure with buckling to north, maximum at $7\frac{1}{2}$ ft. above base, and twisting about 22° .

After failure.—All but filling tile down in middle six courses. Filling tile also thrown off near middle of column. All tie wires broken before or at failure. (Figs. 72 and 124.)

TEST NO. 60. LATTICED ANGLE. 2-IN. OHIO SEMIFIRE-CLAY TILE. TRAP-CONCRETE FILL, PLACED BEFORE TILE WAS SET.—3 to 18 mins.—cracking of tile on all sides; vertical crack on north near west corner open to $\frac{3}{4}$ in., 3 to 6 ft. up. 13 to 18 mins.—ties broke on 9th, 6th, 8th, 4th, and 3d courses, in the given order. 18 $\frac{1}{2}$ mins.—part outer shells spalled on south 3 to 6 ft. up. 26 mins.—tile down on west from 1 to 7 ft. up, exposing concrete. 32 mins.—tile bulged out 2 in. on north 2 to 6 ft. up. 33 mins.—tile down on north from 1 to 6 ft. up. 36 mins.—tile bulged out $1\frac{1}{2}$ in. on east, 2 to 7 ft. up. 38 mins.—a few cracks in tile in upper three courses. 40 mins.—all tile down on east 1 to $10\frac{1}{2}$ ft. up. 48 mins.—tile down on north, $7\frac{1}{4}$ to $10\frac{1}{2}$ ft. up. 1 hr. 2 mins.—tile down on south, 7th course. 1 hr. 47 mins.—maximum expansion. 1 hr. 58 mins.—little change during last hour. 2 hrs. 7 mins.—all tile down on south up to bracket. 2 hrs. 13 mins.— $\frac{1}{8}$ -in. vertical cracks in concrete halfway up increasing to $\frac{1}{2}$ in. at 2 hrs. 24 mins. 2 hrs. 21 mins.—center deflection of $\frac{1}{8}$ in. north-west, increasing to $\frac{3}{4}$ in. at 3 hrs. 7 mins. 3 hrs. 9 $\frac{1}{2}$ mins.—failure with local buckling $4\frac{1}{2}$ to 5 ft. above base.

After failure.—Tile quite generally cracked vertically through both shells, transverse cracks along webs not general. Mortar evidently did not bond tile and concrete. Except at point of failure, cracks in concrete are not wider than $\frac{1}{8}$ in. Concrete very crumbly outside lattice, quite hard inside. (Figs. 72, 125, and 166.)

TEST NO. 61. LATTICED ANGLE. 2-IN. OHIO SEMIFIRE-CLAY TILE. NO FILLING.—3 to 18 mins.—pronounced cracking and bulging especially at middle courses; cracks mostly vertical, opening to $1\frac{1}{4}$ in. maximum on south, 5 to 7 ft. up; very little spalling. 13 mins.—ties broken on 4th, 6th, and 7th courses. 26 mins.—both outer and inner shells fell on northwest corner 4 ft. up for width of about 4 in. 32 mins.—similar spall on southeast corner 4 ft. up. 35 mins.—considerable bulging of tile on south, 3 to 4 ft. up. 44 mins.—cracks and bulging increased slightly, none now more than $\frac{1}{4}$ in. 45 mins.—maximum expansion. 47 mins.—center deflection of $\frac{1}{4}$ in. east. 50 $\frac{1}{4}$ mins.—failure with local buckling about 6 ft. above base.

After failure.—Tile cracked transversely in some cases; greater number cracked vertically through both faces without transverse web cracks. (Figs. 73 and 125.)

TEST NO. 62. ROUND CAST IRON. 2-IN. POROUS SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT; NO FILLING.—Air pressure tanks shut off, load applied by water pressure only. 6 to 20 mins.—a number of vertical cracks in tile and in mortar joints, opening to maximum of $\frac{1}{4}$ in. at end of this period. 45 mins.—cracks now in 40 to 50 per cent of tile units, width varying from fine to $\frac{3}{8}$ in. in column proper and to $\frac{1}{2}$ in. at bracket. 2 hrs.—all ties in place; cracks have opened up slightly. 2 hrs. 25 mins.—horizontal cracks on west between bracket courses about $\frac{5}{16}$ in. wide. 2 hrs. 42 mins.—similar cracks on southwest, $\frac{1}{2}$ in. wide. 3 hrs.—maximum expansion. 3 hrs. 26 mins.—horizontal bracket cracks open to 1 in. maximum. 4 hrs. 10 mins.—cracks open 1 in. at middle of column; no spalling or bulging of tile. 4 hrs.—slight deflection to southeast noted, increasing to $1\frac{3}{8}$ in. at 4 hrs. 10 mins. 4 hrs. 11½ mins.—failure, column unable to support working load. 4 hrs. 13 mins.—load of about 75 000 lbs. held. 4 hrs. 14 mins.—tile fell on east; load of about 25 000 lbs. held. 4 hrs. 14¾ mins.—gas shut off; column buckled to southeast maximum at 8 ft. above base.

After failure.—Column did not crack although metal buckled near surface on compression side at failure point. Cracks in tile are generally vertical, extending through both shells. Tie wires were greatly oxidized and had little strength. (Figs. 73, 126, and 167.)

TEST NO. 63. ROUND CAST IRON. 2-IN. POROUS SEMIFIRE-CLAY TILE, NEW JERSEY DISTRICT; NO FILLING.—6 mins.—vertical crack on east, 4 to 5 ft. up, $\frac{1}{2}$ in. wide. 8 to 11½ mins.—gas shut off accidentally during this period. 13½ to 23½ mins.—fine vertical cracks 2 to 6 ft. long opening up to $\frac{1}{4}$ in. maximum. 32 mins.—outer shell loose on 3d course, east side. 35 mins.—cracks open to $\frac{1}{2}$ in. 47½ mins.—cracks open to $\frac{7}{8}$ in. maximum. 1 hr. 25 mins.—cracks opened but slightly during past 30 mins. 1 hr. 37 mins.—some flaking of outer shells; no spalling. 1 hr. 55 mins.—very little change; tile above 10 ft. up almost intact except for a few cracks $\frac{1}{8}$ in. wide. 2 hrs. 20 mins.—maximum expansion. 2 hrs. 23 mins.—about $\frac{1}{3}$ of outer shell spalled on 3d course, east side. 2 hrs. 24 mins.—bulging on southeast, 4 to 6 ft. up, with spalling of parts of both shells leaving crack $\frac{3}{4}$ to 2 in. wide. 2 hrs. 41 mins.—slight center deflection to north, increasing to $1\frac{1}{4}$ in. at 2 hrs. 55 mins. 2 hrs. 57½ mins.—failure with buckling to north, maximum at $5\frac{1}{2}$ ft. above base.

After failure.—Most cracks in tile were vertical, straight through both shells. Tie wires greatly oxidized, leaving $\frac{1}{8}$ in. effective diameter. At failure point $5\frac{1}{2}$ ft. up, cast iron was mushroomed out about 1 in. for one-half the circumference in a height of $2\frac{1}{2}$ in. Numerous vertical cracks in the iron formed in this region. Tension breaks extend for one-half of the circumference on south, $3\frac{1}{2}$ ft. above the base and $1\frac{3}{4}$ ft. below the head. The thickness of metal at the center break varied from $\frac{1}{2}$ to $1\frac{1}{4}$ in., the thinnest metal being on the south. (Figs. 73, 126, and 167.)

TEST NO. 76. ROLLED H. 2-IN. HOLLOW CLAY TILE COVERED WITH $\frac{3}{4}$ -IN. LAYER OF GYPSUM PLASTER; LIMESTONE CONCRETE FILL. UPPER 4 COURSES, OHIO SHALE; MIDDLE 4 COURSES, OHIO SEMIFIRE CLAY; LOWER 4 COURSES, SEMIFIRE CLAY, NEW JERSEY DISTRICT.—1 to 3 mins.—outer coat of plaster peeled off and shattered at end of this period; vertical cracks extending through to tile in places. 10½ mins.—inner coat of plaster fell on south, exposing tile 3 to 8 ft. up. 11 to 32 mins.—inner coat of plaster fell, exposing about two-thirds of tile at 21 mins.; at 32 mins. about 90 per cent of plaster had fallen below bracket course; plaster still on bracket on all four sides and down to 9 ft. up on west. 28 mins. to 1 hr. 10 mins.—some spalling of outer shells and very slight amount of cracking, mostly at corners; at end of this period northeast corner was spalled 5 to 6 ft. up, also on west 4 to 5 ft. up, and on east 6 to 7 ft. up; shell loose on north 7 to 8 ft. up. 1 hr. 32 mins.—several corners spalled in upper

two thirds of column. 1 hr. 41 mins.—outer shell spalled on north 8th course; plaster on at bracket, except on north side. 2 hrs. 30 mins.—little change during past 50 mins., except for a few spalls of outer shells in upper half. 3 hrs. 30 mins.—no change except outer shell spalled on west, 9th course. 3 hrs. 45 mins.—maximum expansion. 4 hrs.—little change, except for a few vertical cracks, $\frac{1}{8}$ to $\frac{1}{4}$ in., near the top; plaster still on parts of bracket. 4 hrs. 18 mins.—outer shell spalled on east, 9th course; no inner shells spalled before failure; no spalling of outer shells on four lower courses; deflections not measured. 4 hrs. 25½ mins.—failure with local buckling about 9½ ft. above base.

After failure.—Shale tile almost all off; could not be examined; Ohio semifire-clay tile quite shattered, no bond with mortar or fill; semifire clay, New Jersey district, tile in fairly good condition except for a number of fine cracks; fair bond with fill. Concrete fill full, no voids. Mortar joints on sides were fairly full. (Figs. 78, 138, and 46.)

TEST NO. 77. PLATE AND ANGLE. 4-IN. HOLLOW CLAY TILE COVERED WITH $\frac{5}{8}$ IN. LAYER OF LIME PLASTER; LIMESTONE-CONCRETE FILL. UPPER 4 COURSES, SEMI-FIRE CLAY, NEW JERSEY DISTRICT; MIDDLE 4 COURSES, SURFACE CLAY, CHICAGO DISTRICT; LOWER 4 COURSES, SURFACE CLAY, BOSTON DISTRICT.—½ min.—most of plaster spalled off, exposing nearly three-fourths of total tile surface. 15 mins.—practically all of plaster down, except small amount in places near top; some outer shells of tile beginning to buckle in middle courses. 20 to 50 mins.—cracking and some spalling of outer shells, mostly at corners, in middle 4 courses; a few cracks in lower 4 courses, less than $\frac{1}{8}$ in. wide. 1 hr. 10 mins.—very little change during 20 mins., though cracks opened slightly; no cracks observed in upper 4 courses. 1 hr. 35 mins.—a few fine cracks in upper 4 courses, not over $\frac{1}{4}$ in. 1 hr. 49 mins.—a number of horizontal cracks in lower 4 courses up to $\frac{1}{8}$ in. wide; outer shell spalled on west 3 to 4 ft. up. 2 hrs. 20 mins.—horizontal and vertical cracks in lower 4 courses opening up $\frac{1}{8}$ to $\frac{3}{8}$ in.; cracks in upper 4 courses not opening up. 3 hrs. 22 mins.—a few tile in lower courses curved out, splitting near ends; not much change in middle 4 courses. 4 hrs.—cracks in lower 4 courses open $\frac{3}{8}$ to $\frac{1}{2}$ in.; northeast corner spalled 2 to 3 ft. up; possibly fusion 3 ft. up. 4 hrs. 10 mins.—maximum expansion. 4 hrs. 17 mins.—more spalling of outer shells in middle courses; no inner shells spalled. 4 hrs. 20 mins.—tile bulged out from steel just above middle on north and south sides; very decided fusion 1 to 3 ft. up. 4 hrs. 26 mins.—cracks in lower 4 courses open $\frac{1}{8}$ to $\frac{1}{2}$ in.; outer shell 5th to 7th course, north side, spalled; cracks in upper 4 courses open not over $\frac{1}{4}$ in. 4 hrs. 26 mins. to 4 hrs. 30 mins.—considerable spalling of outer shells in middle 4 courses; all inner shells, all courses, in place before failure. Deflections not measured. 4 hrs. 42¼ mins.—failure with buckling to west, maximum at 6 ft. above base.

After failure.—All of upper four courses and middle four courses of tile down, examination not possible. Lower four courses much shattered by vertical and horizontal cracks; incipient fusion through outer shells and through webs where exposed; pieces of this tile bent, curled, and discolored. Concrete fills reentrant portions fully. Appearance of steel indicated that concrete had not fully filled space between flanges and tile. (Figs. 78, 139, and 46.)

(f) GYPSUM BLOCK PROTECTIONS

TEST NO. 64. ROLLED H. 4-IN. WESTERN GYPSUM BLOCK (SOLID). HOLLOW GYPSUM-BLOCK FILL.—10 mins.—fine surface checks developing, about $\frac{1}{32}$ in. wide. 30 mins.—fine surface checks all over faces, about $\frac{1}{2}$ in. apart both ways. 48 mins.—corners spalled not over $\frac{1}{2}$ in. by $\frac{1}{2}$ in. 1 hr.—very little change. 1 hr. 30 mins.—surface checks about $\frac{1}{8}$ in. wide by $\frac{1}{8}$ in. deep; a few vertical joints opened not over

$\frac{1}{8}$ in. 2 hrs. 30 mins.—both horizontal and vertical joints opened about $\frac{1}{8}$ in.; mortar joints project $\frac{1}{4}$ in. 2 hrs. 40 mins.—block fell from bracket on north, exposing west part of bracket steel. 3 hrs. 10 mins. to 3 hrs. 25 mins.—spalling on southwest and northeast corners, just below middle, to 2 in. back. 3 hrs. 35 mins.—surface checks $\frac{1}{8}$ in. wide by $\frac{1}{2}$ in. deep; all cracks open to about $\frac{1}{4}$ in. 3 hrs. 45 mins.—mortar joints throughout project about $\frac{1}{2}$ in. 3 hrs. 55 mins.—piece of block down on southwest side of bracket; blocks on northeast and southeast sides of brackets stand out. 4 hrs. 5 mins.—block down on northeast side of bracket, exposing rest of bracket steel. 4 hrs. 10 mins.—mortar joints project $\frac{3}{4}$ in.; tile appears to be about 3 in. thick. 4 hrs. 15 mins.—block down on southeast side of bracket; both sides of south bracket now exposed; bracket steel glowing dull red. 4 hrs. 20 mins.—blocks on east and west faces tilting out from steel at 7 ft. up; surface checks appear to be about 1 in. deep. 4 hrs. 30 mins.—nearly all of block down on west 6 to 7 ft. up; all fell on east, from 4 ft. up to top of column, on south from 1 to 6 ft. up, exposing flange for 5 ft., also down on north, 10 to 11 ft. up. 4 hrs. 32 mins.—maximum expansion. 4 hrs. 39 mins.—all blocks down on west from 1 ft. up to top. 4 hrs. 30 mins.—slight deflection to northeast noted, increasing to $\frac{3}{8}$ in. at 4 hrs. 41 mins. 4 hrs. 43 $\frac{1}{4}$ mins.—failure with buckling to east, maximum at 5 ft. above base.

After failure.—Exposed column flange on south greatly oxidized during test. Gypsum blocks shrunken to thickness of $3\frac{3}{4}$ to $3\frac{1}{2}$ in. Surface of blocks checked into $\frac{1}{4}$ -in. squares, cracks extending inward to depth of $1\frac{1}{4}$ to 2 in. from surface on sides and to $3\frac{1}{2}$ at corners, the corner cracks extending in diagonally. Gypsum in this zone is brownish green. Further in gypsum is much more crumbly than new material, showing that the heat had affected it. (Figs. 74 and 127.)

TEST NO. 65. PLATE AND CHANNEL. 2-IN. WESTERN GYPSUM BLOCK (SOLID); SOLID GYPSUM-BLOCK FILL.—11 mins.—fine surface checks beginning to develop. 23 mins.—a few fine cracks and slight corner spalls; surface checks more distinct, forming in $\frac{1}{2}$ in. squares. 29 mins.—a few vertical joints open $\frac{1}{8}$ in. 48 mins.—surface checks $\frac{1}{8}$ in. wide by $\frac{1}{4}$ in. deep, distinct throughout courses; cracks $\frac{1}{2}$ in. deep; some corners spalled $\frac{1}{2}$ in. back, corners generally ragged. 1 hr. 5 mins.—cracks in all joints about $\frac{1}{8}$ in. wide; surface checks $\frac{1}{8}$ in. wide. 1 hr. 30 mins.—shrinking of tile apparent, mortar joints project $\frac{1}{8}$ to $\frac{1}{2}$ in.; cracks in joints now $\frac{1}{2}$ in. wide. 1 hr. 45 mins.—block fell on west 4 to 5 ft. up, exposing edges of flanges. 1 hr. 49 mins.—block fell on south 5 to 6 ft. up; mortar in joints projects out generally, $\frac{1}{4}$ to $\frac{3}{8}$ in. 2 hrs.—blocks fell on north 2 by 6 ft. up, steel still covered by mortar. 2 hrs. 5 mins.—blocks fell on east 2 to 6 ft. up, on west 2 to 5 ft. up; three top courses still intact. 2 hrs. 9 mins.—blocks down on south and west 5 to 6 ft. up. 2 hrs. 10 mins.—edges of flanges exposed where blocks have fallen, but steel plates are covered with about 1 in. of mortar in all cases. 2 hrs. 12 mins.—block down on west 6 to 7 feet up; piece fallen on northwest corner at bracket. 2 hrs. 17 mins.—mortar in joints projects $\frac{3}{8}$ in. 2 hrs. 20 mins.—maximum expansion; mortar fallen on north exposing steel; blocks down on north and west 7 to 8 ft. up. 2 hrs. 11 mins.—center deflection less than $\frac{1}{8}$ in. 2 hrs. 21 $\frac{1}{2}$ mins.—failure with local buckling about 3 ft. above base.

After failure.—Blocks shrunken to thickness of $1\frac{1}{2}$ to $1\frac{3}{4}$ in. Surfaces of blocks checked into $\frac{1}{4}$ to $\frac{1}{2}$ in. squares, cracks being $\frac{1}{8}$ to $\frac{1}{2}$ in. wide at surface and $\frac{1}{2}$ to $\frac{3}{4}$ in. deep on sides; cracks at corners extending in diagonally 1 to $1\frac{1}{4}$ in. Outer dehydrated surface quite firm and greenish in color in places; hardness extends to inner zone of checking where color changes from white to brownish. Gypsum farther in is softer than material not exposed to fire. Galvanizing on corrugated sheet-metal ties placed in the joints was removed; otherwise they were not oxidized or corroded. (Figs. 74 and 128.)

TEST NO. 66. LATTICED CHANNEL. 2-IN. EASTERN GYPSUM BLOCK (SOLID); POURED GYPSUM FILLING.—15 mins.—fine surface checks beginning to develop. 30 to 40 mins.—surface checks more distinct, forming in $\frac{1}{4}$ to 1 in. squares, mostly in lower half. 1 hr. 15 mins.—joints open to $\frac{1}{16}$ in. increasing to $\frac{1}{8}$ in. at 1 hr. 35 mins. 1 hr. 35 mins.—mortar in joints project $\frac{1}{8}$ in. 2 hrs.—surface checks now cover whole surface, and are $\frac{1}{4}$ in. to $\frac{1}{2}$ in. in depth; joints open to $\frac{3}{8}$ in. maximum. 2 hrs. 5 mins.—block on east $5\frac{1}{2}$ ft. up, tilting out $\frac{3}{4}$ in. at top. 2 hrs. 14 mins.—small spall on southwest corner at bracket, block below bracket on west tilting out at top. 2 hrs. 15 mins.—block down on east in 4th course, exposing lattice bars where unprotected by filling. 2 hrs. 22 mins.—block fell on east 8th course, exposing steel. 2 hrs. 26 mins.—blocks fell on north $1\frac{1}{2}$ to $5\frac{1}{2}$ ft. up, exposing steel. 2 hrs. 30 mins.—block fell on south, 4th course up, exposing steel. 2 hrs. 32 mins.—maximum expansion; blocks fell on south and west 2d course up. 2 hrs. 33 mins.—center deflection less than $\frac{1}{8}$ in. 2 hrs. 36 mins.—failure with local buckling about $3\frac{1}{2}$ ft. above base, causing column to deflect to west.

After failure.—Filling fairly full, but has some pockets, 2 in. deep; filling extends out between lattice bars, but not much beyond; very crumbly where exposed. Blocks shrunk to thickness of $1\frac{1}{8}$ ins.; little change in color. Surface checks are $\frac{1}{8}$ in. wide at surface and $\frac{1}{2}$ in. to $\frac{3}{4}$ in. deep. Strips of wire mesh in joints not greatly oxidized. (Figs. 74 and 128.)

TEST NO. 67. ROLLED H. 4-IN. EASTERN GYPSUM BLOCK (SOLID); POURED GYPSUM FILLING.—17 mins.—fine surface checks, forming in 1-in. squares beginning to show. 34 mins.—surface checks more marked, $\frac{3}{8}$ to $\frac{1}{8}$ in. wide; checking most pronounced in lower half. 1 hr. 5 mins.—surface checks $\frac{1}{4}$ in. deep and $\frac{1}{8}$ to $\frac{1}{4}$ in. wide; corner cracks $\frac{1}{2}$ to 1 in. deep and 1 to 2 in. long. 1 hr. 20 mins.—joints beginning to open up; several horizontal cracks 1 in. deep across faces; surface checks now forming in $\frac{1}{2}$ to $\frac{3}{4}$ in. squares. 1 hr. 50 mins.—very little change; surface checks $\frac{1}{4}$ to $\frac{1}{2}$ in. deep are general. 2 hrs. 10 mins.—a few mortar joints project $\frac{1}{8}$ to $\frac{1}{16}$ in. 2 hrs. 21 mins.—pieces of corners at bracket spalled. 2 hrs. 40 mins.—joints open to $\frac{1}{8}$ in.; surface checks $\frac{3}{4}$ in. deep and $\frac{1}{4}$ in. wide. 3 hrs. 45 mins.—mortar in joints projects $\frac{1}{4}$ in., vertical joints open to $\frac{1}{4}$ in., little change in surface checks. 4 hrs. 40 mins.—mortar projects to $\frac{1}{2}$ in.; otherwise little change. 4 hrs. 52 mins.—top of block tilts out 1 in. on north $8\frac{1}{2}$ ft. up. 4 hrs. 54 mins.—blocks fell on south 3d and 4th courses; mortar still covers steel quite generally; piece of block 4 in. wide also down at southwest corner in 5th course; joints near top open $\frac{1}{8}$ in. 5 hrs. 1 min.—maximum expansion. 5 hrs. 5 mins.—blocks next to those fallen tilt away from steel; steel now exposed on south 3 to 4 ft. up. 5 hrs. 7 mins.—blocks down on west 3d to 6th course, exposing filling and flange edges. 5 hrs. 13 mins.—block fell on west 2d course; west edge of flange exposed 4 to 6 ft. up; little change after this until failure. 5 hrs. $31\frac{1}{2}$ mins.—failure with local buckling about 4 ft. above base.

After failure.—Fill fairly full, but has some pockets 2 in. deep; mortar where still in place, covers flanges $\frac{3}{4}$ to 1 in.; steel fluxed for 30 in.-length near point of failure; edges of flanges attacked where exposed, unaffected where not exposed; strips of wire mesh oxidized through where they had been exposed to the fire; blocks shrunk to thickness of $3\frac{3}{8}$ to $3\frac{7}{8}$ ins. Surface check cracks formed in $\frac{1}{4}$ to $\frac{1}{2}$ in. squares, cracks $\frac{1}{8}$ to $\frac{1}{16}$ in. wide at surface and generally 2 in. deep, although some extend completely through blocks; radial cracks at corners $2\frac{1}{4}$ in. deep. Block colored brownish yellow in outer 1 in., further in stained dark from oxidation of wood fiber. (Figs. 75 and 129.)

TEST NO. 67 A. ROLLED H. 4-IN. EASTERN GYPSUM BLOCK (SOLID); POURED GYPSUM FILLING.—15 mins.—fine surface checks beginning to show; fine cracks on ends of blocks. 45 mins.—surface checks forming in $\frac{1}{4}$ to 1 in. squares, most pronounced on west. 1 hr. 30 mins.—vertical joints opening slightly. 2 hrs.—cracks in both vertical

and horizontal joints opening up perceptibly; surface checks $\frac{1}{4}$ in. deep. 3 hrs.—vertical joints near middle of column open to $\frac{1}{4}$ in. 3 hrs. 30 mins.—all joints open from $\frac{1}{8}$ to $\frac{1}{4}$ in.; mortar joints project $\frac{1}{4}$ in.; minor spalling from corners. 4 hrs.—all blocks have shrunk considerably; joints open $\frac{1}{8}$ to $\frac{1}{4}$ in. 4 hrs. 30 mins.—small pieces spalling from corners; larger piece fallen from northwest corner, 7 ft. up. 5 hrs. 15 mins.—edge of bracket steel exposed on south side. 5 hrs. 15 mins. to 5 hrs. 45 mins.—considerable spalling at corners. 5 hrs. 45 mins.—maximum expansion. 5 hrs. 56 mins.—blocks down on north 2d and 3d courses, and on west 2d course; steel exposed. 6 hrs. 14 mins.—blocks now down on north from bottom to $5\frac{1}{2}$ ft. up. 6 hrs. 20 mins.—blocks down on east 2d course and on south 3d course. 6 hrs. 21 mins.—block down on west 7th course. 6 hrs. 10 mins.—slight deflection to southwest increasing to $\frac{1}{4}$ in. at 6 hrs. 16 mins. 6 hrs. $24\frac{1}{2}$ mins.—failure with buckling to south, maximum at $4\frac{1}{4}$ ft. above base.

After failure.—Filling fairly full. Column flange on north greatly oxidized for 3 ft. near point of failure. Blocks shrunk to thickness of $3\frac{1}{2}$ to $3\frac{3}{8}$ in.; gypsum very crumbly. Surface checks $\frac{1}{4}$ in. wide, extend in to depth of $2\frac{1}{2}$ in. (Figs. 75 and 130.)

(9) BRICK PROTECTIONS

TEST No. 68. ROLLED H. $2\frac{1}{4}$ -IN. CHICAGO COMMON BRICK LAID ON EDGE; BRICK FILL.—13 mins.—a number of $\frac{1}{8}$ -in. cracks in joints. 15 mins.—some cracking and spalling at corners. 16 to 18 mins.—several vertical cracks developing in middle sections, one on north face $\frac{3}{8}$ in. wide, 2 ft. long. 23 mins.—brick fell on east from $3\frac{1}{2}$ to $8\frac{1}{2}$ ft. up, exposing north flange to depths of from 1 to 3 in. back from edge. 26 mins.—brick fell on west, 4 to $8\frac{1}{2}$ ft. up, exposing south flange for width of 1 in., bricks bulged out $\frac{1}{4}$ in. from flanges on north and south in this region, increasing to $\frac{1}{2}$ in. at 36 mins. 36 mins.—brick in upper third cracked somewhat; brick almost intact up to $3\frac{1}{2}$ ft. above base. 1 hr. 4 mins.—very little change, bulging in middle slightly increased. 1 hr. 10 mins.—maximum expansion. 1 hr. 20 mins.—slight deflection to southeast noted, increasing to $\frac{1}{4}$ in. at 1 hr. 37 mins. 1 hr. $40\frac{3}{4}$ mins.—failure with local buckling $6\frac{1}{2}$ to 8 ft. above base.

After failure.—Brick soft and crumbly and cracks readily. Mortar joints apparently quite full. (Figs. 76, 118, and 166.)

TEST No. 69. ROLLED H. $3\frac{3}{4}$ -IN. CHICAGO COMMON BRICK LAID FLAT. BRICK FILL.—

First Test.—Trouble with one gas burner developed soon after start of test and gas was shut off at $37\frac{1}{4}$ mins. Test postponed for two days. Column observed after test was discontinued. One-half to two-thirds of brick cracked through vertically in one or more places, cracks from very fine to $\frac{1}{8}$ in. wide. Slight flaking and spalling of corners noted. Temperature of steel at end of test 45° C, attaining a maximum of 140° C, 3 hrs. 20 mins. later.

Second Test.—1 hr. 30 mins.—slight flaking, no spalling; cracks developed in first test not opening up. 3 hrs. 30 mins.—no cracks over $\frac{1}{8}$ in., brick flaking off at corners. 3 hrs. 47 mins.—cracks are vertical and generally very fine, maximum $\frac{1}{8}$ in. and not more than 2 ft. in length. 4 hrs. 16 mins.—very little change, cracks open to $\frac{1}{8}$ in. maximum; no fusion. 4 hrs. 52 mins.—decided fusion in lower 3 ft. only. 5 hrs. 20 mins.—maximum expansion. 5 hrs. 38 mins.—fusion extended up to 6 ft., fused brick run down to base at corners. 6 hrs. 10 mins.—fusion up to 10 ft. above base. 6 hrs. 27 mins.—fusion extends up to 11 ft. above base; no spalling, except surface flaking, occurred before failure. 6 hrs.—slight center deflection to west noted, increasing to $\frac{5}{8}$ in. at 7 hrs. 11 min. 7 hrs. $13\frac{1}{4}$ mins.—failure, with buckling to west, maximum at $5\frac{1}{4}$ ft. above base.

After failure.—Fusion at bottom fluxed away about $\frac{1}{2}$ in. of brick. Brick at bracket had just begun to fuse. (Figs. 76, 131, and 168.)

(h) REINFORCED-CONCRETE COLUMNS

TEST No. 70. SQUARE VERTICALLY REINFORCED; LIMESTONE CONCRETE.—16 mins.—corners of column glowing slightly in lower 3 ft. 34 mins.—column luminous entire length; slight flaking on corners near bottom. 1 hr. 11 mins.—fine vertical crack on east, 2 to 3 ft. up. 1 hr. 28 mins.—similar crack 6 in. long 1 ft. from base. 3 hrs. 2 mins.—similar crack on east, 12 in. long near middle of column. 5 hrs.—maximum expansion. 6 hrs. 39 mins.—fine vertical cracks with ends running horizontally to corners appeared on various faces. 7 hrs. 50 mins.—cracks on east opening slightly; also a few additional fine cracks. 8 hrs.—column still supporting working load with no apparent change; no spalling. 8 hrs. 1 min.—load increased with fire going until failure occurred under 294 000 lbs. about 11 ft. above base, at 8 hrs. 40¼ mins.

After failure.—Concrete near outside calcined, but had hard surface due to partial fusion of the sand in the concrete. (Figs. 76 and 132.)

TEST No. 71. SQUARE VERTICALLY REINFORCED; TRAP CONCRETE.—20 mins.—column glowing entire length. 2 hrs. 40 mins.—maximum expansion. 3 hrs.—fine vertical cracks on east and west near bottom. 3 hrs. 10 mins.—slight flaking at corners near middle. 4 hrs. 10 mins.—the fine cracks on east and west faces extending in length. 4 hrs. 40 mins.—several additional fine cracks on east and west faces, 4 to 24 in. long. Very little change before failure; no spalling. 5 hrs.—slight deflection to north noted, increasing to ¼ in. at 7 hrs. 22 mins. 7 hrs. 22¾ mins.—failure by compression about 5 ft. above base, reinforcing bars buckling outward.

After failure.—Concrete fused to average depth of 1 in. up to 7½ ft. above base; incipient fusion from 7½ to 9 ft. up, no fusion above. Concrete dry and loose in texture at top. Bars apparently straight, except where buckled at point of failure. Concrete cracked at bars at some points, but in general its condition at the corners was about the same as at the middle of the sides. (Figs. 77, 133, and 169.)

TEST No. 72. ROUND VERTICALLY REINFORCED; LIMESTONE CONCRETE.—20 mins.—slight surface flaking. 1 hr.—column glowing entire length. 5 hrs.—maximum expansion. 5 hrs. 26 mins.—no spalling or cracking noted. 7 hrs. 55 mins.—a few cracks noted on east and west faces ⅛ to ⅜ in. wide and 3 to 10 in. long, as follows: On east at 3 and 8 ft. up, on west at 2 and 6 ft. up, and at bracket. 8 hrs.—column still supporting working load with no apparent change; no spalling and but few cracks, as noted; deflection less than ⅛ in. 8 hrs. 2 mins.—load increased with fire going until failure occurred under 250 000 lbs., about 6 ft. above base, at 8 hrs. 4½ mins., reinforcing rods buckling outward.

After failure.—Surface hard immediately after test due to partial fusion of sand. A few days after test concrete flaked off to depth of 1 in. due to calcination of limestone. (Figs. 77, 134, and 170.)

TEST No. 73. ROUND VERTICALLY REINFORCED. TRAP CONCRETE.—19 mins.—piece of concrete about 8 in. wide and ½ in. deep spalled on west 2 ft. above base. No cracking or other spalling noted before failure. Furnace gases very heavy, making observation difficult; deflection not measured. 4 hrs. 10 mins.—maximum expansion. 7 hrs. 57½ mins.—failure at 2 to 4 ft. up, concrete crushing and shearing on inclined planes; reinforcing bars buckled outward.

After failure.—Concrete fused to depth of 1 in. at break, fusion being more decided in lower than in upper half. Little or no concrete had run. A large number of fine vertical and horizontal cracks present more or less over whole surface of column. (Figs. 77 and 135.)

TEST No. 74. HOOPED REINFORCED; LIMESTONE CONCRETE.—40 mins.—no spalling or cracking noted. 53 mins.—two fine cracks 3 in. long, on west, 5½ ft. up, opening to ⅛ in. wide and 10 in. long at 2 hrs. 36 mins. 5 hrs. 50 mins.—maximum expansion.

sion. 6 hrs. 50 mins.—several cracks $\frac{1}{8}$ in. wide and about 8 in. long at $2\frac{1}{2}$ ft. up, and one crack $\frac{1}{8}$ in. wide and 8 in. long on east, $5\frac{1}{2}$ ft. up; no cracks above. 8 hrs.—column still supporting working load with no apparent change; no spalling; deflection less than $\frac{1}{8}$ in. 8 hrs. 5 mins.—load increased with fire going until failure occurred under 243 000 lbs. about 3 ft. above base, at 8 hrs. $6\frac{1}{2}$ mins.

After failure.—Eight breaks in spiral reinforcement occurred near failure point. Vertical bars buckled out 3 in. Concrete spalled outside of spiral 2 to 4 ft. up; otherwise no plane of cleavage at spiral. (Figs. 77, 136, and 171.)

TEST NO. 75. HOOPED REINFORCED. TRAP CONCRETE.—

First Test.—Gas shut off after 30 mins. to repair burner and test postponed till next day; column apparently not affected.

Second Test.—36 mins.—column glowing dull red for full length. 40 mins.—fine vertical crack, 4 in. long, on southeast $5\frac{1}{2}$ ft. up, increasing to 12 in. long at 1 hr. 45 mins. 1 to 2 hrs.—some six or eight fine cracks, 2 to 12 in. long noted in lower half. 2 hrs. 20 mins.—all cracks opening slightly. 3 hrs.—many very fine cracks on all surfaces. 4 hrs. 50 mins.—maximum expansion. 6 hrs. 25 mins.—cracks opened somewhat; observation difficult. 8 hrs.—column still supporting working load with little apparent change; no spalling; deflection at 8 hrs. less than $\frac{1}{8}$ in. 8 hrs. $1\frac{1}{2}$ mins.—load increased with fire going, failure occurring under load of 163 000 lbs., about 3 ft. above base, at 8 hrs. $1\frac{3}{4}$ mins.

After failure.—Concrete fused to a depth of $1\frac{1}{2}$ in. up to 11 ft. above base; concrete fluxed off to depth of 1 in., 7 to 9 ft. above base. No fusion in upper $1\frac{1}{2}$ ft. due to excess of mortar near surface. Failure apparently due to yielding of spiral, although no break in it occurred. Vertical bars buckled out 1 in. at failure point. Concrete outside of spiral shells off readily. (Figs. 77 and 137.)

NOTE.—For tests Nos. 76 and 77, see under paragraph (e) above, hollow clay-tile protections, after test No. 63.

(i) TIMBER COLUMNS

TEST NO. 78. LONG-LEAF PINE WITH CAST-IRON CAP AND PINTLE; PROTECTED BY A 1-IN. LAYER OF PORTLAND-CEMENT PLASTER ON METAL LATH.—5 to 8 mins.—vertical cracks in lower half near northwest and southeast corners open to $\frac{3}{8}$ in. and 3 to 5 ft. long. 9 mins.—vertical crack at northeast corner at bracket $\frac{3}{8}$ in. wide, 12 in. long. 11 to 40 mins.—finish coat bulging out and spalling in lower half on north and south sides. 16 mins.—crack at bracket on west, $\frac{1}{8}$ by 8 in. 20 mins.—crack at pintle, northeast corner, open $\frac{1}{4}$ in. 44 mins.— $\frac{1}{8}$ in. vertical crack near northeast corner $1\frac{1}{2}$ to $6\frac{1}{2}$ ft. up. 52 mins.—lath and supporting channel buckle out $1\frac{1}{2}$ in. on south side at west corner, 4 ft. up. 55 mins.—cracks at bracket on northwest corner open $\frac{1}{8}$ in., on southwest corner $\frac{1}{4}$ in. 57 mins.—spurts of flame issue at southwest corner where lath buckled out. 58 mins.—crack at bracket on northeast corner open $\frac{3}{8}$ in., on southeast open $\frac{1}{8}$ in. 1 hr. 13 mins.—flames issue from cracks at bracket on west; flames free and full at southwest corner, 4 ft. up. 1 hr. 21 mins.—plaster spalled to lath, 8 by 12 in., on west, $2\frac{1}{2}$ ft. up; flames issue at this point. 1 hr. 24 mins.—flames issue from crack at bracket on northeast corner. 1 hr. 57 mins.—head of column going down quite rapidly; column cap apparently level; crack noted in plaster at base of cap. 1 hr. 58 mins.—plaster spalled to lath, 12 by 12 in. on east, 2 ft. up. 2 hrs. 6 mins.—cap still level. 2 hrs. 8 mins.—little change in plaster during past 50 mins., except as noted. 2 hrs. 9 mins.—small piece of plaster spalled on northwest corner, at lower part of bracket. Settlement of top of column due to heating of cap $\frac{1}{8}$ in. at 1 hr. 10 mins., $\frac{1}{4}$ in. at 1 hr. 50 mins., $1\frac{1}{4}$ in. at 2 hrs., and $3\frac{1}{4}$ in. at 2 hrs. 15 mins. (See Fig. 47.) 2 hrs. $15\frac{1}{4}$ min.—failure, due to cracking of cap. 2 hrs. 17 mins.—water applied to column extinguishing flames at 2 hrs. 21 mins.

After failure.—Cap broken into three pieces by two transverse cracks across middle portion. A 2 by 2 by $\frac{1}{2}$ in. chip broke off bottom edge of pintle on north. Beam ends charred to depth of 1 in. on sides and ends; inner surface surrounding pintle scarcely charred. Sides of column charred to depth of $1\frac{1}{4}$ in. in lower two-thirds and to about 1 in. toward top, minimum section of unburned wood, about 8 by 8 in.; vertical cracks on sides extend inward not over $\frac{1}{2}$ in. Top bearing surface of column not charred except for $\frac{1}{4}$ in. at edges, but was crushed and frayed, the fibers being bent out over 3 in. beyond sound wood at the sides, and pushed up 2 to 3 in. into cracks in cap. A piece was sawed from southwest corner of top and fibers were found to have been crushed and turned over for a depth of $1\frac{3}{4}$ in. below surface. Wood crushed down on northwest corner 2 in. more than on south side of bearing surface; due to pintle bearing on this side and transmitting the blow from ram at failure. Length of column below cap before test, 11 ft. $1\frac{3}{4}$ in.; average after test, 10 ft. 9 in.; decrease in length, $4\frac{3}{4}$ in. (Figs. 79, 140, and 47.)

TEST NO. 79. LONG-LEAF PINE WITH CAST-IRON CAP AND PINTLE; UNPROTECTED.—2 mins.—surface of column blazing in lower 3 ft. 3 mins.—fine horizontal checks due to charring appeared. 9 mins.—column flaming all over. 11 mins.—three or four vertical cracks beginning to show near middle of each side, $\frac{1}{8}$ in. wide. 18 to 23 mins.—horizontal cracks showing in beam ends at top increasing to $\frac{1}{2}$ in. at 32 mins. 27 mins.—horizontal checks $\frac{1}{8}$ in. wide and $1\frac{1}{2}$ in. apart quite general. 30 mins.—lazy reddish flames from combustion of wood envelop whole column. 32 mins.—vertical cracks in charred column generally $\frac{3}{8}$ in. wide. 42 mins.—head of column going down fast, but no visible sign of distress. 44 mins.—vertical cracks $\frac{5}{8}$ in. wide; horizontal checks $\frac{1}{4}$ in. wide. Settlement of head of column due to heating of cap $\frac{1}{8}$ in. at 25 mins., $1\frac{3}{8}$ in. at 40 mins., and $3\frac{1}{8}$ in. at 46 mins. (See Fig. 47.) 50 mins.—failure due to cracking of cap.

After failure.—Cap broken into two pieces, cracking transversely at center. Pintle intact except for slight rounding of lower bearing surface. Beam ends charred to depth of $1\frac{1}{2}$ in. on sides and ends; inner surface surrounding pintle barely scorched. Sides of column charred to depth of $1\frac{1}{4}$ in.; minimum section of unburned wood $8\frac{3}{4}$ by $8\frac{3}{4}$ in. Vertical cracks on sides extend inward to a depth of $\frac{3}{4}$ in. below sound wood. Column split in two in upper 3 ft. due to end of pintle being forced down through cap into top of column at failure. Top bearing surface very little charred but fibers crushed and broomed, forced outward 3 in. beyond sides, also down into crack in column and up into crack in cap. Fibers tough and ropy and bent over for a length of 3 in. Length of column before test, 11 ft. $1\frac{7}{8}$ in.; average after test, 10 ft. $7\frac{1}{8}$ in.; decrease in length, 6 in. (Figs. 80, 140, and 47.)

TEST NO. 80. LONG-LEAF PINE, WITH STEEL-PLATE CAP AND TIMBER-STRUT BEARING; COLUMN AND CAP PROTECTED BY ONE THICKNESS OF $\frac{3}{8}$ -IN. GYPSUM WALL BOARD.—1 min.—kalsomine burned off, showing filler in joints and nailing. 5 mins.—paper on outer surface of wall board charring and flaking, about one-half off at 7 mins. 15 mins.—about two-thirds of outer paper now off; no curling of wall board. 16 to 18 mins.—cracks noted near corners at bracket on east and west sides, up to $\frac{1}{8}$ in wide, extending and increasing to $\frac{1}{4}$ in. on west at 26 mins. 19 to 25 mins.—horizontal cracks developed on north, south, and west faces from 1 to 4 ft. up; wood burning freely at cracks, also a little along corners. 27 mins.—corner beading buckled out at several places, corners burning freely up to 7 ft. above base and to top at 40 mins. 31 mins.—cracks and free burning of wood on east face to $9\frac{1}{2}$ ft. up. 32 mins.—about two-thirds of wall board on west side of cap fell, exposing steel. 35 mins.—board fell at cap on east, exposing all of steel. 37 mins.—board in lower half buckled out at corners; horizontal cracks about 18 in. on centers on all sides with free burning of wood. 41

to 54 mins.—board fell off in places, exposing wood; at end of this period wood was exposed on north at 4 ft. up, on east 2 to 8 ft. up, and on south 2 to 10 ft. up; board remaining in place, much shattered. 1 hr. 3 mins.—continued cracking and falling of board; steel cap slanting slightly to north. 1 hr. 6 mins.—all board down on north 2 to 4 ft. and 7 to 8 ft. up; on west from 4 ft. up to top. 1 hr. 10 mins.—cap tilted, south edge dropped $1\frac{1}{2}$ in. Settlement of top of column due to heating of cap $\frac{1}{8}$ in. at 15 mins., $\frac{1}{2}$ in. at 40 mins., and $2\frac{7}{8}$ in. at 1 hr. 10 mins. 1 hr. 13 mins.—failure by tilting of cap, and slipping of same on top bearing strut, column and cap being carried to north.

After failure.—Side plates of cap bent, also inner bolt on south side; bearing plate dished up 1 in. at middle. Strut did not slip and its upper bearing surface was uninjured; sides of strut charred to depth of $\frac{1}{2}$ in., lower bearing surface of strut charred about $\frac{1}{4}$ in. on edges, brown over rest of area. Fibers crushed and bent over $1\frac{1}{2}$ to 3 in. Strut cracked by longitudinal shearing. Sides of column charred to depth of 1 to $1\frac{1}{4}$ in.; minimum section of unburned wood, $9\frac{3}{8}$ by $8\frac{3}{4}$ in.; vertical cracks burnt in $\frac{3}{4}$ to 1 in. deeper. Top bearing surface of column charred only at edges; surface convex, middle being about $\frac{3}{8}$ in. higher than edge; fibers crushed and bent over to south for depth of about $\frac{1}{2}$ in.; bearing surface smooth and hard. Length before test, 12 ft. 2 in.; average after test, 12 ft. 1 in.; decrease in length, 1 in. (Figs. 81, 141, and 47.)

TEST No. 81. LONG-LEAF PINE WITH STEEL-PLATE CAP AND TIMBER-STRUT BEARING; UNPROTECTED.—2 mins.—surface of column blazing, lower 3 ft. 6 mins.—columns burning freely on south and west, lower half. 5 mins.—a few vertical cracks near middle. 8 mins.—column charred; irregular surface checks $\frac{1}{8}$ -in. wide, quite general. 16 mins.—several vertical cracks $\frac{1}{4}$ in. wide on all sides; vertical cracks, $\frac{1}{8}$ in. wide. 19 mins.—gases heavy, making observation difficult. 32 mins.—color noted in cap. 34 mins.—all cracks about $\frac{3}{8}$ in. wide; settlement of top of column due to heating of cap $\frac{1}{8}$ in. at 20 mins., increasing to $\frac{1}{4}$ in. at 30 mins., and $1\frac{1}{8}$ in. at 34 mins. 35 mins.—failure with top of column sliding to south and west; fire kept burning until $43\frac{1}{2}$ mins., as it was not possible to ascertain definitely if failure had taken place, owing to heavy smoke; column yielded quite gradually, only slight report heard at failure. 45 mins.—water applied to column extinguishing flames completely at 55 mins.

After failure.—Upper part of side plates of cap bent toward the east; bearing plate dished up $\frac{3}{4}$ in. at middle. Strut did not slip and its upper bearing surface was uninjured; lower bearing surface of strut charred to depth of $\frac{1}{8}$ in. and fibers crushed and bent over to depth of $\frac{1}{2}$ to 1 in.; strut split by longitudinal shearing. Sides of column charred to depth of $1\frac{3}{8}$ in.; minimum section of unburned wood 9 by 9 in. Top bearing surface of column charred not deeper than $\frac{1}{8}$ in. except at edges; fibers crushed and bent over to north for a depth of about $\frac{1}{2}$ in. Length before test, 12 ft. 2 in.; average after test, 12 ft. $1\frac{1}{8}$ in.; decrease in length, $\frac{7}{8}$ in. (Figs. 81, 141, and 47).

TEST No. 82. DOUGLAS FIR WITH CAST-IRON CAP AND PINTLE; UNPROTECTED.—3 mins.—surface of column blazing in lower 3 ft.; horizontal surface checks due to charring, $1\frac{1}{2}$ in. on centers. 10 mins.—column blazing freely all over; several vertical checks on all sides, $\frac{1}{8}$ in. wide, small pieces of charcoal falling. 14 mins.—horizontal checks $\frac{1}{8}$ in. wide. 24 mins.—horizontal and vertical checks about $\frac{1}{4}$ in. wide. 25 mins.—column flaming all over, but not so freely as pine columns at this stage. 36 mins.—pieces of charcoal falling off under cap, indicating rapid dropping of head of column. 38 mins.—cap nearly level; no color. 41 mins.—buckling under cap more pronounced; noise heard at bearing. $42\frac{1}{2}$ mins.—cap still nearly level. 44 mins.—wood fibers under cap appear to crush and buckle out. Settlement of top of column due to heating of cap $\frac{1}{8}$ in. at 15 mins., 1 in. at 30 mins., $3\frac{1}{2}$ in. at 41 mins., and $5\frac{1}{8}$ in. at 45 mins. $45\frac{1}{4}$ mins.—failure due to cracking of cap.

After failure.—Cap broken transversely into two pieces. Small pieces chipped from outer edge of pindle on southwest at bottom; otherwise pindle uninjured. Beam ends charred on sides and ends to maximum depth of 1 in.; inner surface of wood surrounding pindle very slightly charred in places. Sides of column charred to depth of $1\frac{1}{8}$ in.; minimum section of unburned wood, $9\frac{1}{2}$ by $9\frac{3}{8}$ in.; vertical cracks did not extend into uncharred wood. Top bearing surface of column charred not more than $\frac{1}{8}$ in. deep, but was broomed and crushed; fibers bent out beyond sides of column, also pushed up 3 in. into crack in cap. A piece cut out of southwest corner showed fibers crushed and turned over 3 to $3\frac{1}{4}$ in. below bearing surface; fibers tough, hardly scorched. Length before test, 11 ft. $1\frac{5}{8}$ in.; average after test, 10 ft. 6 in.; decrease in length, $7\frac{3}{8}$ in. (Figs. 82, 140, and 47.)

TEST NO. 83. DOUGLAS FIR, WITH STEEL-PLATE CAP AND TIMBER-STRUT BEARING; UNPROTECTED.—2½ mins.—surface of column blazing lower half; top just beginning to burn. 2 to 4 mins.—crackling noises heard. 5 mins.—column blazing all over. 15 mins.—very little blazing, hardly any crackling, secondary air gate found shut; opened. 18 mins.—horizontal surface checks $\frac{1}{8}$ in. wide, 1 in. on centers; 2 to 4 vertical cracks on each side not over $\frac{1}{8}$ in. wide. 24 mins.—column again burning freely. 31 mins.—horizontal checks $\frac{1}{4}$ in. wide. 37 mins.—vertical cracks up to $\frac{3}{8}$ in. wide; very little crackling. 38 mins.—cap tilted, south edge dropped $1\frac{1}{2}$ in.; settlement of top of column, due to heating of cap $\frac{1}{8}$ in. at 14 mins., $1\frac{1}{8}$ in. at 30 mins., and $2\frac{9}{8}$ in. at 38 mins. 38½ mins.—failure with top of column sliding to north carrying cap with it. 40½ mins.—water applied to column extinguishing flames completely at 45½ mins.

After failure.—Side plates of cap bent outward at top, also inner bolt on north bent out; bearing plate dished up 1 in. at middle in longitudinal direction. Strut did not slip, and its upper bearing surface was uninjured; lower bearing surface of strut charred to depth of $\frac{1}{8}$ in. and fibers crushed and bent over to north maximum of 1 in. on north side. Sides of column charred to depth of $1\frac{1}{8}$ in.; minimum section of unburned wood $9\frac{1}{2}$ by $9\frac{1}{2}$ in.; top of bearing surface of column very little charred, but discolored to depth of 1 in.; surface convex, rounded over from $\frac{1}{2}$ to 1 in. at edges; fibers crushed and bent over mostly on east and west sides; wood quite soft to a depth of $\frac{1}{4}$ in. Length before test, 11 ft. 2 in.; average after test, 11 ft. $1\frac{1}{4}$ in.; decrease in length, $\frac{3}{4}$ in. (Figs. 82, 141, and 47).

XI. RESULTS OF FIRE AND WATER TESTS

1. APPLIED LOADS, DURATION AND EFFECT OF FIRE AND WATER

In the fire and water test, the column was subjected to working load, and to fire for a predetermined period not exceeding one hour, after which a hose stream was applied to three sides. On cooling, the column was either loaded to failure or subjected to an excess load equal to about twice the load sustained during the fire and water periods.

A general summary of applied loads, duration and relative intensity of fire exposure, duration and pressure of hose-stream application, and the general effects of fire and water are given in Table 44. Further details of columns and protections are given in Tables 4a and 4b (p. 51-54).

2. PHOTOGRAPHIC RECORDS

Views of the columns in the fire and water series at several stages of the test are given in Figs. 83 to 89, Appendix A (p. 242-248).

3. FURNACE AND COLUMN TEMPERATURES

The temperatures observed in the furnace and test columns are given by the curves in Figs. 142 to 145, Appendix B (p. 303-306). The arrows on the plots indicate the time water was applied in each test.

4. LONGITUDINAL DEFORMATION

The expansion of the column during the fire period and contraction on application of water were determined by measurement of the movement of the head of the column, and the amounts are given in the respective test logs. These effects were quite small except in the case of the unprotected cast-iron columns, both of which attained maximum expansion before water was applied (Fig. 46, p. 124).

5. SUBSEQUENT LOADING TESTS

The loading to which the columns were subjected subsequent to the fire and water test are given in Table 44. They are loaded to their maximum sustaining capacity with the exception of four columns that were loaded to about twice their design working load and reserved for use in further tests, and the one protected by plaster on metal lath on which a subsequent fire test to failure under working load was made.

6. LOG OF FIRE AND WATER TESTS

(a) CONCRETE PROTECTIONS

TEST NO. 101. ROLLED H. 2-IN. CONCRETE PROTECTION. UPPER SECTION, LIMESTONE; MIDDLE SECTION, TRAP; LOWER SECTION, JOILET GRAVEL. TIED.

Fire Test.—30 mins.—no effect noted; flame very smoky. 32 mins.—vertical cracks on east and west faces near corners in lower 3 ft., $\frac{1}{4}$ in. wide at bottom. 38 mins.— $\frac{1}{8}$ -in. vertical crack at center of south face, 2 ft. up. 45 mins.—cracks open $\frac{3}{8}$ in. to $\frac{3}{4}$ in.; small spall on southwest corner, 3 ft. up. 60 mins.—gas shut off; no change in column except cracks had opened up slightly.

Water Test.—61 $\frac{1}{4}$ mins.—water applied to column through a 1 $\frac{1}{8}$ -in. nozzle; pressure at base of play pipe, 50 lbs. per sq. in.; nozzle was maintained at a distance of 20 ft. away from and to the west of column in all tests and the stream applied directly to the west face and at an angle to the north and south of faces; stream was played slowly up and down over all three faces of column. 61 $\frac{1}{2}$ mins.—pieces washed off southwest corner and west face, 3 ft. up. 61 $\frac{3}{4}$ mins.—some pieces fell off west face in upper section. 62 $\frac{1}{4}$ mins.—pieces falling on west face and corners in middle section. 63 $\frac{3}{4}$ mins.—concrete fell in lower 2 ft., exposing steel. 64 $\frac{1}{2}$ mins.—north

TABLE 44.—RESULTS OF FIRE AND WATER TESTS

Test No.	Section	PROTECTION			Load Sustained During Test, Lb.	FIRE TEST			WATER TEST			LOAD TEST	
		Thickness and Kind of Covering	Materials and Details	Age of Covering, Days		Duration, Minutes	Furnace Exposure, Per cent	Results	Duration, Min.	* Water Pressure, Lb. per Sq. In.	Results	Excess Load, Lb.	Ultimate Load, Lb.
101	Rolled H	2-in. concrete	†1:2:4 Chicago limestone concrete 1:2:4 New York trap concrete 1:2:4 Joliet gravel concrete Wire tie wound spirally on 8-in. pitch	515	119500	60	101.5	1/8 to 3/8-in. vertical cracks about 3 ft. long on three sides in bottom section	5	50	Flange entirely exposed on S. in lower 3 ft.; edges of flanges exposed in places on corners of W. face. Concrete generally pitted 1/8 to 1/4 in. on W. face; at 8 ft. up to depth of 1 in.	234000	Not applied
102	Rolled H	2-in. concrete	†1:2:4 New York trap concrete 1:2:4 Joliet gravel concrete 1:2:4 Chicago limestone concrete No tie	518	119500	60	96.8	No cracking or spalling	5	50	N. flange exposed for 10 ft. S. flange partly exposed, concrete being loose for 10 ft. Concrete washed off W. face to depth of 1/2 to 1 1/4 in.		442000
103	Plate and Angle	4-in. concrete	†1:2:4 New York trap concrete 1:2:4 Rockport granite concrete 1:2:4 Chicago limestone concrete Wire tie wound spirally on 8-in. pitch	518	116000	60	101.8	No cracking except small crack at bracket. No spalling	5	50	Concrete washed away on W. side in middle section 1 1/2 to 3 1/2 in. in upper and lower sections, 1/2 in. at center, 3 1/2 in. on corners. Very little effect on N., S., and E. sides.		a537000
104	Plate and Angle	2-in. concrete	†1:2:5 cinder concrete 1:2:4 Cleveland sandstone concrete 1:2:4 New York trap concrete Wire tie wound spirally on 8-in. pitch	517	116000	60	106.3	Vertical cracks, 3 to 4 ft. long and 1/4 to 1/2 in. wide on three sides of center section. Small amount of spalling in center section	5	50	W. side, concrete pitted 1/2 to 2 in. in upper and lower sections and 1/4 in. in center section. Steel bare on S. side, center section. Flange edges partly exposed on W. side in lower and sections	234000	Not applied
105	Plate and Angle	2-in. hollow clay tile	†Surface clay, Boston district. New Jersey semi-fire clay. Ohio shale. No filling. Outside wire ties	541	116000	45	102.3	Outer shells shattered in lower 3 courses. Other courses unaffected except for a few fine vertical cracks	2 1/2	30	Steel exposed on three sides in lower section and in part of middle section. Tile nearly intact on upper half of column		b445000
106	Plate and Angle	2-in. hollow clay tile	†Ohio semi-fire clay. Surface clay, Chicago district. Ohio semi-fire clay. Concrete filling. Outside wire ties, upper half. 3/8-in. wire mesh in horizontal joints, lower half.	528	116000	45	108.2	Cracking and spalling of outer shells especially in middle section and in lower part of upper section	2 1/4	30	All tile washed off except in top and bottom courses and some inner shells on unexposed side. Fill remained in place	228000	Not applied
107	Plate and Channel	4-in. hollow clay tile	†Ohio shale. New Jersey semi-fire clay. Surface clay, Boston district. No filling. Ties same as in No. 106	537	111000	45	103.2	Tile in upper section cracked and a few outer shells spalled. Middle and lower sections not damaged except for a few cracks	2 1/2	30	Almost all tile washed off in 3 upper courses. Middle and lower sections little damaged		b348000
108	Rolled H	2-in. solid gypsum block	Western gypsum, upper half Eastern gypsum, lower half 1:1:4 poured gypsum fill Wall ties in joints, upper half Wire mesh in joints, lower half	507	119500	45	103.7	Surface checks 1/8 in. wide general. Joints near bottom open 1/8 to 1/4 in.		30	3/4 of covering carried down on W., N., and S. sides exposing flanges. Blocks remaining washed off to depth of 3/4 in.		b311000
109	Rolled H	4-in. solid gypsum	Eastern gypsum, upper half Western gypsum, lower half 1:1:4 poured gypsum fill Wire mesh in joints, upper half Wall ties in joints, lower half	507	119500	60	100.8	Surface checks 1/8 to 1/4 in. wide general. Lower vertical joints open 1/32 in.	5	50	Gypsum washed away on three sides for a depth of 1 in., also mortar washed out of vertical joints on W.	234000	Not applied
110	Plate and Angle	Double layer plaster on metal lath	Two 2-coat layers of Portland cement plaster on metal lath, with 1/2-in. air space between layers	500	116000	45	99.8 99.5	Plaster cracked off at bracket due to expansion of covering Corners spalled slightly	3 1/4	30	Outer layer of plaster washed off in a few places, mostly at top and at corners. A number of fine horizontal and vertical cracks appeared	Not applied	c Not applied
111	Square Vertically Reinforced Concrete	2-in. concrete	†1:2:4 Chicago limestone concrete 1:2:4 Meramec R. gravel concrete 1:2:4 Chicago limestone concrete Four 1-in. sq. vertical bars	520	101000	60	100.8	Many vertical and diagonal cracks in middle section 1/8 to 3/8 in. wide. Some corner concrete loose	5	50	Concrete washed away to depth of 2 in. on W. face in middle section. Bars exposed nearly full length on W. side. Concrete washed away to 1/2 to 1 in. depth on parts of N. and S. sides		379000
112	Round Vertically Reinforced Concrete	2-in. concrete	†1:2:4 Chicago limestone concrete 1:2:4 Meramec R. gravel concrete 1:2:4 Joliet gravel concrete Six 1-in. sq. vertical bars	524	107500	60	95.6	Considerable cracking and crushing in middle section, some cracks extending into upper and lower sections	5	50	Concrete washed away to depth of 2 in. exposing bars in middle and part of lower section on exposed sides. Concrete pitted to 1 in. depth on upper section		423000
113	Hoop Reinforced Concrete	2-in. concrete	†1:2:4 New York trap concrete 1:2:4 Meramec R. gravel concrete 1:2:4 Rockport granite concrete Six 3/4-in. sq. vertical bars 1/4-in. φ hooping on 1 1/2-in. pitch	520	129000	60	100.0	General buckling and spalling of outer shell of concrete in middle section exposing spiral at several points. 1 1/2-in. wide vertical crack formed in lower section	5	50	Nearly all concrete washed away outside of spiral on exposed sides also on unexposed side in center section. Concrete pitted to inner line of wire		536000
114	Round Cast Iron	Unprotected	Vertically cast		98500	22 1/2	103.2	Column glowing dull red in lower half	1	30	No cracks in metal. Maximum deflection 1 3/8 in. toward side exposed to water		527000
115	Round Cast Iron	Unprotected	Vertically cast		98500	30	93.7	Column glowing dull red all over	1	30	No cracks in metal. Maximum deflection 1/2 in. toward side exposed to water		502000

† Three kinds of concrete or tile used on each column placed in three vertical sections in the order named beginning at the top of the column.

c—Fire test to failure made subsequent to fire and water test.

*Pressure at base of playpipe. Water applied through a 1 1/8-in. nozzle. Stream directed at west, north and south sides.

a—Column did not fail under load of 547,000 lb., capacity of machine. The concrete covering was then removed for a distance of 2 ft. 6 in. near the middle and load re-applied, the maximum sustained being 537,000 lb.

b—Covering completely removed before applying load.

face washed off to depth of $1\frac{1}{2}$ in., 1 to $2\frac{1}{2}$ ft. up; edge of flange exposed on northwest corner. 65¾ mins.—pieces fell off north face near top. 66¼ mins.—water shut off; duration of water test, 5 mins.

After Test.—Edge of flange exposed on northwest corner 1 to $2\frac{1}{2}$ ft. up, and southwest corner for 4 in. at 5 ft. and at 8 ft. up; these corners spalled not over $1\frac{1}{2}$ by $1\frac{1}{2}$ in. for rest of length. Concrete in center of west face generally pitted from $\frac{1}{8}$ to $\frac{1}{4}$ in. for the whole length; at bottom of upper section concrete pitted to depth of 1 in. for about 6 in. On south face all concrete washed off in lower 3 ft., exposing flange; about two-thirds of rest of surface pitted slightly. On north face surface generally smooth except at northwest corner. Fine vertical cracks in east face nearly full length, $1\frac{1}{2}$ in. from corners. Column expanded 0.26 in. during the fire test and contracted 0.06 in. during the water test.

Load Test.—On the following day the column was subjected to a load of 234 000 lbs. this being calculated as the design dead load plus $2\frac{1}{2}$ times the live load, assuming the former to be one-third of the latter. A faint cracking sound was heard at 69 000 lbs. At about 175 000 lbs. a dull thud was heard and several small pieces of concrete fell off northwest corner about halfway up. Column withstood test satisfactorily. The column recovered from the depression due to the loading within 0.002 in. on release of load. (Figs. 83 and 142.)

TEST NO. 102. ROLLED H. 2-IN. CONCRETE PROTECTION. UPPER SECTION, TRAP; MIDDLE SECTION, JOILET GRAVEL; LOWER SECTION, LIMESTONE. NO TIE.

Fire Test.—21 mins.—slight flaking at southeast corner, 9 ft. up. 41 mins.—column glowing all over. 60 mins.—gas shut off. No cracking or spalling.

Water Test.—62¼ mins.—water applied to column, 50-lb. pressure. 62¾ mins.—cracks noted on west face near both corners; steam very heavy, making observation difficult. 64 mins.—concrete fell on north in lower 10 ft., exposing steel. 65 mins.—long vertical crack at southwest corner, exposing flange in places in lower 10 ft. 67¼ mins.—water shut off; duration of water test, 5 mins.

After Test.—North flange exposed completely $1\frac{1}{2}$ to 11 ft. up. South flange exposed on southwest corner for $2\frac{1}{2}$ by 14 in. at $2\frac{1}{2}$ ft. up, for 4 by 20 in. at 5 ft. up, for 3 by 10 in. at 8 ft., for $1\frac{1}{2}$ by 12 in. at 10 ft. up; concrete loose on south flange from 1 to $11\frac{1}{2}$ ft. up. On west concrete washed off to depth of $\frac{1}{2}$ to $1\frac{1}{2}$ in. at 3 ft., $4\frac{1}{2}$ ft., 8 ft., and $10\frac{1}{2}$ ft. above base; concrete least washed off on middle section; column expanded 0.26 in. during the fire test and contracted 0.09 in. during the water test.

Load Test.—On the following day the column in the unstripped condition was loaded until failure occurred at 442 000 lbs. by buckling to west, about 8 ft. above base. Scaling on the surface of the steel was first noted on the north flange at 373 000 lbs., the yield point of the steel as determined from the depression curve of the top of the column, being attained at 400 000 lbs. (Figs. 83 and 142.)

TEST NO. 103. PLATE AND ANGLE. 4-IN. CONCRETE PROTECTION. UPPER SECTION, TRAP; MIDDLE SECTION, GRANITE; LOWER SECTION, LIMESTONE. TIED.

Fire Test.—60 mins.—gas shut off; $\frac{1}{8}$ -in. vertical crack, 4 in. long, on west near southwest corner at bracket; no other cracking, spalling or other visible effects on column.

Water Test.—62 mins.—water applied to column; 50-lb. pressure. 62¾ mins.—small piece falling on west; face and corners rounded. 63½ mins.—continued falling of small pieces; steam heavy. 64½ min.—water effects most marked at bottom of middle section. 65½ mins.—continued falling of small pieces. 67 mins.—water shut off; duration of water test, 5 mins.

After Test.—Little effect on north and south faces back of corners except in middle section, where granite concrete was pitted to depth of $\frac{1}{4}$ in. quite generally. On west face the middle section was most affected and the upper section least affected; concrete in middle section quite generally washed away to depth of $1\frac{1}{2}$ in. at center, and $3\frac{1}{2}$ in. at corners; in upper and lower sections, $\frac{1}{4}$ to $\frac{1}{2}$ in. at centers and $3\frac{1}{2}$ in. at corners; concrete in middle section cracked along flanges near each corner; steel flanges not exposed. East face unaffected. Column expanded 0.15 in. during the fire test.

Load Test.—On the following day the column in the unstripped condition was loaded to 547 000 lbs., the capacity of the machine, without signs of distress or failure. Column was then stripped from 5 ft. 2 in. to 7 ft. 10 in. above base and loaded until failure occurred at 537 000 lbs. by buckling to west, 7 ft. 8 in. above base. Scaling of steel was first noted at 433 000 lbs. and decided yielding began at 475 000 lbs. (Figs. 83 and 142.)

TEST NO. 104. PLATE AND ANGLE. 2-IN. CONCRETE PROTECTION. UPPER SECTION, CINDER; MIDDLE SECTION, SANDSTONE; LOWER SECTION, TRAP. TIED.

Fire Test.—31 mins.—fine vertical crack, 24 in. long, on west near southwest corner, $6\frac{1}{2}$ ft. up; also on east, southeast corner, $4\frac{1}{2}$ ft. to 7 ft. up; slight crushing on southeast corner 8 ft. up, small piece falling at 33 mins. 35 mins.— $\frac{1}{4}$ -in. crack on center of south face, $6\frac{1}{2}$ to $9\frac{1}{2}$ ft. up, extending to southwest corner at upper end. 50 mins.—cracks previously noted opening up $\frac{1}{4}$ to $\frac{1}{2}$ in. and extending in length in middle section. 53 mins.—piece spalled off northeast corner, 7 ft. up. 60 mins.—gas shut off; only cracks are the three noted in middle section, 3 to 4 ft. long, and $\frac{1}{4}$ to $\frac{1}{2}$ in. wide; no cracks on north.

Water Test.—61½ mins.—water applied to column; 50-lb. pressure. 61¾ mins.—concrete fell off on south 5 to 8 ft. up, exposing steel. 63 mins.—concrete in lower 4 ft. on west washed and pitted considerably. 63½ mins.—concrete on northwest corner fell, 4 to 7 ft. up. 64¾ mins.—upper 4 ft. on west pitted. 65 mins.—piece fell on northwest corner, 10 ft. up. 66 mins.—pieces washed off corners on west in lower 4 ft. 66½ mins.—water shut off; duration of water test, 5 mins.

After Test.—West face pitted $\frac{1}{2}$ to 2 in. in lower section, $\frac{1}{4}$ in. in middle section, and $\frac{1}{2}$ to 2 in. in upper section; northwest corner off for full length, exposing flange edge for $1\frac{1}{2}$ by 12 in. at 2 ft. up, for 2 by 12 in. 4 ft. up, and for $1\frac{1}{2}$ in. 5 to 9 ft. up; north face intact except for spalling on northwest corner; southwest corner spalled full length exposing flange edge for 3 by 4 in. 2 ft. up, and 2 by 6 in. 6 ft. up; entire south flange exposed in middle section; east side intact except for a few fine cracks. Column expanded 0.26 in. during the fire test and contracted 0.09 in. during the water test.

Load Test.—On the following day the column in the unstripped condition was subjected to an excess load of 234 000 lbs. without developing any signs of distress and with full recovery from the deformation on removal of load. (Figs. 84 and 142.)

(b) HOLLOW CLAY-TILE PROTECTIONS

TEST NO. 105. PLATE AND ANGLE. 2-IN. HOLLOW CLAY-TILE PROTECTION, UNFILLED. UPPER SECTION, SURFACE CLAY, BOSTON DISTRICT; MIDDLE SECTION, SEMIFIRE CLAY, NEW JERSEY DISTRICT; LOWER SECTION, OHIO SHALE. NO FILLING. OUTSIDE WIRE TIES.

Fire Test.—2 to 3 mins.—vertical cracks and spalling of parts of outer shells on north and west sides in lower section. 3 to 20 mins.—continued cracking, spalling, and bulging of outer shells in lower section; tile in upper half very little affected. 30 mins.—

a little more spalling in lower section; vertical cracks on west at bracket, and at south and west 5 ft. up, $\frac{1}{8}$ in. wide. 39 mins.—outer shells spalled off on east and west, 3d course, shell hanging loose on east, 2d course. 45 mins.—gas shut off; very little damage to middle and upper sections except for vertical cracks as noted; lower three courses cracked and spalled.

Water Test.—46 mins.—water applied to column; 30-lb. pressure. 46½ mins.—all tile washed off on west 1 to 2 ft. up, and on north 1 to 3 ft. up, exposing steel. 47 mins.—all tile down in lower sections on north and west 1 to 4 ft. up. 47½ mins.—tile down on north and west 4 to 5 ft. up. 48 mins.—tile still in place on south except outer shell spalled 3 to 4 ft. up; tile down on west in center 5 to 6 ft. up. 48½ mins.—water shut off; duration of water test, 2½ mins.

After test.—Tile down and steel exposed on north and east 1 to 5 ft. up and on west 1 to 6 ft. up; tile loose on south 1 to 5 ft. up. Tile in upper half of column almost intact except outer shell cracked on west 6 to 7 ft. up, and part of outer shell off on north at bracket. Ties in lower five courses fell down, other ties in place. Expansion during fire test, 0.22 in.; contraction during water test, 0.13 in.

On the following day the column was stripped. Mortar in joints found to be fairly full, although in about one-half of the joints it had been washed out to depth of $\frac{1}{2}$ to $\frac{5}{8}$ in. Mortar on flanges broke away from the steel and adhered to the tile. Lateral deflection less than $\frac{1}{8}$ in.

Load Test.—The stripped column was loaded until failure occurred at 445 000 lbs. with buckling to east about 6 ft. above base. (Figs. 84 and 143.)

TEST NO. 106. PLATE AND ANGLE. 2-IN. HOLLOW CLAY-TILE PROTECTION. UPPER AND LOWER SECTIONS, OHIO SEMI-FIRE CLAY; MIDDLE SECTION, SURFACE CLAY, CHICAGO DISTRICT. OUTSIDE WIRE TIES IN UPPER HALF OF COLUMN, WIRE MESH IN JOINTS IN LOWER HALF. CONCRETE FILLING.

Fire Test.—2½ to 6 mins.—outer shells spalling at corners in middle section. 9 to 12 mins.—outer shells shattered on west in middle section, but did not spall off; $\frac{1}{8}$ -in. cracks near corners on west in upper section. 12½ to 18½ mins.—outer shells spalled and bulging in middle section; at end of this period, all of outer shells were down in 5th course on north and west and partly down on east. 20½ mins.—vertical crack in upper section on west 9½ to 11½ ft. up, $\frac{5}{8}$ in. wide, opening to 1 in. at 26 mins. 23 mins.—outer shell buckled on east, 9 ft. up. 25½ mins.—tie wire broken on 10th course. 31 to 36 mins.—continued cracking and spalling of outer shells in middle section, also bulging out on north 7 ft. up. 38 mins.—outer shell spalled on 6th course on north. 40 mins.—all outer shells spalled on east 3 to 5 ft. up. 44 mins.—outer shells on east 7 ft. up buckle out 2 in. 45 mins.—gas shut off; outer shells are off on 5th, 8th, and 9th courses on west, and on 5th and 6th courses on north; column expanded 0.24 in. during the fire test.

Water Test.—47½ mins.—water applied to column; 30-lb. pressure. 47¾ mins.—considerable amount of tile fallen; steam obscures view. 48½ mins.—tile down in middle 8 ft. on north, south and west, exposing flanges on north and south; concrete filling in place. 49¾ mins.—water off; duration of water test, 2¼ mins.

After Test.—Steel exposed on north 1 to 11 ft. up, on south from 1 to 10 ft. up except where parts of inner shells remained in place; wire mesh still in place on north at 1 ft., 4 ft., 5 ft. and 6 ft. up. On west tile off, exposing concrete 2 to 10 ft. up; outer shell off 10 to 12 ft. up. On east side, concrete exposed 2 to 4 ft. up; the rest was covered by tile or inner shells of same. Tile all in place on bottom course on all sides and on bracket course on north, east, and south sides.

Load Test.—On the following day the column was subjected to an excess load of 228 000 lbs. without developing any signs of distress and recovered from deformation almost completely on removal of load. (Figs. 85 and 143.)

TEST NO. 107. PLATE AND CHANNEL. 4-IN. HOLLOW CLAY-TILE PROTECTION. UPPER SECTION, OHIO SHALE; MIDDLE SECTION, SEMIFIRE CLAY, NEW JERSEY DISTRICT; LOWER SECTION, SURFACE CLAY, BOSTON DISTRICT. OUTSIDE WIRE TIES ON UPPER HALF OF COLUMN; WIRE MESH IN JOINTS IN LOWER HALF. NO FILLING.

Fire Test.—12 mins.—no cracking or spalling noted. 15 mins.—vertical cracks on east from 8 ft. up to top, $\frac{1}{4}$ in. wide; part of outer shell spalled on 11th course, east; $\frac{1}{8}$ -in. vertical crack on east 6 to 8 ft. up. 19 mins.— $\frac{1}{8}$ -in. vertical crack on west 6 to 9 ft. up. 22 mins.—cracks on east and west near top, $\frac{1}{2}$ to 1 in. wide. 25 to 27 mins.—outer shells on west bulging out $1\frac{1}{2}$ to 2 in. in upper section, held by ties. 31 to 35 mins.— $\frac{1}{4}$ -in. vertical crack on west at south corner from 9 ft. up to top; other cracks opening up and outer shells bulging in upper section. 42 mins.—continued cracking in upper section; new crack on north 1 in. wide, 11th course; tile on east 11th and 12th courses cracked. 45 mins.—gas shut off; tile in upper section generally cracked and outer shells loose; lower section practically intact; middle courses have a few vertical cracks as noted.

Water Test.—46 mins.—water applied to column; 30-lb. pressure. 46½ mins.—little effect; outer shell on west fractured in upper section. 46¾ mins.—little apparent change. 47¾ mins.—outer shell down on west 9 to 11 ft. up. 48¼ mins.—tile almost all down on upper three courses. 48½ mins.—water shut off; duration of water test, 2½ mins.

After Test.—All tile down in 10th, 11th, and 12th courses, except on north side, where tile is held by bracket; some mortar adheres to north and south flanges. Tile shattered on east and west, in 9th course. Tile in good condition in lower 8 ft. except for a few $\frac{1}{8}$ -in. vertical cracks on east and west 6 to 8 ft. up, and small hole in outer shell on west 7 ft. up. Mortar and mesh in joints in lower courses in good condition, it being necessary to wedge the tile apart in order to remove the covering. Column expanded 0.15 in. during the fire test and contracted 0.10 in. during the water test. Maximum deflection after fire and water test, $\frac{3}{8}$ in. to west.

Load Test.—On the following day the column was stripped and loaded to failure at 348 000 lbs., column buckling to west about 6 ft. above base. Decided yielding of the steel began at 322 000 lbs. (Figs. 85 and 143.)

(c) GYPSUM-BLOCK PROTECTIONS

TEST NO. 108. ROLLED H. 2-IN. SOLID GYPSUM-BLOCK PROTECTION. UPPER HALF, WESTERN GYPSUM; LOWER HALF, EASTERN GYPSUM. FILLED.

Fire Test.—13 mins.—fine surface checks beginning to develop. 28 mins.—surface checks generally $\frac{1}{8}$ in. wide and $\frac{3}{8}$ in. deep near lower end of column to very fine at top. 36 mins.—joints at bottom open $\frac{1}{8}$ to $\frac{1}{2}$ in. 43 mins.—very little change in fire effects. 45 mins.—gas shut off.

Water Test.—46 mins.—water applied to column; 30-lb. pressure. 47¾ mins.—little effect noted except for surface erosion on west. 48 mins.—blocks fell on west 2 ft. 10 in. to 5 ft. 8 in. up. 48¼ mins.—blocks down on west $1\frac{1}{2}$ to 9 ft. up, exposing flange edges and filling. 48½ mins.—blocks down on south, $4\frac{1}{4}$ to 7 ft. up. 48¾ mins.—blocks down, exposing steel nearly full length on north and south. 49 mins.—water off; duration of water test, 3 mins.

After Test.—All blocks down on north $1\frac{1}{2}$ to 9 ft. up, exposing steel; on south, from $1\frac{1}{2}$ ft. up to top of bracket, exposing steel except for a 2 ft. length above middle of column, where mortar remained in place; on west, $1\frac{1}{2}$ to $4\frac{1}{4}$ ft., 6 to 9 ft., and from 11 ft. to top. Filling washed out on west, exposing web from 2 ft. 4 in. to 3 ft. up. All blocks in place on east except at places on corners. Blocks remaining on column on west washed off to a depth of $\frac{3}{4}$ in. to point where fiber in blocks was not charred.

Mortar on flanges, and filling in web had evidently been quite full. Steel not rusted except where exposed by water. Column expanded 0.05 in. during the fire test and contracted 0.05 in. during the water test. Maximum lateral deflection after fire and water tests, $\frac{1}{8}$ in.

Load Test.—On the following day the column was loaded to failure at 311 000 lbs. column buckling to west about 7 ft. above base. (Figs. 86 and 143.)

TEST NO. 109. ROLLED H. 4-IN. SOLID GYPSUM-BLOCK PROTECTION. UPPER SECTION, EASTERN GYPSUM; LOWER SECTION, WESTERN GYPSUM.

Fire Test.—16 mins.—a few corner cracks developed in lower half, $\frac{1}{8}$ by 1 in. 22 mins.—fine surface checks beginning to develop near bottom. 42 mins.—surface checks quite general, forming in $\frac{3}{8}$ -in. squares, checks $\frac{3}{8}$ in. wide at bottom and $\frac{1}{4}$ in. wide at top; corner cracks opening to maximum of $\frac{1}{8}$ in.; vertical joints in lower half open $\frac{3}{8}$ in.; slight spalling at corners. 53 to 57 mins.—surface checks open from $\frac{1}{8}$ in. at base to $\frac{1}{2}$ in. at top of column. 59 $\frac{1}{2}$ mins.—gas shut off; surface checked all over, most at bottom; otherwise covering and column little affected.

Water Test.—60 $\frac{3}{4}$ mins.—water applied to column; 50-lb. pressure. 61 $\frac{3}{4}$ mins.—hose burst, water off; gypsum washed off on west to depth of about 1 in., mortar in joints washed out from 1 to 4 in. depth, exposing ends of ties. Gypsum washed off on north and south sides, $\frac{3}{4}$ to 1 in. depth; east side intact. 72 $\frac{1}{2}$ mins.—water again applied; 50-lb. pressure. 73 $\frac{1}{2}$ mins.—outer 1 in. of gypsum washed off clean on north and south. 74 mins.—almost all of mortar washed out of vertical joints on west. 76 $\frac{1}{4}$ mins.— $\frac{1}{8}$ -in. vertical crack on west near south corner 9 to 10 ft. up. 76 $\frac{1}{2}$ mins.—water shut off; total duration of water test, 5 mins.

After Test.—On west side surface washed very smooth, corners rounded; blocks slightly less than 3 in. thick; no mortar in vertical joints; ties all in place and hold mortar in horizontal joints; on north and south sides, faces of blocks rough, with fibers projecting; blocks about 3 in. thick; all joints generally full; east side little affected; no blocks loose; edges of bracket and stiffener angles exposed. Column expanded 0.04 in. during the fire test; less than 0.01 in. contraction during the water test.

Load Test.—On the following day the column in the unstripped condition was subjected to an excess load of 234 000 lbs., which it withstood without any signs of distress except for a few small vertical cracks in the blocks. (Figs. 86 and 143.)

(d) PLASTER ON METAL-LATH PROTECTION

TEST NO. 110. PLATE AND ANGLE. TWO LAYERS OF PORTLAND-CEMENT PLASTER ON METAL LATH WITH $\frac{3}{4}$ -IN. AIR SPACE BETWEEN LAYERS.

Fire Test.—6 mins.—furnace gases heavy, difficult to see column. 18 mins.—fire more luminous. 19 mins.—plaster cracked and spalled on all sides at bottom of bracket; this probably occurred at about 14 mins.; corners spalled slightly on west 32 mins.—fine crack on east, 2 in. long, 4 ft. up. 45 mins.—gas shut off; several fine cracks noted on all faces.

Water Test.—46 $\frac{1}{4}$ mins.—water applied to column; 30-lb. pressure. 46 $\frac{3}{4}$ mins.—vertical cracks near corners on all faces about 6 ft. long. 47 $\frac{1}{4}$ mins.—piece of plaster, 8 in. square, spalled off north face, 6 ft. up. 48 mins.—plaster washed off southwest corner in upper and lower 3 ft. 48 $\frac{3}{4}$ mins.—plaster washed off northwest corner, lower 4 ft. 49 $\frac{1}{4}$ mins.—water off; duration of water test, 3 $\frac{3}{4}$ mins.

After Test.—On west side plaster off, 1 by 1 in., exposing lath on northwest corner 1 to 4 ft. up. Fine horizontal cracks across face at 4 ft., 6 ft., 7 ft., 7 $\frac{1}{2}$ ft., 8 ft., 9 $\frac{1}{2}$ ft., and 11 ft. up. On north side, lath exposed near west corner, 5 by 18 in. 6 ft. up. On south side, lath exposed on southwest corner, 1 by 2 in., from 7 ft. up to top; fine

horizontal cracks across face at 5 and 8 ft. up. On east side, vertical crack runs full length near south corner; fine horizontal cracks at $2\frac{1}{2}$, 4, and 6 ft. up. Plaster cracked and lath exposed on all sides at bottom of bracket. Column expanded 0.14 in. during the fire test. No contraction during the water test (see Fig. 46).

Subsequent Fire Test.—Since the protection was little injured by the fire and water test, it was subjected to a second fire test to failure on the day following the fire and water test. 5 mins.—plaster bulging out 2 in. on north at bottom of bracket. 7 mins.—crack on west at southwest corner open $\frac{1}{2}$ in., 7 to 8 ft. up, extending further 4 ft. vertically at 11 mins. 14 mins.—bulging of plaster on southeast below bracket. 39 mins.—plaster bulging away from lath in places on southwest corner from 7 ft. up to top. 46 mins.—northeast corner cracks opening up, $8\frac{1}{2}$ to 11 ft. up. 1 hr.—bulging of plaster increasing on all faces at bracket. 1 hr. 54 mins.—no change except all cracks opening up slightly. 2 hrs. 25 mins.—about one half of lath exposed at bracket on west. 2 hrs. 28 mins.—small areas of lath exposed at bracket on east. 2 hrs. $47\frac{1}{4}$ mins.—failure with buckling to west, maximum at $7\frac{1}{2}$ ft. above base. Column expanded maximum of 0.92 in. at 2 hrs. 30 mins.; from this point to one minute before failure it compressed $\frac{1}{2}$ in. (Figs. 87, 144, and 46.)

(e) REINFORCED-CONCRETE COLUMNS

TEST NO. III. SQUARE VERTICALLY REINFORCED CONCRETE. UPPER SECTION, LIMESTONE; MIDDLE SECTION, MERAMEC RIVER GRAVEL; LOWER SECTION, LIMESTONE.

Fire Test.—14 to 24 mins.—slight spalling of small pieces from corners in middle section. 26 to 31 mins.—cracking of concrete in middle section on all sides, cracks $\frac{1}{8}$ to $\frac{3}{8}$ in. wide, and 12 to 30 in. long; cracking and crushing at southwest corner $6\frac{1}{2}$ ft. up, cracks extending diagonally downward on both south and west faces. 36 mins.—upper and lower sections apparently unaffected. $36\frac{1}{2}$ mins.—crack on south $4\frac{1}{2}$ ft. up, extending downward to 2 ft. above base. 37 mins.—cracks at southwest $6\frac{1}{2}$ ft. up, now open $\frac{3}{8}$ in. 46 min.—same crack open $1\frac{1}{2}$ in., new crack extending upward 12 in. from same; crack on east, $5\frac{1}{2}$ ft. up open $\frac{3}{8}$ in. 48 mins.—small vertical and diagonal cracks extending from crack at southeast corner $4\frac{1}{2}$ to $5\frac{1}{2}$ ft. up. 53 mins.— $\frac{1}{8}$ in. vertical and diagonal cracks near northwest corner on north and west 5 ft. up. 60 mins.—gas shut off; very little spalling; concrete at southwest corner in middle section, crushed.

Water Test.— $61\frac{1}{2}$ mins.—water applied to column; 50-lb. pressure. 63 mins.—steam heavy; observations impossible. $63\frac{3}{4}$ mins.—concrete spalled to reinforcing rod on northwest corner in lower half. $63\frac{1}{2}$ mins.—west face washed off to about $1\frac{1}{2}$ -in. depth. 64 mins.—northwest rod exposed to 10 ft. up. $64\frac{1}{2}$ mins.—southwest rod exposed to 11 ft. up. 65 mins.—concrete washed off in middle of column on west, exposing tie. $65\frac{3}{4}$ mins.—little change during past $\frac{3}{4}$ min. $66\frac{1}{2}$ mins.—water off; duration of water test, 5 mins.

After Test.—Concrete washed off both corners of west face 4 by 4 in., exposing reinforcing rods 1 to 11 ft. up; in center of west face concrete is washed off to average depth of 1 in. in upper section, 2 in. in middle section, exposing ties, and $\frac{1}{8}$ in. in lower section; on north, east, and south faces, concrete in place in upper and lower sections except at west corners; in middle section, concrete washed off south face to depth of 1 in., $4\frac{1}{2}$ to $5\frac{1}{2}$ ft. up, and off north face to depth of $\frac{1}{2}$ in. in places from 4 to 6 ft. up; east side unaffected by water, but has several vertical cracks in middle section from the fire test. Estimated minimum area of cross section of concrete not spalled or loose in middle section, 187 sq. in., a decrease of a little more than 25 per cent of the original section. Column expanded 0.28 in. during the fire test and contracted 0.09 in. during the water test.

Load Test.—On the following day the column was loaded to failure at 379 000 lbs.; column failed by diagonal shearing from a point $4\frac{1}{2}$ ft. up on southeast corner to a point $8\frac{1}{2}$ ft. up on northwest corner; no cracking or yielding was noted before the maximum load was applied. (Figs. 88 and 145.)

TEST NO. 112. ROUND VERTICALLY REINFORCED CONCRETE. UPPER SECTION, LIMESTONE; MIDDLE SECTION, MERAMEC RIVER GRAVEL; LOWER SECTION, JOLIET GRAVEL.

Fire Test.—18 mins.—flaking on southeast corner, 6 ft. up. 23 to 25 mins.—flaking in middle section on south and east. 30 mins.—concrete crushed on south, 6 to $7\frac{1}{2}$ ft. up, cracks opening to $\frac{1}{2}$ in. at 32 mins.; loose pieces ready to fall. 31 mins.—vertical crack on southwest $\frac{1}{4}$ in. wide, 4 to $7\frac{1}{2}$ ft. up. 39 mins.—crack on south, 6 to 7 ft. up, open $\frac{3}{8}$ in. 43 mins.—several new cracks $\frac{1}{8}$ in. wide and about 12 in. long in middle section. 51 to 57 mins.—spalling in middle section to depth of $1\frac{1}{2}$ in.; upper and lower sections intact. 60 mins.—gas shut off.

Water Test.—61 $\frac{1}{4}$ mins.—water applied to column; 50-lb. pressure. 62 $\frac{1}{4}$ mins.—concrete washed off, exposing tie on west, middle section. 62 $\frac{1}{2}$ mins.—some concrete fell on south, middle section. 63 $\frac{1}{2}$ mins.—concrete down on south, 2 to 5 ft. up. 64 mins.—reinforcing bars exposed on west in lower 7 ft. 64 $\frac{1}{2}$ mins.—piece fell on west, 7 ft. up. 66 mins.—piece fell on south, 8 ft. up; not much change during past 2 mins. 66 $\frac{1}{4}$ mins.—water off; duration of water test, 5 mins.

After Test.—Concrete off to reinforcing bars from 3 to $8\frac{1}{2}$ ft. up on west and from 3 to 8 ft. up on south; also piece off, 1 by 6 by 12 in., at 12 ft. up, and 1 by 4 by 5 in., at 11 ft. up on west. On north concrete loose to outside of ties, 2 to 6 ft. up and piece off, $1\frac{1}{2}$ by 12 by 6 in., at $6\frac{1}{2}$ ft. up; on east, crack $\frac{1}{2}$ in. wide from 6 to $9\frac{1}{2}$ ft. up; concrete shattered near middle; crack on northeast corner, $\frac{1}{4}$ in. wide, $1\frac{1}{2}$ to $4\frac{1}{2}$ ft. up; several smaller horizontal and vertical cracks; section at middle of column reduced about 30 per cent due to spalled or loose concrete. Column expanded 0.14 in. during the fire test and contracted 0.02 in. during the water test.

Load Test.—On the following day, the column was loaded to failure at 423 000 lbs.; failure occurred by diagonal shearing from 5 ft. up on north to 7 ft. up on south; cracking sounds were heard at about 290 000 lbs., which increased as failure was approached. (Figs. 88 and 145.)

TEST NO. 113. HOOPED REINFORCED CONCRETE. UPPER SECTION, TRAP; MIDDLE SECTION, MERAMEC RIVER GRAVEL; LOWER SECTION, GRANITE.

Fire Test.—9 to 20 mins.—flaking and cracking in middle section, cracks $\frac{1}{4}$ in. maximum, 2 to 3 ft. long. 21 mins.—piece $\frac{1}{2}$ by 4 by 4 in. spalled on southwest, 5 ft. up. 22 mins.—crack on southwest in middle section open $\frac{3}{4}$ in. 23 mins.—flaking and buckling of concrete shell general in middle section. 27 mins.—piece 1 by 4 by 10 in. spalled on southwest, $4\frac{1}{2}$ ft. up. 28 mins.—piece spalled, $1\frac{3}{4}$ by 10 in. on south 4 to 7 ft. up. 34 mins.—buckling of concrete shell in middle section to 2 in. maximum. 37 mins.—upper and lower sections intact except for cracks extending from middle section. 38 mins.—spalling on south, 4 to 8 ft. up; spiral exposed for 8 turns on 10-in. width. 43 mins.—concrete shell on west middle section, standing out 2 in. 43 $\frac{1}{2}$ mins.—spalling on east, 4 to 8 ft. up; spiral exposed for 16 turns on 5-in. width. 46 mins.—cracks on west extending down to 1 ft. from base, are $\frac{1}{2}$ in. wide at 3 ft. up, opening to $1\frac{1}{2}$ in. at 52 mins. 58 mins.—cracks on east in middle section, extending down to 3 ft. above base; finer cracks below. 60 mins.—gas shut off.

Water Test.—61 $\frac{1}{4}$ mins.—water on, 50-lb. pressure. 61 $\frac{1}{2}$ mins.—hose burst; water shut off; concrete stripped to spiral on west from 1 to $8\frac{1}{2}$ ft. up and partly on north and south 2 to 6 ft. up; above this on north and south concrete off to within 1 in. from spiral; aggregate split on plane of spiral. 79 $\frac{1}{2}$ mins.—water again applied. 80 $\frac{1}{4}$

mins.—pieces fell on west, exposing spiral at $6\frac{1}{2}$ ft. up. $80\frac{3}{4}$ mins.—pieces fell on west, exposing spiral in upper section. 82 mins.—one-half of the circumference of the spiral exposed on west up to 9 ft. above base and one-fourth of its circumference exposed above this point; little change during past two minutes except concrete washed out between wires. $84\frac{1}{4}$ mins.—water shut off; total duration of water test, 5 mins.

After Test.—Concrete washed off to spiral all around in middle section, 4 to 8 ft. up. In lower section spiral exposed all around 1 to 4 ft. up, except for strip about 12 in. wide on east. In upper section, spiral exposed for about half of circumference on west, 8 to $10\frac{1}{2}$ ft. up and for about one-fourth of circumference up to 12 ft. above base. Column expanded 0.14 in. during the fire test and contracted 0.045 in. during the water test.

Load Test.—On the following day the column was loaded until failure occurred at 536 000 lbs., $6\frac{1}{2}$ ft. above base; all bars buckled out $1\frac{1}{4}$ in., and the spiral wire broke in tension with reduced section; faint cracking sounds were first heard at about 300 000 lbs.; at 400 000 lbs. the compressive deformation due to a given load increment was about twice that at the start, and at 500 000 lbs. about five times the initial rate. (Figs. 89 and 145.)

(f) UNPROTECTED CAST-IRON COLUMNS

TEST NO. 114. ROUND CAST IRON. VERTICALLY CAST; UNPROTECTED

Fire Test.— $22\frac{1}{2}$ mins.—gas shut off; column glowing low red in lower half; no visible deflection.

Water Test.—23 mins.—water applied to column; 30-lb. pressure. $23\frac{1}{4}$ mins.—column still glowing on north, east, and south. $23\frac{3}{4}$ mins.—still glowing on east. 24 mins.—water off; duration of water test 1 min.; column still glowing in lower third on east.

After Test.—Column unaffected as far as could be seen except for deflection of $1\frac{3}{8}$ in. to west, 4 ft. above base. Column expanded 0.99 in. during the fire test and contracted 0.51 in. during the water test. (See Fig. 46.)

Load Test.—On the following day the column was loaded to failure and supported a maximum load of 527 000 lbs.; pumping was continued, but load fell off gradually until at 428 000 lbs. column broke in two with great violence 6 ft. above base; during the loading the column deflected $\frac{1}{8}$ in. to west at center at 445 000 lbs., and $\frac{1}{2}$ in. at 520 000 lbs. fracture coarse, gray, crystalline; break very jagged. (Figs. 89, 146, and 46.)

TEST NO. 115. ROUND CAST IRON. VERTICALLY CAST; UNPROTECTED

Fire Test.—20 mins.—trace of color visible on column. 30 mins.—gas shut off; column glowing low red entire length; no visible deflection.

Water Test.— $31\frac{1}{4}$ mins.—water applied to column; 30-lb. pressure. $31\frac{1}{2}$ mins.—column still glowing on north, east, and south. $31\frac{3}{4}$ mins.—still glowing on east. 32 mins.—still glowing in lower half on east. $32\frac{1}{4}$ mins.—water off; duration of water test, 1 min.; no color on column.

After Test.—Column unaffected as far as could be seen except for deflection of $\frac{3}{8}$ in. to west, $5\frac{1}{2}$ ft. above base; column expanded $\frac{7}{8}$ in. during the fire test and contracted $\frac{5}{8}$ in. during the water test. (See Fig. 46.)

Load Test.—On the following day the column was again loaded and supported a maximum load of 502 000 lbs.; pumping was continued with load decreasing, until at 348 000 lbs. it was removed to avoid breaking the column; during the test, column deflected $\frac{3}{4}$ in. to east at center at 290 000 lbs., when deflection changed direction; at 450 000 lbs. it was $\frac{1}{8}$ in. east, increasing rapidly to the west, until at maximum load it was $1\frac{1}{2}$ in. west; deflection continued to increase until load was removed, when column recovered a large part of the deflection. (Figs. 89, 145, and 46.)

XII. GENERAL SUMMARY AND DISCUSSION

Included under this head are considerations relative to the quality of the materials of the test columns and their coverings; the general effects of load, fire, and water; the useful limit of the various types of columns as loaded and exposed to fire; test durations, effects of methods of application, and causes of variation in results.

1. CHARACTERISTICS OF COLUMNS AND THEIR MATERIALS

These relate to the structural quality of the test columns and the physical and thermal effects of load and fire.

(a) STRUCTURAL-STEEL COLUMNS

(1) MATERIAL AND FABRICATION.—The structural steel for the test columns was made by the open-hearth process in four mills in different localities, and in point of chemical and mechanical properties came fairly within accepted specification limits. Measurements of area of structural sections indicated general agreement with the nominal or handbook areas within 1 per cent, although a few measured areas differed from the nominal by amounts up to 4 per cent.

The columns were detailed according to standards of current practice, and as furnished fabricated by five companies no defects due to faulty design or fabrication were noted before test or developed as a result of the tests, except that the bearings as received were too uneven to insure uniform distribution of load.

A number of structural-steel columns that had been subjected to fire and water tests in the protected condition without injury to the steel were subsequently stripped of their covering and loaded to failure, the maximum sustained being 30 600 to 37 600 pounds per square inch, which represent ultimate factors of safety from a little less than 3 to over 4 on the computed working loads.

(2) EFFECT OF SLENDERNESS RATIO.—The slenderness ratio $\left(\frac{l}{r}\right)$ within the limits 40 to 80 appears to have little influence on the strength of structural-steel building columns as exposed to fire. In the case of some of the columns of higher slenderness ratio deflections were larger at a given stage of the test than for columns of more rigid section. The columns had flat end bear-

ings, the upper end being fully restrained and the lower end partly restrained by the base angles and anchor bolts.

(3) LATERAL DEFLECTION.—The direction of the lateral deflection before and after failure conformed with the line of least rigidity of the section, except in a few cases where the direction of the deflection was influenced by local heating of portions of the column. Deflections immediately before failure of a little over 2 inches were noted in a few tests, although generally they were between $1\frac{1}{2}$ and 2 inches. Decided lateral deflection did not, as a rule, develop until the point of maximum expansion had been passed.

(4) VERTICAL DEFORMATION.—Steel columns when loaded and exposed to fire expand up to a point where the rate of compression of the metal due to the load becomes equal to or greater than the thermal expansion. The total expansion varied from $\frac{7}{8}$ of an inch for the columns nearly uniformly heated over the full exposed length to $\frac{1}{8}$ of an inch for those subjected to local heating due to failure of portions of the covering.

The measured expansion per unit length up to the point of maximum expansion, as measured over a 37-inch gage length, varied from 0.0044 to 0.0066, the lower values being due mainly to local heating. The unit expansion per °C. rise of temperature, or coefficient of expansion of the columns as loaded and exposed to fire, averaged 0.0000125 (0.000007 per °F.) as taken from room temperature up to the point where decided yielding of the metal was apparent.

(5) LOAD CARRIED BY THE COVERING.—On application of working load, the covering takes portions of the load proportionate to its area and rigidity with reference to the steel. With increase of temperature the higher rate of expansion of the steel causes a larger portion of the load to be transferred to the steel section, this condition continuing up to the point of maximum expansion. Subsequently the compressive yielding of the steel causes load to be again transferred to the covering, the amount depending on the stability and load-carrying capacity retained by the covering materials after the fire exposure.

(6) AVERAGE EFFECTIVE TEMPERATURES.—For the steel columns for which average effective temperature determinations were made (Table 43), the range at maximum expansion was

from 484 to 593° C (903 to 1099° F), with an average of 530° C (986° F). At failure the average temperature range was from 570 to 837° C (1058 to 1539° F), the average of all determinations being 668° C (1234° F). The applied working loads if carried by the steel alone correspond to stresses of 8900 to 14 500 pounds per square inch, as varying for the different structural sections, the average being 11 600 pounds per square inch (Table 41, p. 97).

The lower temperatures given above correspond nearly with those that hold for steel at maximum expansion and at failure under the given unit loads. The higher temperatures obtained in the tests where the covering carried portions of the load, this effect being much less marked at maximum expansion than at failure.

(7) GENERAL CAUSE OF FAILURE.—The failure in the fire test was due in all cases to decrease in mechanical strength of steel with increase of temperature. The temperature required to cause failure depended mainly on the unit load carried by the structural section, although uneven stress distribution as caused by incidental eccentricity of load application, uneven bearings, and deflection of the column, entered as possible modifying conditions. The general or local lateral deflections occurring immediately before failure were due to yielding of the metal and can be considered as failure effects.

The deflection and distortion at failure caused large permanent loss of load-carrying capacity, depending on the amount of the deflection and the rigidity of the section, the remaining strength being estimated at 5 to 50 per cent of that before test.

(b) CAST-IRON COLUMNS

(1) MATERIAL AND MANUFACTURE.—Of the 10 cast-iron columns tested, 7 were cast horizontally and 3 were cast in vertical position. The iron of the horizontally cast columns conformed with published specifications for gray-iron castings in point of chemical and physical properties, the specimens being cut from the ribs in the head section of the columns. The tests made on the iron of the vertically cast columns were too few to be conclusive, but they indicated difference in the strength of the iron of 15 to 20 per cent as between the two ends of the column (Table 9, p. 339).

The wall thickness of the columns differed from the nominal thickness by a maximum of $\frac{1}{4}$ of an inch for the horizontally

cast columns and $\frac{1}{16}$ of an inch for the vertically cast pipe columns, the difference being caused mainly by displacements of the molding core in casting the columns.

Two of the vertically cast columns that had been subjected to fire and water tests, which induced permanent lateral deflections in one of $\frac{7}{8}$ inch and in the other of $1\frac{3}{8}$ inch, were subsequently loaded to failure, the maximum sustained being 36 400 and 34 700 pounds per square inch, respectively.

(2) DEFORMATION AND TEMPERATURE.—The same general characteristics of deformation and temperature obtained for the cast-iron as for the steel columns. The average unit expansion attained by the cast-iron columns was 0.0064 against 0.0054 for the steel columns, the average temperature at maximum expansion and at failure being about 70° C (126° F) higher than as obtained with the steel columns, the difference being due to the lower allowable working loads applied to the cast iron. The average applied unit loads, if assumed carried by the metal section alone was 6500 pounds per square inch as compared with 11 600 pounds per square inch average for the steel columns, a difference that was considerably reduced by the interaction of the coverings, which were heavier around the steel columns and generally carried larger proportions of the applied loads than those of the cast-iron columns.

(3) CAUSE AND CHARACTER OF FAILURE.—Failure in the fire test was primarily due to inability of the metal to sustain load at the given temperature, the bucking and fracture incident with failure being in the nature of failure effects. The partial or full fracture of the metal section at one or more points, resulted in almost complete loss of load-sustaining capacity. The direction of the deflection for the columns having uneven wall thickness was quite uniformly toward the side with the heavier thickness, due evidently to compressive yielding of the thinner and more highly stressed metal on the opposite side.

(c) PIPE COLUMNS

(1) MATERIAL AND MANUFACTURE.—No tests of the metal of the pipes were obtained, but the material was apparently mild steel. The metal was of standard thickness. Cylinders taken of the 1:1½:3 concrete mixed for the filling of the plain pipe

column developed an average compressive strength of very nearly 4000 pounds per square inch at the time the column was tested.

The columns were filled at the manufacturer's plant and furnished complete with bearing details, the latter being arranged to approximate conditions of use in buildings (Fig. 7, p. 24).

(2) DEFORMATION AND TEMPERATURE.—The metal being exposed, attained early in the fire test a higher temperature than the filling, and expanding away from the latter, would assume most of the applied load, the unit stress, if all of the load was carried by the pipe metal, being 16,500 pounds per square inch for the plain concrete-filled pipe column. The point of maximum expansion was in this case well defined and was attained at an earlier period and lower temperature than in tests of steel and cast-iron columns of about the same duration, due to the higher load sustained by the metal. The compressive deformation subsequent to maximum expansion was large and developed the strength of the concrete filling before failure occurred.

The reinforced-pipe column which had structural angles embedded in the concrete filling, expanded only a small amount during the fire test, yielding of the pipe metal under the high induced stresses beginning early in the test. Toward the end of the test the temperature of the pipe was so high that it could have sustained only a small part of the load, which must have been carried mainly by the steel reinforcement with stresses too high to permit much expansion.

Lateral deflections began at about the time the pipe metal began to yield and increased as failure was approached.

(d) REINFORCED-CONCRETE COLUMNS

(1) MECHANICAL PROPERTIES OF THE CONCRETE.—Made under conditions approximating those obtaining in building construction, the concrete developed wide variability in strength and elastic properties, the principal cause of which was difference in water content of the concrete mixture. The average compressive strength of 8 by 16 inch cylinders of 1:2:4 limestone and of trap-rock concrete made from the concrete mixed for the reinforced-concrete columns was 1525 pounds per square inch at 28 days and 1930 pounds per square inch at 16 months, with maximum variations above and below the averages of 67 and 54 per

cent, respectively. The modulus of elasticity at 650 pounds per square inch had an average value of 3 080 000 pounds per square inch with concrete aged 16 months, its variability being somewhat greater than that of the compressive strength. This variability in strength and elastic properties appears to have had little influence on the fire resistance of the concrete.

(2) DEFORMATION AND TEMPERATURE.—Maximum unit expansion of 0.0023 to 0.0046 were observed in fire tests of reinforced-concrete columns, the average being about one-half of that found for cast-iron columns. The indications are that a considerable portion of the expansion was due to the vertical steel reinforcement, maximum expansion being coincident with temperatures in the reinforcement of 400 to 500° C, in which region yielding of the metal takes place under stresses that were likely to be induced in the bars by their higher expansion rate with reference to the concrete. With limestone or trap-rock concrete and with lateral ties not less than 12 inches apart no tendency for the vertical bars to buckle before failure was noted.

Following maximum expansion, the columns gradually compressed, the rate being less rapid than for steel and cast-iron columns (Figs. 169 to 171). Temperatures at failure in the center of the trap-rock concrete columns that failed in the fire test averaged 450° C (842° F), and in the vertical reinforcing bars 894° C (1641° F).

(3) CHARACTER OF FAILURE.—The columns failed locally by compression or by combined compression and shearing on inclined planes, the vertical bars buckling and lateral ties or hoop-ing, breaking, or yielding at the point of failure. No large lateral deflections developed either before or at failure.

(e) TIMBER COLUMNS

(1) QUALITY OF MATERIAL.—The timber was long-leaf pine and Douglas fir of select structural grade and conformed with the requirements of published specifications for structural timber (Table 2).

(2) DEFORMATION AND TEMPERATURE.—Except for slight expansions noted during the first few minutes of the test, the timber contracted or compressed under the combined load and fire condition, most of the deformation occurring in the wood at its bearings on the metal cap introduced near the top of the column, the crushing of the wood at these points causing progressive depression of the top of the timber columns (Fig. 47, p. 126).

The temperature in the timber away from the surface and the cap bearings was retarded by the evaporation of moisture and the low heat conductivity of the wood and did not exceed 100°C (212°F) until near failure (Figs. 140 and 141). In the metal cap at the edge of the column bearing the temperatures at failure varied for the different tests from 432 to 544°C (810 to 1011°F) (Table 43, p. 122).

(3) CAUSE OF FAILURE.—The cause of failure of the timber columns was loss of strength of the wood at the cap bearings, due to conduction of heat from the flanges of the metal caps to the bearing plates and into the wood. The consequent softening of the wood caused the columns to slip laterally on their bearings with the steel-plate caps and to fracture the cast-iron caps. While the test fire reduced the area of the columns by 29 to 55 per cent their resistance to fire and load outside of the bearings was not fully developed in the tests.

2. USEFUL LIMITS

The useful limit point with reference to fire exposure of columns is here taken as the limit of the period within which the effects on the elements designed to carry load do not permanently impair their essential structural properties to such extent as to make them unsuitable for further use.

(a) STRUCTURAL-STEEL AND CAST-IRON COLUMNS

The test characteristics indicate that for columns restrained at the ends and with slenderness ratio not exceeding 80, the useful limit is approximately equal to the period of expansion as given in Table 43 (p. 122) and Fig. 45. The measured center deflections of columns within the given $\frac{l}{r}$ limit did not exceed $\frac{1}{2}$ inch at the end of the expansion period. The deflection of some of the columns of higher slenderness ratio was larger, although at the given stage all measured deflections were within 1 inch. Since failure in details such as riveting was not apparent in any test, unfavorable conditions in this respect could not have existed at any time preceding failure.

Recent tests made by the Bureau of Standards on specimens of structural shapes that were heated under load up to the point of maximum expansion and cooled indicate little or no distortion of shape, loss in strength, or change in elastic properties. The compressive set on cooling and removal of load was not large enough to be objectionable as applied to a building column.

In the case of the structural-steel columns with 2-inch Chicago limestone or Joliet gravel concrete coverings the interval between maximum expansion and failure was equal to 73 per cent, or more, of the expansion period, and, considering the consequent lower rate of deformation of the column, the useful limit may possibly extend considerably beyond the expansion period. The same applies to the columns with 4-inch concrete coverings made with the Chicago limestone aggregate, which withstood the fire test in excess of 8 hours.

Damage to the covering is not considered in the above discussion, as it is commonly not regarded as a load-carrying element.

(b) PIPE COLUMNS

For the unreinforced pipe column the useful limit can be taken as the period of expansion in the fire test.

For the reinforced pipe column the transfer of load, as caused by successive heating and expansion, first to the pipe and later to the structural steel reinforcement, confounds the relation existing between maximum expansion and useful limit. The temperatures on the outside of the pipe indicate that the useful limit was reached after between 20 and 30 minutes of fire exposure. The center lateral deflection at the end of this interval was $\frac{3}{4}$ of an inch.

(c) REINFORCED-CONCRETE COLUMNS

The reinforced-concrete columns expanded less than the steel and cast-iron columns, reached their point of maximum expansion at a relatively earlier stage, and the subsequent compressive deformations and center deflections were smaller. Comparisons of the temperatures of the reinforcing bars and the attending unit deformation indicate the presence of high compressive stresses, although with concrete of the given aggregates and the methods of tying employed no tendency to buckle before failure was noted. For the columns that withstood the 8-hour fire test and sustained large additional loads before failure it is believed that the useful limit extended up to the end of the fire test. In the case of the columns that failed in the fire test or sustained but small additional load after 8 hours the matter is more indeterminate, although it can be stated that large compressive deformations did not obtain until within the $1\frac{1}{2}$ hours preceding failure.

These considerations do not preclude damage to the outer concrete covering which it might be necessary to repair or replace after fire exposures within the useful limit.

(d) TIMBER COLUMNS

The effect that limits the usefulness of timber columns after fire exposure, as far as it concerns the timber, is surface damage from burning away of the wood, with consequent reduction of effective section. Large compressive deformation of the column due to impairment of the unburned wood from the heat apparently obtains only for points nearer failure. The amount of reduction in area that can be allowed depends on the basis of design. In common with general practice, the whole area of the test columns was assumed to carry load, no portion being deducted as protection. Assuming that the initial factor of safety was no more than adequate, minor reductions in area would render the column unfit for further use. Practical requirements relative to minimum and standard sizes often result in loadings lower than the safe working limit, in which case proportionate reductions in area due to fire could take place without impairing the safe load-carrying capacity.

With reference to fire effects on unprotected steel and cast-iron bearings, the rapid rates of deformation obtaining after 20 minutes in tests of unprotected timber columns indicate that the useful limit in this particular did not extend much beyond this period (Fig. 47, p. 126). For the protected timber columns the useful limit in this respect extended to within about the same time interval (15 to 30 minutes) before failure, as in the tests of unprotected timber columns.

(e) PRACTICAL APPLICATION

The above conclusions relative to useful limits find application where it is desired to design and protect columns so they will not be materially injured by a fire that consumes the contents of the structure supported by them. In applying the results of these tests for this purpose it is necessary to make deduction for variations in material and workmanship not developed in the tests. The percentage reduction can be safely taken as that used in deducing ultimate fire resistance periods in the concluding section of this paper. The period of expansion is influenced to less extent by incidental conditions, such as load-carrying capacity of covering materials, than is the time to failure.

While the useful limit may be made the basis of design of columns and coverings in special cases, it is believed that the most general use of the test results will be that based on the ultimate fire-resisting point; hence no detailed application of the former will here be made.

FIG. 48.—Results of fire tests compared by groups

3. DISCUSSION OF TEST DATA

The considerations herewith given relate to test duration, period of expansion, and the causes immediately affecting them. Comparisons of the time to failure of columns in the fire-test series arranged by groups are given in Fig. 48, which also provides a measure of the variation obtaining within each group. Further comparisons of time to failure and period of expansion are given in Table 43 (p. 122) and Figs. 42 (p. 114) and 45 (p. 121). In the discussion the time periods will be given to the nearest minute.

(a) DIFFERENCE IN COLUMNS, TEST EXPOSURE, AND SERVICE CONDITIONS

(1) VARIATIONS DUE TO DIFFERENCE IN COLUMNS.—Among the effects tending to produce variation in results with given types of protection for the same thickness of covering are size and shape of structural section and consequent differences in intensity of loading and resistance to fire; differences in fire-resisting properties of subclasses of covering material within each group, such as concrete of the different aggregate combinations and hollow clay tile of the various types of clay; variations in methods and details of application; incidental variations in material and workmanship. These are discussed in connection with each group whenever the test results developed information on any given particular.

A fairly wide range of structural section was employed in the tests, and it is believed that the influence of this factor was sufficiently determined.

One or more representative materials from each of the main subclasses of covering material were introduced. It is appreciated that considerable differences in mineral composition and structure may exist between material in a given subclass as occurring in different localities, and, while the difference in fire-resisting properties may be of minor order, wide application of the results of the tests will have to be made with care until the knowledge on the subject has been extended by further tests.

The full range in results due to difference in methods of application and in incidental variations in materials and workmanship was not developed in the tests, as this required a greater number of test duplications than could be introduced. In general, poured coverings are subject to smaller incidental differences than those built

up from small units; also, if stability is assured, a large element of variability is eliminated. It is also thought probable that columns and protections in buildings are subject to greater variability than those constructed for the tests, the latter representing workmanship of more uniform quality.

(2) VARIATIONS DUE TO DIFFERENCE IN LOAD AND FIRE CONDITIONS.—In the tests the amount of load applied on a given column section or column type was subject to minor variations only, the errors being generally within 1 per cent. Comparisons of the initial deformations produced in opposite flanges of individual columns indicate maximum outer-fiber stresses in several tests up to 25 per cent above what would obtain for uniform distribution of load. These can be ascribed mainly to bending stresses induced by uneven bearings and are no larger than those generally incident with compression tests or with normal loading of columns in buildings.

The loads applied in the tests were about 10 per cent higher than those derived from the column formula employed. While floors in buildings are subject to overload by more than this amount, the resultant load on the columns is likely to be reduced by the distribution of load concentrations to several columns and by the equalizing effect of loadings from several stories.

As exposed to fire in buildings, individual columns may take added load due to higher temperature, and consequent greater expansion, than the adjacent columns of the story, particularly if they are not fully protected. For steel or cast-iron columns the additional load assumed is limited by the ultimate flexural strength of their connections to the floor beams framing into them. In the case of interior columns the additional load possible to assume on this basis will seldom exceed 20 per cent of the nominal floor loads. Reinforced-concrete columns have more rigid floor connections than steel or cast-iron columns, but, due to their lower expansion and rate of temperature rise, more unfavorable loading conditions as due to unequal expansion are not likely to occur.

The furnace exposures are given in the test results as percentages of the average of all tests, the quantity compared being the area under the average time-temperature curve of each test. For the shorter tests with upper limit of duration of about one hour the difference in exposure may have had a considerable influence on results, considering also that the percentages given do not indicate

the full measure of the difference due to lag of the pyrometers. For the longer tests, variations in test results due to difference in furnace exposure were apparently of minor importance.

Comparing the average time-temperature curve of the column tests with the reference curve (Fig. 39) a fair agreement is seen to have been attained. The reference curve was adopted as being consistent with furnace exposures used in previous fire tests, and there was at the time no distinct understanding whether it should represent indicated temperatures or temperatures corrected for lag and radiation effects, little information on the extent of these effects being available. Investigations conducted subsequent to the completion of the column tests disclosed large effects due to lag during the initial period of the fire exposure and smaller, but more persistent, effects due to radiation (p. 101-106, Figs. 40 and 41). Any time temperature relation adopted as standard is necessarily more or less arbitrary, particularly for the initial period. Temperatures attained in fires afford some guidance, although they indicate a wide range in intensity. From the information available, points on the reference curve, considered either as indicated or as corrected temperatures, while not the maximum attained in fires of exceptional intensity, represent severe fire conditions.

(b) UNPROTECTED COLUMNS

Included under this head are structural steel, cast-iron, and pipe columns that were tested without protective coverings, all parts of their sections being assumed to carry proportionate portions of the applied load.

(1) STRUCTURAL-STEEL.—The time to failure of the unprotected structural-steel columns varied from 11 to 21 minutes (Table 42a, p. 108). The difference in results for the various column types, while due in part to variation in furnace exposure, is attributable also to difference in thickness of metal and in the unit loads sustained, sections with thin members under the higher unit loads failing sooner than sections whose members were arranged to form heavy metal thickness. Fig. 49 is a plot between unit load sustained, and time to failure in tests of unprotected structural-steel columns, showing the inverse relation obtaining between them. The variation in the unit load applied was due to difference in slenderness ratio between the different sections.

FIG. 49.—Effect of load on fire resistance, unprotected structural-steel columns

The average time to failure of the eight structural-steel sections was 15 minutes and the average period of expansion was 13 minutes (Table 43). Maximum temperatures from 578 to 668° C (1072 to 1234° F) were attained on the outside of the metal near the edges. It is difficult to estimate the average effective temperature, since the rise was too rapid to permit assuming temperature uniformity over the thickness of the metal. Tests recently made by the Bureau of Standards on small specimens indicate that for the loads sustained by the unprotected columns the failure temperatures of the structural steel fell within the limits 550 to 650° C (1022 to 1202° F).

(2) CAST-IRON.—The average time to failure of the unprotected and unfilled cast-iron columns, one of which was tested with ends restrained and two with unrestrained ends, was 34 minutes, the variation being less than one minute for the three tests. The periods of expansion varied from 22 to 24 minutes. The longer test periods and higher temperatures attained as compared with the structural-steel columns can be attributed largely to the lower allowable unit loads applied to cast iron and the relatively smaller surface exposed to the fire. As judged by the direction of the lateral deflection, failure in compression appears to have started on the side having the thinnest metal. Filling the interior with concrete (test No. 11) increased the time to failure 11 minutes, with a smaller proportionate increase in the length of the expansion period.

In the two fire and water tests of cast-iron columns, water was applied when the columns had attained maximum expansion, the metal being at low red heat. No cracks developed in the metal, the only effect being permanent lateral deflections of, respectively, $\frac{3}{8}$ and $\frac{7}{8}$ inch toward the side on which water was applied.

(3) PIPE COLUMNS.—The time to failure of the 7-inch pipe column was 36 minutes and maximum expansion was attained at 14 minutes.

The 8-inch reinforced pipe column failed after 1 hour 12 minutes, and maximum expansion was attained at 52 minutes. The column expanded only a small amount after 25 minutes, the total expansion being about $\frac{1}{4}$ of an inch. The high temperatures on the surface of the pipe indicate that during the last half of the test period most of the load was carried by the structural-steel reinforcement.

(c) PARTLY PROTECTED COLUMNS

Filling the reentrant portions or interior of the structural-steel sections with concrete greatly increased their fire-resisting property as compared with that in the unprotected condition (Table 42*b*). The difference was due to slower temperature rise in the metal resulting from the heat insulating and absorbing properties of the filling and to the load-carrying capacity of the concrete, the longer time intervals between maximum expansion and failure being due in great part to the latter factor.

Temperature differences up to 450° C (810° F) existed between the exposed flanges and the protected webs, the temperature of the latter at failure being generally below 500° C (932° F).

(1) EFFECT OF SECTION AND SIZE.—No decided effect due to variation in shape of structural section was noted except as it affected the size of the column.

In Fig. 50 is shown the relation between time to failure and area of material in cross section for all tests of partly protected columns, except No. 22, which was eliminated as being in effect a protected column. The relation shown accounts for the variation in results within the group as being due in large part to difference in size.

(2) EFFECT OF CONCRETE AGGREGATE AND TIES.—In the tests of short duration the concrete aggregate appears to have affected

FIG. 50.—Effect of size, partly protected columns

results to a minor extent only, while in those of longer duration (Nos. 18 and 22) the results compare with those obtained for concrete protections, which they more nearly resemble.

Metal ties were placed in the concrete of all partly protected columns where not contained by the section members as in the latticed columns. No tendency was noted on the part of the filling to spall, buckle, or otherwise come loose before failure. The extent to which the ties functioned to prevent such effects is not fully determinate, although in tests of concrete protections made with the same aggregate little cracking or spalling was noted within the given time periods.

(d) PLASTER ON METAL-LATH PROTECTIONS

(1) MATERIAL AND DESIGN.—The Portland-cement plaster, of proportion 1 : $\frac{1}{10}$: $2\frac{1}{2}$ volume parts of Portland cement, hydrated lime, and coarse lake sand, as mixed by the workmen for the protections and tested in 2-inch cubes, developed an average compressive strength of 1677 pounds per square inch at 28 days and 2623 pounds per square inch at an average age of $16\frac{1}{2}$ months, with maximum range in individual tests of 44 per cent below and 60 per cent above the given averages (Table 25, Fig. 23). Cubes made in the laboratory of the same materials and average water content (16.7 per cent) gave lower compressive strength, both as stored in air and in water (Table 28, p. 356).

The metal lath was wrapped around the structural-steel columns on bars or on pressed-steel channels acting as spacers, and the plaster applied in layers $\frac{7}{8}$ to $1\frac{1}{8}$ inches thick, each consisting of two body coats. The double-layer protections had a $\frac{3}{4}$ -inch air space between layers. For the cast-iron column the plaster was applied on high-ribbed metal lath supported directly on the column. A broken air space of about $\frac{1}{2}$ an inch thickness was formed next to the metal, due to failure of the plaster to fully fill the space back of the lath.

(2) TEST RESULTS.—The average time to failure of the structural-steel columns with single layer protection was 1 hour 16 minutes, and of those with double-layer protection 2 hours 38 minutes, the two tests of each varying from the average by less than 15 minutes (Table 42c, p. 109). The cast-iron column protected by a single layer of $1\frac{1}{2}$ -inch average thickness with a broken air space between it and the iron (test No. 27) stood up longer by a few minutes than any of the other columns in the group.

The temperature distribution across the section was generally very uniform, and the variations in the length of the column were not large (Figs. 97 and 98). This was due to the air space between the covering and the structural section, which permitted free heat interchange in the column, unmodified by the temperature gradients in the covering material.

(3) CRACKING DUE TO EXPANSION OF COVERING.—During the first 20-minute period in all tests of Portland-cement plaster protections, cracking and disruption of the plaster took place below the bracket near the top of the column. This was evidently due to expansion of the plaster layer, which was restrained at the top of the column and at the bottom bearing. This effect appears to have had little influence on the time to failure in the given tests, the region of maximum column temperature and failure being in all cases within the middle 4 feet of the column height.

The sand used in the plaster was high in insolubles (chiefly silica) and low in calcite and dolomite (Table 14, p. 342).

(4) EFFECT OF VARIATION IN DETAILS OF APPLICATION.—Comparing the layer thickness of test No. 23, where the plaster was applied on expanded metal lath, with that of test No. 24, where woven-wire lath was used, and also the outer layer thickness with the inner in test No. 110, applied, respectively, on expanded metal and on woven wire, a heavier layer thickness by $\frac{1}{8}$ inch was attained in all cases with the expanded metal (Tables 3*d* and 4*d*). This difference in layer thickness may account in part for the longer test duration of No. 23 as compared with No. 24. Further indications that layer thickness is an important element in the protection given is had in the case of tests Nos. 25 and 26, where, with a difference in layer thickness of $\frac{1}{8}$ inch, a difference in time to failure of 16 minutes obtains.

The tests developed no evidence that the method of supporting the lath had any influence on results (cf. Sec. IV, par. 2*a*, p. 56).

All double coverings had an air space about $\frac{3}{4}$ of an inch wide between the inner and outer layers. No tests were made with a single layer equivalent in thickness to that of two double layers, hence no direct evidence relative to the value of the air space as an insulating medium was obtained.

(5) EFFECT OF WATER APPLICATION.—The water carried away some loose pieces near the top of the column where cracking and spalling had taken place during the fire period (Fig. 87). It exposed the lath at the corners of the outer layer for portions of the height. The test did not appear to have materially injured the insulating value of the covering since the time to failure in the subsequent fire test nearly equaled that of the corresponding test (No. 23) in the fire series (Table 44, test No. 110).

(c) CONCRETE PROTECTIONS

(1) MECHANICAL PROPERTIES OF THE CONCRETE.—The compressive strength of 8 by 16 inch cylinders made from 1:2:4 gravel or crushed-stone concrete mixed for the column coverings under conditions approximating those of building practice, averaged 1520 pounds per square inch at 29 days and 2100 pounds per square inch at an average age of 15 months, the maximum range of individual test results being 118 per cent above and 57 per cent below the given average values (Table 21, p. 346). As shown in Fig. 15 (p. 66), the range in results increased with the number of tests in the group, indicating that in the groups with the smaller number of tests the full possibility of variation was not developed.

The principal cause of the variation in strength was apparently difference in consistency of the concrete mixtures. Increasing the time of mixing from one minute to two minutes gave an indicated increase in the average compressive strength of 45 per cent (Fig. 19, p. 69).

The modulus of elasticity of the concrete varied approximately with the compressive strength from less than 1 000 000 to over 4 000 000 pounds per square inch (Fig. 21). This large variability in the mechanical properties of the concrete appears to have had little influence on its fire-resistive properties, the latter depending chiefly on the mineral composition of the aggregates employed.

(2) FUNCTION OF CONCRETE AS A COVERING MATERIAL.—Concrete applied as a protective covering or filling to steel or cast-iron columns retards the temperature rise in the metal when the column is exposed to fire and further retards the failure by carrying portions of the column load proportionate to its relative area and rigidity as compared with the metal.

The protections were applied as square or round coverings, generally 2 inches and 4 inches in thickness as measured from the surface of the covering to the metal. The time to failure in the fire tests varied from 1 hour 47 minutes to 7 hours 57 minutes for the 2-inch protections, and from 3 hours 41 minutes to over 8 hours for the 4-inch protections (Table 42*d*, p. 110).

(3) VARIATIONS DUE TO CONCRETE AGGREGATE.—With a given thickness or size of covering the main cause of variation in results was the difference in fire-resisting properties of concrete made with different aggregates. In this particular the concrete can be placed in three groups. That giving the most unfavorable results was the concrete made with Meramec River sand and gravel, a number of large cracks forming early in the tests followed by spalling of large and small pieces of concrete not held by the ties (test Nos. 39 and 45). This sand and gravel consist almost wholly of quartz and chert grains and pebbles, the gravel having a particularly high chert content. Both minerals are forms of silica (SiO_2), the quartz being crystalline and anhydrous, and the chert amorphous, with a variable amount of water in chemical combination. On being heated part of the combined water in the chert is liberated, and the consequent vaporization disrupts the pebbles. Other causes of disruption of concrete made with

siliceous aggregates are abrupt volume changes, points of which are known to exist for chert as low as 210°C (410°F). Quartz has a decided point of abrupt volume change at 573°C (1063°F), where it is transformed into the mineral tridymite, the change extending over a considerable temperature range when the heating is rapid. Liquid inclusions contained in small cavities formed when the rock crystallized from the molten condition may be the cause of some of the cracking incident with fire exposure.

The middle group includes concrete made with trap rock, granite, sandstone, and hard-coal cinder.

In tests with trap-rock and cinder concrete a small amount of cracking developed during the last part of the fire period, but no spalling of note occurred before failure. In the granite-concrete protections the cracks developed earlier in the test, and portions of the corners spalled off during the last 30 minutes of the test period. In the tests with sandstone-concrete protections cracking and spalling of corners outside of the wire tie began in the first 30-minute period and continued during the next hour, after which there was little apparent change before failure. The spalling exposed portions of the flange edges, which to some extent hastened the failure. The average time to failure in tests with sandstone-concrete protections was intermediate between those with trap-rock and those with cinder concrete. The cracking of sandstone concrete after a short fire exposure can be ascribed mainly to the abrupt volume change of the constituent quartz grains as noted above.

Fusion of the trap-rock concrete occurred where the tests extended beyond 7 hours, the concrete being affected to a depth of about $1\frac{1}{2}$ inches. Flowing of concrete due to fusion, while not general, occasionally formed pockets up to a 2-inch depth. Incipient fusion to about the same depth occurred in the 4-inch granite-concrete protections, although no actual flowing of concrete took place.

The third group comprises protections of Chicago-limestone concrete and Joilet-gravel concrete. The composition of this gravel is similar to that of the Chicago limestone, and the fire-resisting properties of the concrete made with each compare quite closely. Very little cracking resulted on exposure to fire and their heat-insulating value was increased by the change of the calcium and magnesium carbonate to the corresponding oxides.

This process retarded the flow of heat through the region of change and left material of good insulating properties. Immediately after test the surface of the concrete was firm, but after a few weeks exposure the hydration of the oxides caused slaking and crumbling of the calcined material (Fig. 62, p, 221).

(4) COMPARISON OF 2 AND 4 INCH PROTECTIONS.—In the comparison given in Fig. 51, the tests of concrete protections are arranged by groups as defined in the preceding paragraph, the line in the case of the middle group connecting the average value for each thickness. Test Nos. 40, 46, and 47 are omitted in this comparison on account of extreme shape and size of section, and No. 44 on account of leaner concrete mixture.

FIG. 51.—Comparison of 2-inch and 4-inch concrete protections

The time to failure under working load was not determined for the 4-inch limestone-concrete protections, as they were loaded to failure after withstanding the fire test in excess of 8 hours. They all attained maximum expansion within the last 30 minutes of the 8-hour fire period, and from comparison with results obtained with the corresponding 2-inch protections, failure in the case of the 4-inch protections would not have taken place before the end of 10 hours, assuming the same load and the same furnace temperature rise as obtained during the 8-hour period.

(5) EFFECT OF SIZE.—In Fig. 52 the time to failure in tests with concrete protections of the middle group is plotted against the area of steel and concrete in the cross section. Variations

FIG. 52.—Effect of size, concrete protections

from the general trend can be accounted for as due to concrete aggregate, proportions of concrete mixture, type of section, and to incidental differences in test conditions and test columns. The extreme variation in furnace exposure as measured by the area under the furnace temperature curves is within 2 per cent as between all the tests plotted, except Nos. 34 and 34A, between which there is a difference of 4 per cent (Table 42d), which latter may be responsible for the difference in failure time of the two tests.

(6) EFFECT OF STRENGTH OF CONCRETE.—Some decrease in fire resistance due to leaner mixture may be noted by comparison of Nos. 33 and 33A with No. 35, and No. 43 with No. 44.

No evidence was developed that variation in the strength of the concrete of the same aggregate and proportion of mixture had any appreciable influence on the results of fire tests of concrete protections. This was due to the large change in mechanical properties produced by the heat. Concrete as made with different aggregates preserves strength to different degrees on exposure to fire. This had a decided influence on results, the longer test periods, and particularly the longer intervals between maximum expansion and failure of the limestone concrete and Joilet-gravel concrete coverings, can be attributed in a great part to this cause.

(7) INFLUENCE OF SHAPE OF SECTION AND COVERING.—As in the case of the partly protected columns, the only well-defined effect of change in shape of structural section was primarily due to the resulting difference in the cross-sectional area of the covered column.

Round and square coverings of trap-rock concrete displayed only minor differences in the number and size of cracks that developed before failure. Fine vertical cracks 2 to 4 inches from the corners formed in the square coverings at somewhat earlier periods than the first cracks noted in the round coverings. In neither case do these cracks appear to have had any appreciable influence on the time to failure. Coverings made of concrete more subject to cracking may possibly develop greater differences due to shape, although with concrete made with highly siliceous aggregates, the influence of the aggregate is so large that other effects are small in comparison. This is shown in test No. 45, where the round, siliceous-gravel concrete covering sustained severe cracking and spalling early in the test which caused failure over one hour earlier than in any other test of concrete protection.

(8) FUNCTION OF THE WIRE TIE.—In all tests of concrete protection, except Nos. 28A, 33A, and 47, the concrete was tied by a wire tie wound spirally around the structural-steel section. In Nos. 28A and 33A the concrete aggregate was Chicago limestone. No cracking of consequence developed before the end of these tests, and the absence of the tie had no influence on the results. In the case of the cinder-concrete protection in test No. 47 no cracking of note occurred until near failure and after the column had sustained large compressive deformation. About two minutes before failure most of the covering fell off. This would have been prevented if the wire tie had been present, although the temperature of the metal and the deformation of the column was such that failure was imminent.

It is not possible to state the extent to which the tie functioned in all tests of concrete protection, but it was undoubtedly of value where there was any tendency for the concrete to crack and fall off before failure. In the case of Meramec River gravel concrete and sandstone concrete, spalling of portions of the covering outside of the tie occurred early in the test, the concrete on the middle of the flanges and webs being apparently held by the ties.

(9) EFFECT OF WATER APPLICATION.—In the three tests where the wire was placed in the coverings the water pitted the exposed faces of the concrete and carried away portions of the corners and sides, leaving parts of the flanges and flange edges exposed. The damage was most marked in the regions where cracking was noted during the fire period (Figs. 83 and 84, p. 242-243).

In the case of the one covering that was not tied the water loosened or carried away most of the protection on the flanges, leaving the column in the condition of partial protection. Most of the damage was incurred after 2 minutes of water application, the total period being 5 minutes.

(f) HOLLOW CLAY-TILE PROTECTIONS

(1) MECHANICAL PROPERTIES OF THE TILE.—The average compressive strength of specimens of the hollow clay partition tile used in the column coverings was 5350 pounds per square inch as tested on end and 4370 pounds per square inch tested on edge, the maximum variation above these values being 134 per cent and below 78 per cent, of the lower average value. For the same type of clay the range in results was smaller. The strength was generally proportional to the density of the tile as indicated by percentage of porosity and of absorption (Table 31, p. 358-359).

In transverse tests of hollow tile the average computed outer fiber stress at failure was 527 pounds per square inch and the shear 233 pounds per square inch, the failure being apparently due to combined shear and tension. The range in results was larger than in the compression tests (Table 32).

There appears to be little relation between the mechanical strength and the fire-resistive properties of the tile, the latter depending mainly on the type of clay. This is significant as specifications based on the mechanical properties often disqualify tile desirable from the standpoint of resistance to fire.

(2) TEST RESULTS.—In tests of hollow-clay tile protections, using tile of the given types of clay applied according to the methods previously described, the time to failure ranged from 50 minutes to 4 hours 42 minutes. This large range in results was due to a number of factors that influence the effectiveness of this type of protection, including, besides type of clay, methods of manufacture of the tile, thickness of shells and webs, the presence or absence of concrete or other filling, and the methods used for tying the tile (Table 42e, p. 111).

(3) VARIATIONS DUE TO TYPE OF CLAY AND DETAILS OF MANUFACTURE.—The extent to which the tile cracked and spalled on exposure to fire varied with the type of clay of which it was made and the degree of hardness to which it was burned. All of the tile used in the coverings was straight, nonporous partition tile, burned without sawdust or other filling, except the round tile in test Nos. 62 and 63, which was porous.

The semifire-clay tile generally gave the most favorable results in the fire tests, and of the two represented the tile of medium hardness and having heavier webs and shells developed few cracks and little spalling before failure. In the coverings of round, porous semifire-clay tile a number of vertical cracks formed after a short fire exposure, which became wider as failure was approached. Few other disruptive effects were noted, almost all material remaining in place till the end of the test. Only minor differences in behavior were noted between tile made of the two kinds of surface clay, cracking and spalling of outer shells and buckling out of the tile being characteristic of tests of both. In the case of shale tile, these effects were even more pronounced, severe cracking taking place during the first few minutes of the test, followed by general spalling of outer shells.

(4) COMPARISON OF 2 AND 4 INCH PROTECTIONS.—In Fig. 53 is given a comparison of time to failure in tests with 2 and 4 inch

FIG. 53.—Comparison of 2-inch and 4-inch hollow clay-tile protections

hollow clay-tile protections, all other details being comparable except as noted. The tests show little difference between the two, the thickness of the air space and minor variations in thickness of shells having apparently little influence on results. The difference in results in test Nos. 50 and 50A as compared with 51 and 51A can be attributed to larger differences in thickness of shells and webs and also to the greater stability of the 4-inch tile set on end and tied with outside wire ties, as against the 2-inch tile laid flat in 6-inch courses without ties (p. 137-138).

(5) EFFECT OF SIZE.—In Fig. 54 is shown the relation between the time to failure in all tests of hollow clay-tile protections made with nonporous partition tile and the total area of solid material

FIG. 54.—Effect of size, clay-tile and brick protections

in the cross section, a fairly consistent variation of the one with the other being evident. Nos. 58 and 59, having double layer of tile with tile filling, gave less favorable results in this comparison than tests of protections with a single layer of tile and concrete fill.

(6) EFFECT OF TIES AND FILLING.—Fig. 55 gives a comparison of results attained with the two methods used for tying the tile and of including or omitting the concrete or tile filling. As shown on the diagram and confirmed by test characteristics, the mesh in the horizontal joints is a little more effective in holding the tile than the outside wire ties; also the concrete or tile filling is an important element in the protecting property of a hollow-tile protection.

The efficiency of wire mesh in the joints as against outside wire ties, in the case of protections of surface-clay tile with hollow tile filling, is shown by comparison of results in test Nos. 58 and 59. It should be considered in this connection that minor parts of the difference may be due to incidental variations in workmanship and test conditions, and also that protections of tile less subject to cracking and disruption on exposure to heat would show relatively smaller differences in results, due to the methods used for tying the tile.

The concrete filling not only serves as a protecting medium, but also assists in holding the tile in place by adhesion. In test

FIG. 55.—Effect of ties and filling, hollow clay-tile protections

No. 60 the concrete filling was placed before the tile was set, the tile being bonded to the filling with a thin mortar joint and tied with outside wire ties. A large number of tile units fell off early in the test, the behavior in this particular being distinctly different from that of protections with concrete fill placed after the tile was set, where up to points near failure the inner shell generally remained in place.

As indicated by the average temperatures in the steel at failure, the concrete filling carried portions of the applied load varying with the area, the higher temperatures at failure being generally

incident with the tests having the larger fills. In the latter tests the periods between maximum expansion and failure were comparable with those obtaining for concrete protections, whereas in the tests where the concrete fill was omitted or of small area this time interval was relatively short (Table 43, p. 122).

(7) EFFECTIVENESS OF PLASTERING.—The tile in test Nos. 76 and 77 were covered with standard applications of gypsum and of lime plaster, respectively, applied 3 days after the concrete fill was placed, the columns being tested 42 and 45 days after plastering.

In test No. 76 the gypsum plaster began to fall off early in the test, exposing a few tile units at 2 minutes and more than half of the tile surface at 20 minutes. General cracking and spalling was probably delayed to some extent by the insulation given the tile by the plaster during the first few minutes of the fire exposure.

The lime plaster in test No. 77 fell off during the first half minute of the test, exposing about three-fourths of the total tile surface, its influence on the test result being apparently very small.

These results with plaster may not be applicable where it has seasoned for a longer time, and without further tests they should not be taken to hold rigidly for well-cured plaster coatings that from conditions of normal exposure are thoroughly dry.

(8) EFFECT OF WATER APPLICATION.—The water generally carried away the tile that had been decidedly damaged during the preceding fire exposure, although adjacent courses of relatively sound tile were in some instances carried down along with those impaired by the fire (Figs. 84 and 85). A large proportion of the effects took place during the first minute. In the test where the tile protection was filled with concrete the condition of the column after the water application approached that of partial concrete protection. In the case of the unfilled columns the steel in the region stripped of tile was unprotected, except for the portions of the mortar joint that adhered to the flanges.

No decided difference was noted between the outside wire ties and the wire mesh in the joints in effectiveness in holding the tile during the water application, although the comparisons were too few to afford definite conclusions.

(g) BRICK PROTECTIONS

(1) **PROPERTIES OF THE BRICK.**—The common brick used in the brick protections was a wire-end cut brick made in the Chicago (Ill.) district of calcareous surface clay. Compression tests gave average values of 3200, 1960, and 2815 pounds per square inch as tested on end, edge, and side, respectively, and average transverse strength of 862 pounds per square inch, with maximum variations above or below the averages of between 50 and 100 per cent (Tables 35 and 36). The brick was soft, with relatively low fusion point and high percentages of porosity and absorption (Tables 33 and 34, p. 361).

(2) **TEST RESULTS.**—In the two tests of columns protected by brick approximately the same relation obtained between time to failure and sectional area of the covered columns as for the hollow clay-tile protections (Fig. 54, Nos. 68 and 69).

In No. 68, where the brick was set on edge and end, the lack of stability somewhat shortened the test, a considerable amount of brick falling during the first 30 minutes.

In No. 69, with the brick laid flat, little cracking or spalling developed before failure. Fusion of the brick began between the fourth and fifth test hours, and after test the brick was found fluxed away to a depth of about $\frac{1}{2}$ an inch. The uniform temperature rise in the metal indicates that the fusion of the brick did not contribute greatly to the failure of the column, the same being evidently caused by normal transmission of heat through the covering (Fig. 131, p. 292).

(h) GYPSUM-BLOCK PROTECTIONS

(1) **STRENGTH AND POROSITY OF THE GYPSUM BLOCK.**—Compression tests of the solid gypsum block used for column covering gave an average strength of 468 pounds per square inch, with maximum range in values of 22 per cent below and 40 per cent above the average. The transverse strength averaged 160 pounds per square inch, the extreme range in results of individual tests being a little higher than in the compression tests (Tables 38 and 39). The porosity was quite uniform and high, averaging 62.4 per cent, as based on the total volume (Table 37, p. 363).

(2) **COMPARISON OF 2 AND 4 INCH PROTECTIONS.**—A comparison in points of time to failure of 2 and 4 inch gypsum protections is given in Fig. 56, where the line connects the average results

FIG. 56.—Comparison of 2-inch and 4-inch gypsum block protections

attained with each thickness. The 2-inch protections withstood the fire test 2 hours 22 minutes and 2 hours 36 minutes, and the 4-inch protections 4 hours 43 minutes, 5 hours 32 minutes, and 6 hours 24 minutes, respectively (Table 42*f*, p. 112.)

The protections were of solid 2 and 4 inch partition blocks set in gypsum and sand mortar, $\frac{3}{4}$ to $1\frac{1}{4}$ inch thick between blocks and column flanges, and with metal ties in the horizontal joints, the space between the blocks and the column webs being filled with gypsum block set in place or with a filling poured in place consisting of calcined gypsum, sand, and broken gypsum block.

The variations in results obtaining for each thickness of covering come within limits where they can be ascribed to incidental differences in material, workmanship, and test conditions, considering that the duration of the test was dependent upon the stability of individual blocks.

(3) CHARACTERISTIC FIRE EFFECTS.—The gypsum coverings failed due to checking, shrinking, and disintegration of the blocks which caused them to loosen and fall off. Characteristic heat effects are shown in Fig. 57. The process responsible for these effects consists mainly in the transformation of hydrated gypsum of the formula, $\text{Ca SO}_4 + 2\text{H}_2\text{O}$, to anhydrous calcium sulphate, by evaporation of the chemically combined water.

FIG. 57.- Blocks from gypsum coverings after test

Failure of the column occurred within 20 to 40 minutes after the first blocks had fallen. The interval between maximum expansions and failure was relatively short, due to the rapid temperature rise in the steel and the low load-carrying capacity of the covering material that remained in place.

(4) **HEAT-INSULATING PROPERTIES.**—The maximum temperature in the steel up to the point where the blocks began to fall off was generally below 150°C (302°F), which was much lower than those obtaining in comparable tests with the other covering materials after the same duration of fire exposure (Figs. 127 to 130). The high heat-insulating value of gypsum is due in part to the heat consumed by the change in crystalline structure noted above.

(5) **EFFECT OF WATER APPLICATION.**—In the case of the 2-inch. protection the effect of the first 2 minutes of the water application was confined to washing away of the partly calcined gypsum near the outer face of the covering, all blocks remaining in place. During the third minute most of the blocks on the sides on which water was applied were carried down along with portions of the poured filling (Fig. 86, p. 245).

On the 4-inch protection the water application of 5-minutes duration washed away the gypsum on three sides to a depth of 1 inch from the surface, all blocks remaining in place.

(i) **REINFORCED-CONCRETE COLUMNS**

In the reinforced-concrete columns of the fire-test series the coarse concrete aggregates used were Chicago limestone and New York trap rock, and the application of the results should be limited to columns made with these concrete aggregates. Comparisons given in paragraph (e) above on the behavior of concrete made with these and other aggregates and applied in coverings for steel columns indicate that less favorable results would be obtained with some of them when applied in reinforced-concrete columns than was obtained with the columns tested. Also in the fire and water tests, the sections of the columns made of siliceous gravel concrete developed much greater disruptive effects during the relatively short fire exposure preceding the water application than the sections made of limestone or trap-rock concrete. The behavior of limestone and trap-rock concrete in tests of reinforced-concrete columns was similar to that of the corresponding concrete of the column coverings, little cracking or spalling of consequence occurring before failure.

(1) INFLUENCE OF CONCRETE AGGREGATE.—The limestone-concrete columns all withstood the 8-hour fire test and while hot sustained loads exceeding twice the load applied in the 8-hour period. The two vertically reinforced trap-rock concrete columns failed after 7 hours 23 minutes and 7 hours 57 minutes, respectively, and the hooped column withstood the 8-hour fire test and failed under a load about 25 per cent greater than the load sustained during the fire test (p. 113). A 2-inch thickness of concrete next to the surface was assumed as covering in all cases and not included in the area used in computing working loads. The difference in results within the group can be attributed to concrete aggregate, the other incidental factors being comparable or favoring the tests giving the lower results. The trap-rock concrete fused and fluxed at some points to a depth of about 1 inch, which undoubtedly affected the time to failure to some extent. The results obtained with the concrete of both aggregates show a high degree of fire resistance.

(2) EFFECT OF FORM OF COLUMN AND REINFORCEMENT.—No effects due to shape of column or form of reinforcement were evident, differences in results being within the limits of incidental variations in test columns and test conditions.

No line of cleavage outside of the wire reinforcement was found after test in the hooped column of limestone concrete, except in the immediate region of failure, where it was apparently induced by the strains that developed when the column failed. In the case of the corresponding trap-rock concrete column, more evidence indicating separation of the outer protection from the core at the line of the reinforcement was found, effects which may in part have been caused by the fire exposure.

(3) RECOVERY OF STRENGTH AFTER FIRE TEST.—One length of each of the hooped reinforced-concrete columns about 3 feet long was cut outside of the failure region in the fire test and subsequently tested in compression. The limestone-concrete specimen sustained a total load of 517 000 pounds as against 243 000 pounds immediately following the fire test, and the trap-rock concrete specimen 342 000 pounds, compared with 163 000 pounds at the end of the fire test. While a portion of the difference may be due to initial variations in the strength of the concrete, the greater part can be ascribed to recovery in strength of concrete and reinforcement.

(4) **EFFECT OF WATER APPLICATION.**—The concrete of the columns subjected to fire and water tests was placed in three sections to permit using two or three kinds in each column.

In the case of the square, vertically reinforced column, No. 111, the water carried away the concrete at the corners outside of the bars and pitted the concrete on the most exposed face to depths of from $\frac{1}{8}$ to 1 inch for the limestone concrete and to a depth of 2 inches for the Meramec River gravel concrete in the middle section (Fig. 88, p. 247).

In the round, vertically reinforced column, the limestone concrete was pitted to a depth of 1 inch, and some of the concrete in the upper portion of the Joilet gravel concrete section was carried away. In the middle section, consisting of Meramec River gravel concrete, the outer concrete was stripped off by the water, exposing the reinforcing bars on two sides. In this, as in the preceding test, large cracks had formed in the concrete of the middle section during the fire period.

In the fire and water test of the hooped reinforced-concrete column, the water stripped the Meramec River gravel concrete and the granite concrete from the wire reinforcement on three sides during the first 15 seconds of the water period (Fig. 89). Spalling of concrete had exposed portions of the reinforcement in the middle section during the fire period. Further application of water caused stripping of reinforcement in the upper section of trap-rock concrete and increased the effects in the lower sections.

The condition of none of the reinforced concrete columns was such as to cause apprehension of early failure on being again exposed to fire after the water test, since the proportion of the load normally carried by the steel reinforcement was not large. Load tests to failure made after the water test gave factors of safety of over four as based on the calculated working load.

(J) **TIMBER COLUMNS**

Six tests of timber columns were made, four being tested unprotected. One was protected by a single layer of Portland-cement plaster on metal lath and one by a single layer of gypsum wall board. Two species of wood, longleaf pine and Douglas fir, and two types of post cap details were employed (Fig. 10).

(1) **UNPROTECTED TIMBER COLUMNS.**—The time to failure of the unprotected timber columns varied from 35 to 50 minutes,

failure occurring in all cases at the bearings on the steel or cast-iron cap introduced near the top of the column (Figs. 80 to 82). The deformation at the bearing increased rapidly after the first 20 minutes, and at the failure the consequent depression equaled 3 inches or more (Fig. 47, p. 126).

The average time to failure in the tests with longleaf pine was nearly the same as that obtained with Douglas fir, although the tests were hardly comparable on account of the higher moisture content of the Douglas fir in the condition tested (Table 2, p. 30).

The columns with the cast-iron cap and pintle stood up 7 minutes and 15 minutes longer than the columns with the steel-plate cap, the deformations sustained by the wood at the cap before failure ensued being larger in the case of the cast-iron bearing.

(2) PROTECTED TIMBER COLUMNS.—The outer coat of the plaster on parts of the lower half of the metal lath and plaster protection (test No. 78) spalled during the first half hour of the fire test, followed later by local buckling out of lath and supporting channels. Flames from the column issued through this opening before the end of the first hour, and about 15 minutes later flames issued from cracks in the plaster around the column cap. Crushing of the wood at the bearing due to heating of the cap caused a rapid rate of depression, beginning at 1 hour 40 minutes (Fig. 47), failure accompanied by fracture of the cap occurring at 2 hours 15 minutes (Fig. 79, p. 238).

The wall board consisted of gypsum-plaster filling between paper facings. At 20 minutes flames issued from horizontal cracks in the plaster board. The board began to fall off the body of the column at about 40 minutes, and at 54 minutes nearly one-half of the covering had fallen. At 32 minutes part of the covering fell off the flanges of the steel-plate cap, the failure at 1 hour 13 minutes being due to slipping at the cap bearing same as for the unprotected timber columns (Fig. 81, test No. 80).

The protections increased the ultimate resistance of the columns by 100 to 200 per cent as compared with that in the unprotected condition. Protections on timber columns should be applied with due consideration for possible deterioration of the wood that may under certain conditions be induced by the presence of the coverings, particularly when the wood is not fully seasoned or where it is exposed to dampness.

(3) **STRENGTH AFTER FIRE TEST.**—The failure being localized at the bearing did not develop the full resistance of the column. The wood was burnt and charred to depths of $\frac{1}{4}$ to $1\frac{1}{4}$ inches, involving reductions in effective area of 29 to 55 per cent. One specimen 3 feet long was cut from each of the tested columns and tested in the compression about 10 weeks after the fire test. The results of the tests are given in Table 45, where also are given results of tests on comparable specimens of unburnt timber. The results indicate a reduction in the total load approximately in proportion to the reduction in area, the average of the maximum unit loads sustained by the burnt and unburnt timber of each species being nearly equal.

The strength remaining in the timber at the end of the fire test was considerably lower than the values given in Table 45, due to its heated condition.

**TABLE 45.—COMPRESSIVE STRENGTH OF TIMBER AFTER
FIRE TEST**

Specimens 3 ft. long

Specimen No.	Species	Section Before Test		Section After Test		Reduction, Per cent	*Moisture Content, Percent	Maximum Load	
		Outside Dimensions, In.	Area, Sq. In.	Outside Dimensions, In.	Effective Area, Sq. In.			Total, Lb.	Lb. per Sq. In.
78-1	Longleaf pine.	11.3 by 11.5	129	7.9 by 8.4	58	55	15.5	370,000	6380
79-1	Longleaf pine.	11.2 by 11.2	125	8.8 by 9.1	76	44	14.4	381,000	5010
80-1	Longleaf pine.	11.2 by 11.3	126	9.0 by 9.0	75	40	15.8	399,000	5320
81-1	Longleaf pine.	11.4 by 11.4	129	9.0 by 9.1	77	40	16.2	475,000	6170
	Longleaf pine.	Average...					15.5		5720
82-1	Douglas fir....	11.4 by 11.4	129	9.5 by 9.5	87	33	22.7	324,000	3725
83-1	Douglas fir....	11.4 by 11.4	129	10.6 by 10.7	91	29	18.8	400,000	4395
	Douglas fir....	Average...					20.7		4060
P-1	Longleaf pine.	11.2 by 11.4	127	Specimen of unburnt timber.			19.3	750,000	5705
F-1	Douglas fir....	11.4 by 11.4	129	Specimen of unburnt timber.			18.9	502,000	3890
F-2	Douglas fir....	11.4 by 11.4	129	Specimen of unburnt timber.			18.9	500,000	3875
	Douglas fir....	Average...					18.9		3880

*Determined from samples taken near point of failure, dried to constant weight at 100° C.

XIII. FIRE-RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS

The results of the tests in point of time to failure will be summarized in terms of hours and minutes of fire resistance afforded by the different types of columns and protections tested.

1. BASIS OF DERIVATION

(a) METHOD OF COMPUTATION

A given resistance period is taken to hold, if the time to failure in the fire test or the average of the time to failure in a group of similar tests is equal to $1\frac{1}{2}$ times the given resistance period. The deduction of one-third of the test duration is made to allow for incidental variations in material and workmanship of columns and coverings and differences in load and fire conditions that cause variations in results with nominally comparable columns. Individual test results within a given group may be below this limit, but not below the designated resistance period.

(b) INTERVALS

Resistance periods are taken 5 minutes apart up to one-half hour, at 15-minute intervals from one-half hour to 1 hour, at one-half hour intervals from 1 to 4 hours, and at 1-hour intervals from 4 to 8 hours. A variation of one-fourth of an interval is allowed, a given even value of the resistance period being taken if the period, computed according to the method given above, is not more than one-fourth of the interval below the given period value.

(c) TABLE OF FIRE-RESISTANCE PERIODS

A tabulation of the fire-resistance periods obtained from the results of the present series of tests is given in Table 46.

(d) DERIVATION OF METHOD

The method of computation is obtained from comparison of duplicate or nearly duplicate column tests considered in connection with conditions affecting comparable column constructions in buildings. In the fire tests, the maximum range in time to failure of unprotected structural steel columns was 49 per cent, comparable tests of partly protected steel columns gave a range in values of 24 per cent, plaster on metal lath protections, 15 per cent,

TABLE 46.—FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS

Type of Column	PROTECTION		Minimum Area of Solid Material, Sq. In.	Nominal Thickness of Protection, In.	Fire Resistance Period
	Material	Details			
Structural steel	Unprotected	Minimum metal thickness, .20 in.	8		10 min.
Structural steel, solid section	Partly protected by filling, reentrant spaces with concrete. Concrete aggregates; limestone, calcareous gravel, trap rock, granite, sandstone, or hard coal cinder	Mixture, 1:6 or 1:8. Concrete tied with vertical and horizontal steel ties	35		½ hr.
Structural steel, solid section	Partly protected by filling, reentrant spaces with concrete. Aggregates; limestone, calcareous gravel, or trap rock	do.....	60		¾ hr.
Structural steel, open latticed section	Partly protected by filling interior and reentrant spaces with concrete; trap rock aggregate	Mixture, 1:6. Filling extends to outside rivets and covers lattice and main members	120		2 hr.
Structural steel, open latticed section	Partly protected by filling interior and reentrant spaces with concrete; limestone or calcareous gravel aggregate	do.....	120		3½ hr.
Structural steel	Portland cement plaster on metal lath	Proportion of plaster, 1:½, 2½, Portland cement, hydrated lime, and sand	40	1 layer 1 in.	¾ hr.
Structural steel	do.....	do.....	80	2 layers each ⅝ in.	1½ hr.
Structural steel	Concrete; siliceous gravel aggregate	Mixture, 1:6. Concrete tied with steel ties or wire mesh, equivalent to not less than No. 5 (B. & S. gage) wire on 8-in. pitch	100	2	1 hr.
Structural steel	do.....	do.....	200	4	2½ hr.
Structural steel	Concrete; granite, sandstone, or hard coal cinder aggregate	Mixture, 1:6. Concrete tied as above	100	2	2½ hr.
Structural steel	do.....	do.....	140	3	3½ hr.
Structural steel	do.....	do.....	200	4	5 hr.
Structural steel	Concrete; trap rock aggregate	Mixture, 1:6. Concrete tied as above	100	2	3 hr.
Structural steel	do.....	do.....	140	3	4 hr.
Structural steel	do.....	do.....	200	4	5 hr.
Structural steel	Concrete; limestone or calcareous gravel aggregate	Mixture, 1:6. Concrete tied as above	100	2	4 hr.
Structural steel	do.....	do.....	140	3	6 hr.
Structural steel	do.....	do.....	200	4	8 hr.

TABLE 46.—FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS—Continued

Type of Column	PROTECTION		Minimum Area of Solid Material, Sq. In.	Nominal Thickness of Protection, In.	Fire Resistance Period
	Material	Details			
Structural steel, solid section	Hollow tile; semi-fire clay, medium hard. No filling	Mortar joint between tile and column flanges. Outside wire ties	80	2, 3, or 4	1 hr.
Structural steel, solid section	Hollow tile; surface clay or shale. Concrete filling on web sides	do	100	2, 3, or 4	1 hr.
Structural steel, solid section.	Hollow tile, extra heavy; surface clay. Concrete filling on web sides	do	160	3 or 4	1½ hr
Structural steel	Hollow tile; semi-fire clay or surface clay. Concrete filling all around	Outside wire ties	160	2, 3, or 4	2 hr.
Structural steel	do	Metal ties in horizontal joints	160	3 or 4	2½ hr.
Structural steel, solid section	Hollow tile; surface clay. Hollow tile filling	Mortar joint between tile and column flanges and webs. Metal ties in horizontal joints	240	2 layers each 2 in.	3 hr.
Structural steel, solid section	do	Mortar joint between tile and column flanges and webs. Outside wire ties	240	2 layers each 2 in.	1 hr.
Structural steel	Common brick; surface clay	Brick set on edge and end	140	2¼	1 hr.
Structural steel	do	Brick laid on side	240	3¼	5 hr.
Structural steel	Solid gypsum block. Gypsum block or poured gypsum filling	Metal ties in horizontal joints. Mortar joint between blocks and column flanges	130	2	1½ hr.
Structural steel	do	do	180	3	2½ hr.
Structural steel	do	do	240	4	3½ hr.
Round cast iron	Unprotected; unfilled	Minimum thickness of metal, .60 in.	12		20 min.
Round cast iron	Unprotected; interior filled with concrete	do	35		½ hr.
Round cast iron	Portland cement plaster on high ribbed metal lath	Proportion of plaster, 1:½:2½, Portland cement, hydrated lime and sand	60	1 layer, 1½ in. ½ in. air space	2 hr.
Round cast iron	Concrete; trap rock, granite, or hard coal cinder aggregate	Mixture, 1:7. Concrete tied with steel ties equivalent to not less than No. 5 (B. & S. gage) wire on 8-in. pitch	70	2	2 hr.
Round cast iron	Hollow tile; porous semi-fire clay	Outside wire ties. Mortar joint between tile and column	70	2	2 hr.

TABLE 46.—FIRE RESISTANCE PERIODS DERIVED FROM THE TEST RESULTS—Concluded

Type of Column	PROTECTION		Minimum Area of Solid Material, Sq. In.	Nominal Thickness of Protection, In.	Fire Resistance Period
	Material	Details			
Steel pipe	Unprotected. Filled with concrete	Concrete mixture, 1:1½:3	35		25 min.
Reinforced steel pipe	Unprotected. Filled with concrete and reinforced in the fill with structural shapes	Concrete mixture, 1:1½:3	45		¾ hr.
Reinforced concrete	Limestone or calcareous gravel concrete	Mixture 1:6. Concrete reinforced with vertical bars and lateral ties or hooping	220	2	8 hr.
Reinforced concrete	Trap rock concrete	do.	220	2	5 hr.
Timber, long-leaf pine or Douglas fir	Unprotected	Unprotected cast iron or steel cap	120		25 min.
Timber, long-leaf pine or Douglas fir	Gypsum wall board	Cast iron or steel cap	140	1 layer, ¾ in.	¾ hr.
Timber, long-leaf pine or Douglas fir	Portland cement plaster on metal lath	Cast iron or steel cap. Proportion of plaster, 1:½:2½. Portland cement, hydrated lime, and sand	160	1 layer, 1 in. with ¾ in. air space	1½ hr.

and 2-inch concrete protections 25 per cent, all percentages being based on the highest value within the group. The time to failure in three tests of unprotected cast-iron columns differed by less than 1 per cent. In hollow clay-tile protections, excluding Nos. 50 and 50A on account of not being tied, the maximum range in test duration within a group of comparable tests was 30 per cent, and in gypsum-block protections 26 per cent. Unprotected timber columns with exposed steel or cast-iron caps gave a range in time to failure of 30 per cent, part of which was due to difference in bearing details.

In the case of unprotected structural steel columns, the range in shape of section tested was large and a deduction of one-third from the average time to failure gives a period below which test results, obtained under comparable load and fire conditions, are not likely to fall, with the limitations in thickness of metal and area of section given in Table 46. All of the other variations noted above come within 30 per cent of the highest value in the respec-

tive groups, although it is appreciated that the test duplications were not sufficient in number to develop the full possibility of difference in results; also, that columns and protections in buildings are subject to greater variability than the test columns. The basis of derivation adopted assumes possibility of variation of about 33 per cent above and below an average value, or a total of about 66 per cent of the average. Irrespective of whether or not this range in results would fully develop in a sufficiently large number of comparable cases in buildings, the use of the one-third reduction to obtain minimum ultimate fire-resistance periods is considered justified because of the few number of tests on which most of the resistance periods are based, several of them being derived from single tests, the results of which it is necessary to assume fall within the higher rather than the average or lower range of possible variation.

(e) **RESISTANCE TO WATER APPLICATION**

With reference to fire and water exposure the results are regarded as satisfactory if the applied load was safely sustained during the fire and water periods, and in the case of the protected columns, if the covering remained in place, after a 2-minute period of water application, to such extent as to prevent early failure of the column on being again exposed to fire, precluding in all cases complete removal of the covering from any section of the column.

(f) **SIZE LIMITATIONS**

The derived fire-resistance periods apply most nearly to columns of approximately the same size as those tested and should be applied with considerable caution in connection with smaller columns. Minimum areas with relation to the derived periods are given in Table 46. The studies made on effect of size indicate that the test results can be applied with safety to columns of larger size than those tested.

(g) **APPLICATION TO BUILDING CONDITIONS**

It is believed that the types of columns and protections tested can be applied with confidence in building construction as being able to resist fires of duration equal to the pertaining resistance periods herewith derived for the respective types, provided that reasonable care is taken to identify the material used and to secure a fair grade of workmanship.

In constructing the coverings and columns, a consistent effort was made to introduce methods and conditions similar to those incident with building construction, and it is thought that the columns as tested are fairly representative of the average attained in good building practice.

The columns tested covered a wide range in material and shape of section, and each class of covering material was represented by its main varieties in common use. In applying the results broadly some difficulty may be experienced due to difference in materials of the same kind or name as occurring in different localities, relative to which some explanatory notes and cautions are given in deriving fire-resistance periods with the respective materials. It is believed that if the materials are properly identified and classed with the corresponding materials employed in the column tests, the resulting difference in fire-resisting properties will not be large, provided the mineral constituents and impurities are within the limits hereafter given.

In the column tests the temperature indicated by the pyrometers, while corresponding on the average quite closely with those on the reference curve (compare reference curve and average curve, Fig. 39), was lower than that of the furnace gases surrounding them, due mainly to radiant heat interchange between the pyrometer and the colder furnace inclosure. The difference was determined to be as large as 150°C for points near the beginning of the test, with gradual decrease to less than 50°C at the end of an 8-hour run (Sec. IX, par. 2*d*, Fig. 41). The high intensity of the furnace exposure to which the columns were subjected can be taken as compensating to some extent for differences in material incident with broad application of the test results, and give assurance that such application is justified where the variance from the materials tested is not too large.

The loads applied to the columns during test come within the higher range of normal working loads, and it is believed that the loads imposed on columns under fire conditions in buildings will seldom be much larger, either as caused by floor loads or by unequal expansion of adjacent columns during the fire.

The column coverings had a full and firm bearing at the base of the column and were full and continuous at the top, and therefore took portions of the applied load proportionate to their

relative area and rigidity with respect to the structural section. Coverings as applied in buildings will sometimes have less firm bearing at the base and be less continuous at the top, due to obstructions to proper placement offered by the floor members. This difference in ability to carry load will affect chiefly the concrete protections and fillings, since the other types have little load-carrying capacity near failure. With concrete protections it will affect only the period between maximum expansion and failure, the metal section assuming almost all of the load during the preceding period, due to its higher rate of expansion. Concrete will develop bond within a relatively short distance with a number of forms of metal section, making the covering in effect a part of the column in the middle portion of its length, even if it is not fully continuous at the ends. It is therefore apparent that the difference between the test condition and the condition of possible occurrence in buildings is not large as far as it concerns the fire resistance of the column, and considering the relatively smaller variations due to other causes generally incident with concrete protections, it can be taken as sufficiently allowed for in the reduction of the time to failure adopted for deriving fire-resistance periods.

The fire resistance afforded by columns is based on the time to failure rather than the useful limit because the former is definitely determined by the test procedure. It is deemed, however, that with the interpretation of test results given above the protection given by the columns and coverings will generally be sufficient to prevent permanent damage of such extent as to require repair or replacement after fire exposures corresponding in duration to the resistance periods here established. Only where very severe local fire exposure is coincident with other unfavorable conditions, such as inferior material and workmanship, will the resistance of the column beyond its useful limit be likely to be developed.

2. DERIVATION OF FIRE-RESISTANCE PERIODS

In deriving the resistance periods the method described in paragraph 1*a* of this section is followed, giving consideration also to effects of water application as explained in paragraph 1*e*. Interpolations for covering thicknesses intermediate between those tested are made where justified by the test results. Limited extension of results to materials not included in the particular

combination considered, but giving similar test characteristics in other related tests of this series, is made in the case of concrete and hollow clay-tile protections (Table 46, p. 197-199).

For columns with square or rectangular protections, all portions of the main members are assumed covered by not less than the nominal thickness of material specified. With round coverings, on columns of square or rectangular outline, the distance from the surface to the edge of the main column members may be reduced somewhat, provided the resulting cross-sectional area is not less than that of a square or rectangular protection of the same nominal thickness.

Details having relatively large areas, such as lattice bars and splice plates, need to be covered by the nominal thickness specified. For smaller areas as of rivets and extreme edges of bracket or supporting angles, the covering thickness may be reduced to 1 inch.

Where partitions or pipe inclosures are supported by column coverings the connection should be made so that the stability of the covering is not affected by buckling or failure of the supported walls. Pipes should not be placed within the thickness of covering required for the given resistance periods nor incorporated in the covering in such manner as to dislodge any essential portion thereof by expansion or buckling incident with normal use or with fire conditions.

(a) UNPROTECTED STRUCTURAL-STEEL COLUMNS

As based on test Nos. 1 to 8, and with not less than the minimum metal thickness and area of section given in Table 46, unprotected structural-steel columns give *10-minute* fire resistance.

The low degree of resistance afforded indicates unsuitability for use where fires of any appreciable degree of intensity and duration are possible.

(b) PARTLY PROTECTED STRUCTURAL-STEEL COLUMNS

The protection consists of concrete that fills the interior and reentrant portions to the outside of the extreme metal on the main members. Except on the open sides of latticed columns, the concrete is secured with horizontal and vertical metal ties.

(1) SOLID-SECTION COLUMNS.—Structural-steel columns of solid rolled or riveted section, whose reentrant portions are filled with

concrete made with any of the aggregates included in the present series of tests, except highly siliceous sand and gravel, mixed in proportion of 1 part Portland cement to not more than 8 parts fine and coarse aggregates combined, and with combined area of steel and concrete not less than 35 square inches, give *one-half hour* fire resistance. Similar columns filled in the same manner with limestone, calcareous gravel, or trap-rock concrete, and with area of filled column not less than 60 square inches, give *three-fourths hour* fire resistance.

The one-half hour period is based on test Nos. 15, 16, and 17, and the three-fourths hour period on test Nos. 14, 20, and 21.

(2) OPEN LATTICED SECTION.—Structural-steel columns of open latticed section, with the interior and reentrant portions filled with concrete of proportion 1 part Portland cement to not more than 6 parts fine and coarse aggregates combined, and with total area of not less than 120 square inches, give *2-hour* fire resistance if concrete is made with trap-rock aggregate and *3½-hour* fire resistance if limestone aggregate is used.

The above conclusions are based on test Nos. 18 and 22, the higher resistance over the solid section columns being due to absence of large areas of exposed metal. It is essential to use concrete that is not subject to cracking or spalling, since the covering on parts of the main members is usually little more than 1 inch thick and not tied.

(c) STRUCTURAL-STEEL COLUMNS WITH PLASTER ON METAL-LATH PROTECTIONS

The plaster can be applied on either expanded metal or woven-wire lath. The layer thickness is measured from the inner line of the lath. The proportion of the plaster considered is 1 part Portland cement to not more than $2\frac{1}{2}$ parts medium-coarse sand, with lime added equal to one-tenth of the cement volume. The periods derived are based on test Nos. 23 to 26 and 110, in which the above proportion was used. The effect on fire-resistive properties produced by varying the lime content is otherwise not known.

(1) SINGLE-LAYER PROTECTION.—Structural-steel columns protected by a layer of Portland-cement plaster 1 inch thick applied on metal lath give *three-fourths hour* fire resistance.

(2) DOUBLE-LAYER PROTECTION.—Structural-steel columns protected by two layers of Portland-cement plaster each $\frac{7}{8}$ inch thick with $\frac{3}{4}$ -inch air space between layers give *1½-hour* fire resistance.

(d) CONCRETE PROTECTIONS ON STRUCTURAL-STEEL COLUMNS

The protections, preferably placed without construction joints, should extend through floors, where the floor filling is of other material than concrete. Lateral ties consisting of iron or steel wires or mesh, embedded in the covering and extending to within about 1 inch from its surface, are to be equivalent to not less than a No. 5 (B. and S. gage) steel wire wound spirally on a pitch of 8 inches.

The proportion of mixture taken to hold is 1 volume part Portland cement to 6 parts fine and coarse aggregates combined, 94 pounds of the cement being taken as its weight per cubic foot. The effect of reducing the cement content of the mixture was not definitely determined in the tests, although some reduction in fire resistance was noted as between the 1:6 and the 1:8 proportion, estimated as equivalent to reductions in the resistance periods of not more than $\frac{1}{2}$ hour for periods up to 3 hours and possibly as much as 1 hour for the longer resistance periods.

Limitations relative to size are given in Table 46 (p. 197). For columns of dimensions much larger than the given minimum, the periods apply with a considerable margin of safety.

(1) SILICEOUS-GRAVEL CONCRETE PROTECTION.—Structural-steel columns covered with a 2-inch thickness of concrete made with gravel aggregate of high silica content give *one-hour* fire resistance, and with similar 4-inch protections give *2½-hour* fire resistance.

Since the failure of coverings of this type of concrete is dependent on stability of portions thereof, no adequate basis is present for estimating the value of the resistance period for thicknesses intermediate between 2 and 4 inches.

The periods are based on test Nos. 39 and 45, where the aggregate consisted mainly of chert grains and pebbles. Since this form of silica develops as great or greater disruptive effects on exposure to fire as quartz, flint, and related minerals, the derived protection periods apply generally where concrete made with all common forms of siliceous aggregates is used.

(2) GRANITE, SANDSTONE, OR HARD-COAL CINDER CONCRETE PROTECTION.—Structural-steel columns covered with 2-inch protections of granite, sandstone, or hard-coal cinder concrete give *2½-hour* fire resistance, with 3-inch coverings give *3½-hour* fire resistance, and with 4-inch coverings give *5-hour* fire resistance.

Tests were made of 2-inch coverings of concrete made with sandstone and cinder (Nos. 31, 32, 32A, 43, 44, and 104), and in the 4-inch thickness, with granite as the coarse aggregate (test Nos. 34, 34A, and 103). The extension of the results with sandstone and cinder concrete to cover thicknesses heavier than 2 inches, and of granite concrete to cover thicknesses lower than 4 inches, is based on relative effectiveness with respect to trap-rock concrete in coverings (test Nos. 29, 36, and 37) of comparable thickness.

The granite used in the concrete of the column coverings tested contained 35 per cent quartz and about 60 per cent feldspars, chiefly orthoclase (Table 20, p. 345), the size of crystals ranging from 0.03 inch to 0.35 inch. The range in quartz content of granites is generally from 20 to 40 per cent, with extremes from 5 to 50 per cent. In crystal size the general range is from 0.02 inch to 0.40 inch, with extremes up to 2½ inches. It appears, therefore, that the granite tested came within the higher range in quartz content and had crystals of average size. To what extent changes in mineral composition and crystal size of granite affect the fire-resistive properties of concrete made with it is not known, although it would be expected that those of lower quartz content will be the least subject to destructive fire effects.

The Berea sandstone used in these tests was a silica cemented stone of average hardness consisting almost wholly of subangular grains of quartz. Some disintegration from heat due to loss of cementing properties was noted, but not enough to affect the fire-resisting properties, the principal fire effects being cracking and dislodgment of relatively large masses, induced presumably by the high quartz content.

Among sandstones used as concrete aggregate, quartzite has all pore spaces filled with silica cement, resulting in a hard, homogeneous, quartz mass. Flame tests made on small specimens of the stone do not indicate any greater disruptive effects than those incident with similar tests of other sandstones.

Silica cemented sandstones range in hardness from compacted sand to a condition approaching quartzite, all being of high silica content, usually over 95 per cent. The softer grades are generally eliminated from use as concrete aggregate on the score of deficient strength and hardness.

The lime-cemented sandstones range in mineral composition from nearly pure sandstone to sandy limestones carrying only small amounts of silica. The iron (limonite) and clay cemented sandstones, of which the brownstones are representative, carry variable amounts of feldspar and mica, the range in free quartz content being from 50 to 90 per cent. The bluestones of New York, Pennsylvania, and West Virginia contain from 20 to 60 per cent quartz, the other minerals being mainly feldspar and hornblende.

While not possible to predict fully the behavior of combinations of minerals on exposure to fire, it is probable that the addition of calcareous, clayey, or feldspathic minerals to quartz to form the sandstones noted above will decrease rather than increase the cracking incident with fire exposure of purer forms of the rock.

The hard-coal cinder contained about 10 per cent unburned coal and 5 per cent ash, and to obtain comparable results these percentages should not be greatly exceeded.

(3) TRAP-ROCK CONCRETE PROTECTION.—Structural-steel columns protected by 2-inch coverings of trap-rock concrete give 3-hour fire resistance, with 3-inch coverings give 4-hour fire resistance, and with 4-inch coverings give 5-hour fire resistance (test Nos. 29, 36, 37, 40, 101, 103, and 104).

The periods for the 2 and 4 inch covering thicknesses were derived directly from the test results, and that for the 3-inch thickness taken as their average.

Trap rock is a dark-colored, fine-grained igneous rock that does not vary greatly in mineral composition and carries at most only a trace of quartz (Table 20). The term does not include the hard sandstones known under the same name in some localities.

(4) LIMESTONE OR CALCAREOUS-GRAVEL CONCRETE PROTECTION.—Structural-steel columns protected by 2-inch coverings of limestone or calcareous-gravel concrete give 4-hour fire resistance, with 3-inch coverings give 6-hour fire resistance, and with 4-inch coverings give 8-hour fire resistance.

Tests were made of 2-inch coverings of limestone and calcareous-gravel concrete (test Nos. 28, 28A, 30, 38, 101, 102, and 104) and of 4-inch coverings of 1:6 and 1:8 limestone concrete (test Nos. 33, 33A, 35, 41, 42, and 103). The period for the 2-inch thickness was derived directly from the test results. The columns with the

4-inch covering all withstood the 8-hour fire test, and while hot sustained such large additional loads as to justify the conclusion that in the lower range of results with comparably protected columns the working load will be sustained during an 8-hour fire period.

The limestone and the calcareous gravel of the tests were dolomitic and contained about 5 per cent of clayey impurities (Table 20). It is thought that the results apply without modification where high calcium limestone or gravel of nearly the same purity is used. The silica impurities in limestone occur as free silica (quartz, chert, flint, and opal) and silica combined in clay and other silicates. Most limestones carry from 5 to 10 per cent of combined clayey impurities, but not many have more than 10 per cent of free silica, and within the given limits these impurities are not deemed objectionable for stone or clean gravel used as concrete aggregate in fire-resistive construction. Some limestones contain higher percentages of free silica and particularly the waste from limekiln and cement plant quarries is likely to be high in chert.

Gravels in the glaciated area, which includes nearly all of the territory north of the Missouri and Ohio Rivers, New England, New York, northern Pennsylvania, and northern New Jersey, may contain material from any point to the northward and formed under the most diverse conditions will vary greatly in mineral composition both vertically and horizontally. This also applies to the gravels of the Great Lakes and the ocean gravels as far south as New York. If the superior fire-resistive qualities of concrete made with calcareous gravel is to be assured, or the highly siliceous gravel avoided, it is necessary to identify the material in each bank by means of suitable chemical or mineralogical analyses at successive times as the development proceeds. *

Outside of the glaciated area, the river and shore gravels are more uniform, although varying with changes in the rock formations of the drainage area or headlands.

The calcareous content of the sands (Fox River and Joliet) used in combination with the limestone and calcareous gravel was higher than the combined quartz and chert content (Table 20). While all sands are likely to contain considerable silica, and the extent of their influence on the thermal properties of concrete made with them is not known, it is thought best to avoid using highly siliceous

sands in combination with limestone and calcareous gravel where the full fire-resistive value of the concrete is to be utilized. As used with the other coarse concrete aggregates, the mineral composition of the sand is of minor importance, provided it meets the common requirements for a good concrete sand. The sand in a given deposit is generally more uniform in composition than the larger-sized material.

(c) **HOLLOW CLAY-TILE PROTECTIONS ON STRUCTURAL-STEEL COLUMNS**

The tile is assumed applied in courses about 12 inches high with fairly full horizontal and vertical mortar joints between the tile units. The mortar should also fill the space between the tile and column flanges except where it is filled with concrete. The proportion of mortar used in the tests was 1 volume part Portland cement to 1 of stiff slaked lime, and 4 of fine sand. While the influence of mortar proportion on the effectiveness of the covering is not definitely known, the test results are not fully applicable unless mortar having about the same content of cementing materials as that given above is used, and particularly the Portland cement should not be reduced below the proportion given. As an aid in assigning fire-resistance periods for combinations of tile and details of application not included in the tests, it can be stated that in the tests the semifire-clay tile burnt to medium hardness displayed the best fire-resistance properties, followed in order by tile of hard burnt semifire clay, of surface clay, and of shale.

The coverings tested were generally tied with one No. 12 (B. & S. gage) iron or steel wire placed tightly around the outside of each course, or by strips of woven wire of $\frac{3}{8}$ -inch mesh, placed in the horizontal joints and lapping at the corners. The latter method is considered superior to the outside wire ties and can be substituted for them where desired. The difference in effectiveness of the two methods is not considered so large as to justify requiring the mesh except for tile subject to severe disruptive effects on exposure to fire and where the higher resistance periods are to be attained. Other forms of interior ties may be used if equivalent in effect to the wire mesh.

In assigning fire-resistance periods to given protections, consideration is given to the effect of area of solid material in the cross section of the covered column, having reference to the minimum values given in Table 46 (p. 198).

The proportion of mixture for the concrete filling was 1 part Portland cement to not more than 8 parts fine and coarse aggregates combined, the filling being placed after the tile was set. Any aggregate used in these tests may be employed except highly siliceous gravel.

(1) UNFILLED PROTECTION.—Structural-steel columns with hollow clay-tile protections of 2, 3, or 4 inch semifire-clay tile of medium hardness, tied with outside wire ties, give *one-hour* fire resistance.

This period is based on results of fire tests Nos. 48 and 49 taken together with fire and water tests Nos. 105 and 107. The tile in these tests developed fewer disruptive effects on exposure to fire than any other kind tested. Tile more sensitive to abrupt temperature change is not adapted for use in unfilled protections, as the cracking and spalling resulting after a short fire exposure allows the heat to readily reach the steel, also, on application of water, parts of the impaired covering are liable to be carried away, leaving the metal unprotected against a possible recurring fire.

The use of unfilled protections should generally be avoided as the filling materially increases their stability and insulating value.

(2) SHALE OR SURFACE-CLAY TILE PROTECTION WITH CONCRETE FILLING REENTRANT SPACES.—Structural-steel columns with hollow clay-tile protections of 2, 3, or 4 inch shale or surface-clay tile, tied with outside wires and with concrete filling the reentrant spaces between tile and column web, give *one-hour* fire resistance (test Nos. 50, 50A, 52, 53, and 106).

Similar protections of 3 or 4 inch surface-clay tile with heavy shells and webs give *1½-hour* fire resistance (test Nos. 51 and 51A).

(3) SEMIFIRE-CLAY OR SURFACE-CLAY TILE PROTECTION WITH FULL CONCRETE FILLING.—Structural-steel columns with hollow-tile protections of the given types of clay, set 1 inch away from the flanges and edges, with concrete filling all spaces between the steel and the tile, give *2-hour* fire resistance where outside wire ties are used (test Nos. 54, 55, and 57), and with wire mesh in the horizontal joints give *2½-hour* fire resistance (test Nos. 56 and 77).

The thickness of tile, using outside wire ties, may be either 2, 3, or 4 inches. With the wire mesh the thickness should preferably be not less than 3 inches to allow ties of sufficient width to be used.

(4) **DOUBLE 2-INCH TILE PROTECTION.**—Structural-steel columns with protections of two layers of 2-inch surface-clay tile, and with the space between the inner tile layer and the column filled with tile set in place, give *3-hour* fire resistance when the covering is bonded with wire mesh in the horizontal joints and *1-hour* fire resistance if tied with outside wire ties.

The periods are based on test Nos. 58 and 59, which indicated a marked difference in results due to the method of tying the surface-clay tile applied in the given type of protection. Concrete filling may be substituted for the tile filling if desired.

(f) **BRICK PROTECTIONS**

The fire resistance periods are derived from the results of test Nos. 68 and 69. The same qualifications relative to mortar apply as for the hollow clay-tile protections, as also the size limitations given in Table 46 (p. 198).

Structural-steel columns thus protected with common surface-clay brick, set on edge and end outside of the flanges and edges and filling the whole space to the steel, give *1-hour* fire resistance, and with brick laid flat on side to form a solid protection of about 4-inch thickness *5-hour* fire resistance is developed.

(g) **GYPSUM-BLOCK PROTECTIONS**

The derived resistance periods apply to columns covered with solid gypsum block set in mortar of proportion, 1 part calcined gypsum to 3 parts fine sand, the blocks being bonded with strips of corrugated iron or wire mesh placed in the horizontal joints over all vertical joints. The space between the outer blocks and column can be filled with gypsum blocks set in place or with poured filling consisting of calcined gypsum, broken gypsum blocks, and sand, mixed with enough water to enable proper placement.

Structural-steel columns with protections of 2-inch solid gypsum blocks placed according to the above details give *1½-hour* fire resistance (test Nos. 65, 66, and 108), and with similar protections of 4-inch solid gypsum blocks give *3½-hour* fire resistance (test Nos. 64, 67, 67A, and 109). Interpolation between the two thicknesses appears justified, the protection made with the 3-inch solid block giving accordingly a resistance period of *2½ hours*.

The area limitations given in Table 46 apply less rigidly to gypsum-block protection than to most other types, because the

failure is induced by loss of stability of the blocks after a given fire exposure and not to normal transmission of heat through the covering.

(b) CAST-IRON COLUMNS

(1) UNPROTECTED COLUMNS.—With the minimum area and wall thickness given in Table 46, unprotected cast-iron columns give *20-minute* fire resistance.

The fire tests (Nos. 9, 10, and 10A) were made on round columns of about $\frac{3}{4}$ -inch wall thickness, and gave uniform results, although they were not sufficient in number to develop possible variations due to quality of metal and differences in wall thickness. The fire and water tests, while inducing large permanent deflections, did not cause the columns to lose ability to sustain working loads.

Filling the interior with concrete (test No. 11) gives an increase in resistance of 10 minutes or a total period of *one-half hour*.

(2) PLASTER ON METAL-LATH PROTECTION.—A single heavy layer of Portland-cement plaster of $1\frac{1}{2}$ -inch average thickness applied on high-ribbed metal lath to round cast-iron columns gives a fire resistance of *2 hours* (test No. 27).

The same proportion of plaster applies as given above under (c) for protections on steel columns. To attain the required average layer thickness, the surface of the covering will extend about 2 inches outside of the metal on account of voids inside of the lath.

(3) CONCRETE PROTECTION.—Round, cast-iron columns with 2-inch protections of trap-rock, granite, or hard-coal cinder concrete, give *2-hour* fire resistance.

The period is based on test No. 47 where cinder concrete was used. Extension to include concrete made with the other aggregates is made because of characteristics similar to cinder concrete developed by them in other tests of concrete protection. Limestone and calcareous gravel concrete would on the same basis give longer resistance periods. The proportion of the concrete can be taken as 1 part Portland cement to not over 7 parts aggregate. Relatively small-sized aggregates need to be used, to enable good placement of the covering, and with some aggregates it may be necessary to increase the thickness of the covering to secure this result. The ties, equivalent to not less than No. 5 (B. & S. gage) steel wire on 8-inch vertical pitch, are to be supported about 1 inch away from the column.

(4) **HOLLOW CLAY-TILE PROTECTION.**—Cast-iron columns covered with 2-inch curved porous semifire-clay tile, with $\frac{3}{4}$ -inch mortar thickness between tile and column, and tied with outside wire ties, give 2-hour fire resistance (test Nos. 62 and 63).

The same details of application can be taken to hold as for hollow-tile protections on structural-steel columns.

(i) UNPROTECTED PIPE COLUMNS

Unprotected columns consisting of steel or wrought-iron pipes not smaller than the standard 6-inch pipe size, with filling of concrete, give 25-minute fire resistance.

Pipe columns not smaller than the 7-inch standard pipe size, filled with concrete and reinforced in the fill with structural shapes, give *three-fourths hour* fire resistance.

The proportion of the concrete filling is taken to be 1 part Portland cement to not more than $4\frac{1}{2}$ parts fine and coarse aggregates combined (Table 46, p. 199).

The conclusions relative to pipe columns are based on test Nos. 12 and 13, made, respectively, on a 7-inch plain pipe column and on an 8-inch reinforced pipe column, both being standard new steel pipes with wall thickness of about $\frac{5}{16}$ of an inch. Extension of the result with the former to make it applicable to the 6-inch pipe size and of the latter to apply to the 7-inch size, appears justified.

(j) REINFORCED-CONCRETE COLUMNS

The proportion of the concrete for which the derived periods hold is 1 part Portland cement to 6 parts fine and coarse aggregates combined, the coarse aggregates being trap rock or limestone (cf. Sec. XII, par. 3*i*). The same considerations relative to possible variations in the aggregate hold as for concrete protection.

The columns may be round or square and reinforced with vertical bars held by lateral ties spaced not farther apart than 12 inches or by vertical bars and spirally wound wire hooping, all reinforcements to be covered by not less than 2 inches of concrete placed integrally with the structural portion of the column.

Under these conditions reinforced-concrete columns made with limestone or highly calcareous gravel-aggregate can be taken as giving 8-hour fire resistance (test Nos. 70, 72, 74, 111, and 112), and made with trap-rock aggregate, 5-hour fire resistance (test Nos. 71, 73, 75, and 113) (Table 46, p. 199).

The limestone-concrete columns withstood the 8-hour fire test, and while hot sustained ultimate loads of from a little less than twice to nearly three times the load sustained during the 8-hour period. This gives reasonable assurance that all similarly constructed columns, using a properly identified limestone aggregate, will sustain working load until the end of an 8-hour fire period, taking into account all probable variations. Calcareous gravel can be used in place of limestone if the free silica and other impurities are within the limits given in paragraph (d4) (p. 208).

Two of the trap-rock concrete columns failed during the 8-hour period and the other withstood the 8-hour fire test and about 25 per cent additional load at the end of this period. The fire-resistance period is therefore almost fully determinate on the basis of test duration.

(k) TIMBER COLUMNS

With unprotected or protected timber columns having exposed cast iron or steel caps, the resistance to fire is limited by failure at the bearings before the full resistance of the timber has been developed elsewhere. (Cf. Sec. XII, pp. 167 and 194). The resistance periods apply most fully to the species tested, long leaf pine and Douglas fir, although with failure occurring at the bearings the species of timber is not a governing consideration.

(1) UNPROTECTED TIMBER COLUMNS.—As based on test Nos. 79, 81, 82, and 83, unprotected timber columns with exposed cast iron or steel caps give *25-minute* fire resistance.

(2) PROTECTED TIMBER COLUMNS.—Timber columns having cast iron or steel caps and protected by a layer of gypsum wall board $\frac{3}{8}$ inch thick, give *three-fourths hour* fire resistance (test No. 80), and protected by a 1-inch layer of Portland-cement plaster on metal lath, with an air space between the timber and the plaster layer, give *1½-hour* fire resistance (test No. 78).

The protections are assumed to cover all exposed metal of the caps as well as the timber. The proportion of the Portland-cement plaster can be taken the same as given for plaster coverings on steel columns (Table 46, p. 197).

Protections on timber columns should be applied with caution on account of possible unfavorable effects upon the timber in point of susceptibility to decay. (Cf. Sec. XII, p. 194).

3. CONDITIONS GOVERNING FIRE DURATION IN BUILDINGS

The intensity and duration of building fires depend upon the character and amount of combustible material in the building construction itself and in the building contents. The latter will be determined in a general way by the occupancy. Office and residence occupancies generally support fires of shorter duration than manufacturing, merchandizing, or storage. The possible duration of fire is also determined to some extent by the floor load for which the building is designed, since it limits the amount of material subject to fire.

The effective fire duration, as far as it concerns a given building member, pertains only to the duration of the combustion taking place near enough to it to impart temperatures sufficiently high to cause failure under its supported load. This is not necessarily the same as the total duration of the fire within the building, or within a subdivision thereof, such as a building story. It is necessary to assume maximum probable conditions both with regard to building contents and air supply, as considered with respect to intensity and duration of a possible fire. Compensations and adjustments between intensity and duration may be necessary under some conditions in order to approximate a fire duration having intensity equivalent to that of the exposure in the fire test. Limitation of the degree of resistance to be developed to the requirements of present or immediate future occupancy is generally not justified, unless the building, its location or use is such that no change to a use involving a more severe fire condition is probable.

While it is outside of the scope of the present paper to attempt the determination of probable fire duration for various types of buildings and occupancies, it is a step in the application of the test results that will require careful consideration. The fire-resistance periods were derived under exacting conditions in point of test columns, test conditions, and interpretation of results, and the types of columns and protections to which they pertain are deemed to be fully adequate for resisting building fires of corresponding duration.

XIV. APPENDIXES**APPENDIX A.—Views of Columns**

Fig. No.	Test No.	Protection	Page
BEFORE AND AFTER FIRE TEST			
58.....	1 to 5.....	Unprotected structural-steel columns.....	217
59.....	6 to 8.....	do.....	218
60.....	9 to 13.....	Unprotected cast-iron and pipe columns.....	219
61.....	14 to 18.....	Partly protected structural-steel columns.....	220
62.....	19 to 22.....	do.....	221
63.....	23 to 27.....	Plaster on metal-lath protections.....	222
64.....	28A to 31.....	Concrete protections.....	223
65.....	32 to 34A.....	do.....	224
66.....	35 to 39, 45.....	do.....	225
67.....	40 to 44, 46, 47.....	do.....	226
68.....	48, 49, 50.....	Clay-tile protections.....	227
69.....	50A, 51, 51A.....	do.....	228
70.....	52, 53, 54.....	do.....	229
71.....	55 to 58.....	do.....	230
72.....	59, 60.....	do.....	231
73.....	61, 62, 63.....	do.....	232
74.....	64, 65, 66.....	Gypsum-block protections.....	233
75.....	67, 67A.....	do.....	234
76.....	68, 69.....	Brick protections.....	235
76.....	70.....	Reinforced-concrete column.....	235
77.....	71 to 75.....	Reinforced-concrete columns.....	236
78.....	76, 77.....	Plastered clay-tile protections.....	237
79.....	78.....	Longleaf pine column, protected.....	238
80.....	79.....	Longleaf pine column, unprotected.....	239
81.....	80.....	Longleaf pine column, protected.....	240
81.....	81.....	Longleaf pine column, unprotected.....	240
82.....	82, 83.....	Douglas fir columns, unprotected.....	241
BEFORE AND AFTER FIRE AND WATER TEST			
83.....	101, 102, 103.....	Concrete protections.....	242
84.....	104.....	Concrete protection.....	243
84.....	105.....	Clay-tile protection.....	243
85.....	106, 107.....	Clay-tile protections.....	244
86.....	108, 109.....	Gypsum-block protections.....	245
87.....	110.....	Plaster on metal-lath protection.....	249
88.....	111, 112.....	Reinforced-concrete columns.....	247
89.....	113.....	Reinforced-concrete column.....	248
89.....	114, 115.....	Cast-iron columns.....	248

FIG. 58.—Unprotected structural-steel columns, test Nos. 1 to 5, after fire test

No. 6.—I-beam and channel

No. 7.—Latticed angle

No. 7.—Detail of failure

No. 8.—Starred angle

FIG. 59.—Unprotected structural-steel columns, test Nos. 6 to 8, after fire test

FIG. 60.—*Unprotected cast iron and pipe columns, test Nos. 9 to 13, after fire test*

FIG. 61.—Partly protected structural-steel columns, test Nos. 14 to 18, after fire test

FIG. 62.—Partly protected structural-steel columns, test Nos. 19 to 22, after fire test

FIG. 63.—Plaster on metal-lath protections, test Nos. 23 to 27, after fire test

FIG. 64.—Concrete protections, test Nos. 28A to 31, after fire test

FIG. 65—Concrete protections, test Nos. 32 to 34A, after fire test

FIG. 66.—Concrete protections, test Nos. 35 to 39, and 45, after fire test

FIG. 67.—Concrete protections, test Nos. 40 to 44, 46, and 47, after fire test

FIG. 68.—Clay-tile protections, test Nos. 48, 49, and 50, before and after fire test

FIG. 69.—Clay-tile protections, test Nos. 50A, 51, and 51A, before and after fire test

FIG. 70.—Clay-tile protections, test Nos. 52, 53, and 54, before and after fire test

FIG. 71.—Clay-tile protections, test Nos. 55 to 58, before and after fire test

FIG. 72.—Clay-tile protections, test Nos. 59 and 60, before and after fire test

FIG. 73.—Clay-tile protections, test Nos. 61, 62, and 63, before and after fire test

FIG. 74.—Gypsum-block protections, test Nos. 64, 65, and 66, before and after fire test

FIG. 75.—Gypsum-block protections, test Nos. 67 and 67A, before and after fire test

FIG. 76.—Brick protections, test Nos. 68 and 69; reinforced-concrete column, test No. 70, before and after fire test

FIG. 77.—Reinforced-concrete columns, test Nos. 71 to 75, after fire test

FIG. 78.—Plastered clay-tile protections, test Nos. 76 and 77, before and after fire test

Before test

After test

Detail of cap failure

FIG. 79.—Longleaf pine column, protected, cap and pintle bearing, test No. 78, before and after fire test

Before test

After test

Detail of cap failure

FIG. 80.—Longleaf pine column, unprotected, cap and pintle bearing, test No. 79, before and after fire test

FIG 81.—Longleaf pine columns, steel-plate cap bearings, test Nos. 80 and 81, before and after fire test

FIG. 82.—Douglas fir columns, unprotected, test Nos. 82 and 83, before and after fire test

FIG. 83.—Concrete protections, test Nos. 101 to 103, after fire and water tests

FIG. 84.—Concrete protection, test No. 104; clay-tile protection, test No. 105, after fire and water tests

FIG. 85.—Clay-tile projections, test Nos. 106 and 107, after fire and water tests

FIG. 86.—Gypsum-block protections, test Nos. 108 and 109, after fire and water tests

FIG. 87.—Plaster on metal-lath protection, test No. 110, before and after fire and water tests

FIG. 88.—Reinforced-concrete columns, tests Nos. 111 and 112, after fire and water tests

FIG. 89.—Reinforced-concrete column, test No. 113; cast-iron columns, test Nos. 114 and 115, after fire and water tests

APPENDIX B.—Time-Temperature Curves

Fig. No.	Test No.	Columns and protections	Page
90.....	1 to 4.....	Unprotected structural steel.....	251
91.....	5 to 8.....	do.....	252
92.....	9 to 11.....	Unprotected cast iron.....	253
93.....	12, 13.....	Unprotected pipe.....	254
94.....	14 to 17.....	Partly protected structural steel.....	255
95.....	18, 22.....	do.....	256
96.....	19, 20, 21.....	do.....	257
97.....	23, 24.....	Double layer of plaster on metal lath.....	258
98.....	25, 26, 27.....	Single layer of plaster on metal lath.....	259
99.....	28.....	2-inch limestone concrete.....	260
100.....	28A.....	do.....	261
101.....	29.....	2-inch trap-rock concrete.....	262
102.....	30.....	2-inch Joliet gravel concrete.....	263
103.....	31.....	2-inch sandstone concrete.....	264
104.....	32, 32A.....	2-inch cinder concrete.....	265
105.....	33.....	4-inch limestone concrete.....	266
106.....	33A.....	do.....	267
107.....	34.....	4-inch granite concrete.....	268
108.....	34A.....	do.....	269
109.....	35.....	4-inch limestone concrete.....	270
103.....	36.....	2-inch trap-rock concrete.....	264
110.....	37.....	4-inch trap-rock concrete.....	271
111.....	38.....	2-inch Joliet gravel concrete.....	272
112.....	39.....	4-inch Meramec River gravel concrete.....	273
113.....	40.....	2-inch trap-rock concrete.....	274
114.....	41.....	4-inch limestone concrete.....	275
115.....	42.....	do.....	276
116.....	43, 44.....	2-inch sandstone concrete.....	277
112.....	45.....	2-inch Meramec River gravel concrete.....	273
117.....	46.....	2-inch trap-rock concrete (3½ inch outside of angles).....	278
101.....	47.....	2-inch cinder concrete.....	262
118.....	48, 49.....	Semifire-clay tile.....	279
119.....	50, 50A.....	Surface-clay tile.....	280
120.....	51, 51A.....	do.....	281
121.....	52, 53.....	Shale tile.....	282
122.....	54, 55.....	Semifire-clay tile.....	283
123.....	56.....	Semifire clay.....	284
123.....	57.....	Surface-clay tile.....	284
124.....	58, 59.....	do.....	285
125.....	60, 61.....	Semifire-clay tile.....	286
126.....	62, 63.....	Porous semifire-clay tile.....	287
127.....	64.....	4-inch gypsum block.....	288
128.....	65, 66.....	2-inch gypsum block.....	289
129.....	67.....	4-inch gypsum block.....	290
130.....	67A.....	do.....	291
118.....	68.....	2¼-inch surface-clay brick.....	279
131.....	69.....	3¾-inch surface-clay brick.....	292
132.....	70.....	Square, reinforced, limestone concrete.....	293
133.....	71.....	Square, reinforced, trap-rock concrete.....	294
134.....	72.....	Round, reinforced, limestone concrete.....	295

APPENDIX B.—Time-Temperature Curves—Continued

Fig. No.	Test No.	Columns and protections	Page
135.....	73.....	Round, reinforced, trap-rock concrete.....	296
136.....	74.....	Round, hooped, limestone concrete.....	297
137.....	75.....	Round, hooped, trap-rock concrete.....	298
138.....	76.....	Plastered-clay tile.....	299
139.....	77.....	do.....	300
140.....	78, 79, 82.....	Timber columns, cap and pntle.....	301
141.....	80, 81, 83.....	Timber columns, steel-plate cap.....	302
142.....	101 to 104.....	Concrete, fire and water tests.....	303
143.....	105 to 107.....	Clay tile, fire and water tests.....	304
143.....	108, 109.....	Gypsum block, fire and water tests.....	304
144.....	110.....	Plaster on metal lath, fire and water tests.....	305
145.....	111 to 113.....	Reinforced concrete, fire and water tests.....	306
145.....	114, 115.....	Unprotected cast iron, fire and water tests.....	306

NOTES

The upper curves in the diagrams, usually four in number, give indicated furnace temperatures at four points within the furnace chamber, two at 3 feet and two at 9 feet above the fireproofing line at the base of the test column, the lower location being designated by *L* and the upper *U*. The other letters of the curve designations indicate the corner of the furnace nearest to the given thermocouple location. Thus:

L-NW, lower northwest location

L-SE, lower southeast location

U-NE, upper northeast location

U-SW, upper southwest location

cf. Section VII, paragraph 2*a*, and Fig. 31.

The lower curves in the diagrams give temperatures at four general levels on the test column, and for one or two of the levels the temperatures are given for several points on the section, as shown by the location diagram given for each test. The levels are indicated by the letters *L*, *N*, *M*, and *T*, for locations 1½, 4½, 7½, and 10½ feet, respectively, above the fireproofing line at the base of the column. Thus for test No. 14, Fig. 94:

*B*₂, 1½-foot level at center of column flange

*N*₁, 4½-foot level at center of column web

*N*₂, 4½-foot level at center of column flange

*M*₁, 7½-foot level at center of column web

*M*₂, 7½-foot level at center of column flange

*M*₃, 7½-foot level at edge of column flange

*T*₂, 10½-foot level at center of column flange

cf. Section VII, paragraph 3*b*, and Fig. 31.

The arrows in Figs. 90 to 141 indicate the time of failure of the columns in the fire tests. The arrows in Figs. 142 to 145 indicate the time water was applied to the columns in the fire and water tests.

FIG. 90.—Furnace and column temperatures, unprotected structural-steel columns, test Nos. 1 to 4

FIG. 91.—Furnace and column temperatures, unprotected structural-steel columns, test Nos. 5 to 8

FIG. 92.—Furnace and column temperatures, unprotected cast-iron columns, test Nos. 9 to 11

FIG. 93.—Furnace and column temperatures, unprotected pipe columns, test Nos. 12 and 13

FIG. 94.—Furnace and column temperatures, partly protected structural-steel columns, test Nos. 14 to 17

FIG. 95.—Furnace and column temperatures, partly protected structural-steel columns, test Nos. 18 and 22

FIG. 96.—Furnace and column temperatures, partly protected structural-steel columns, test Nos. 19, 20, and 21

FIG. 97.—Furnace and column temperatures, plaster on metal-lath protections, test Nos. 23 and 24

FIG. 98.—Furnace and column temperatures, plaster on metal-lath protections, test Nos. 25, 26, and 27

FIG. 99.—Furnace and column temperatures, 2-inch limestone concrete protection, test No. 28

FIG. 100.—Furnace and column temperatures 2-inch limestone concrete protection, test No. 28A

FIG. 101.—Furnace and column temperatures, 2-inch trap-rock concrete protection, test No. 29; 2-inch cinder concrete protection, test No. 47

FIG. 102.—Furnace and column temperatures, 2-inch Joliet gravel concrete protection, test No. 30

FIG. 103.—Furnace and column temperatures, 2-inch sandstone concrete protection, test No. 31; 2-inch trap-rock concrete protection, test No. 36

FIG. 104.—Furnace and column temperatures, 2-inch cinder concrete protections, test Nos. 32 and 32A

FIG. 105.—Furnace and column temperatures, 4-inch limestone concrete protection, test No. 33

FIG. 106.—Furnace and column temperatures, 4-inch limestone concrete protection, test No. 33A

FIG. 107.—Furnace and column temperatures, 4-inch granite concrete protection, test No. 34

FIG. 108.—Furnace and column temperatures, 4-inch granite concrete protection, test No. 34A

FIG. 109.—Furnace and column temperatures, 4-inch limestone concrete protection, test No. 35

FIG. 110.—Furnace and column temperatures, 4-inch trap-rock concrete protection, test No. 37

FIG. III.—Furnace and column temperatures, 2-inch Joliet gravel concrete protection, test No. 38

FIG. 112.—Furnace and column temperatures, 4-inch Meramec River gravel concrete protection, test No. 39;
2-inch Meramec River gravel concrete protection, test No. 45

FIG. 114.—Furnace and column temperatures, 4-inch limestone concrete protection, test No. 41

FIG. 115.—Furnace and column temperatures, 4-inch limestone concrete protection, test No. 42

FIG. 116.—Furnace and column temperatures, 2-inch sandstone concrete protections, test Nos. 43 and 44

FIG. 117.—Furnace and column temperatures, 2-inch trap-rock concrete protection ($3\frac{1}{8}$ -inch of outside angles), test No. 46

FIG. 118.—Furnace and column temperatures, semifire-clay tile protections, test Nos. 48 and 49; 2¼-inch surface clay brick protection, test No. 68

FIG. 119.—Furnace and column temperatures, surface-clay tile protections, test Nos. 50 and 50A

FIG. 120.—Furnace and column temperatures, surface-clay tile protections, test Nos. 51 and 51A

FIG. 121.—Furnace and column temperatures, shale tile protections, test Nos. 52 and 53

FIG. 123.—Furnace and column temperatures, semifire-clay tile protection, test No. 56; surface-clay tile protection, test No. 57

FIG. 124.—Furnace and column temperatures, surface-clay tile protections, test Nos. 58 and 59

FIG. 126.—Furnace and column temperatures, porous semifire-clay tile protections, test Nos. 62 and 63

FIG. 127.—Furnace and column temperatures, 4-inch gypsum-block protection, test No. 64

FIG. 128.—Furnace and column temperatures, 2-inch gypsum-block protections, test Nos. 65 and 66

FIG. 129.—Furnace and column temperatures, 4-inch gypsum-block protection, test No. 67

FIG. 130.—Furnace and column temperatures, 4-inch gypsum-block protection, test No. 67A

FIG. 131.—Furnace and column temperatures, $3\frac{3}{4}$ -inch surface-clay brick protection, test No. 69

FIG. 132.—Furnace and column temperatures, square, reinforced, limestone concrete column, test No. 70

FIG. 133.—Furnace and column temperatures, square, reinforced, trap-rock concrete column, test No. 71

FIG. 134.—Furnace and column temperatures, round, reinforced, limestone concrete columns, test No. 72

FIG. 135.—Furnace and column temperatures, round, reinforced, trap-rock concrete column, test No. 73

FIG. 136.—Furnace and column temperatures, round, hooped, limestone concrete column, test No. 74

FIG. 137.—Furnace and column temperatures, round, hooped, trap-rock concrete column, test No. 75

FIG. 138.—Furnace and column temperatures, plastered clay-tile protection, test No. 76

FIG. 139.—Furnace and column temperatures, plastered clay-tile protection, test No. 77

FIG. 141.—Furnace and column temperatures, timber columns, steel-plate cap bearings, test Nos. 80, 81, and 83

FIG. 143.—Furnace and column temperatures, clay-tile and gypsum-block protections, fire and water tests, Nos. 105 to 109

FIG. 145.—Furnace and column temperatures, reinforced-concrete and unprotected cast-iron columns, fire and water tests, Nos. 111 to 115

APPENDIX C.—Deformation and Average Temperature Curves

Fig. No.	Test No.	Columns and protections	Page
146.....	1 to 5.....	Unprotected structural steel.....	308
147.....	9 to 11.....	Unprotected cast iron.....	309
147.....	12.....	Unprotected pipe.....	309
148.....	23, 24.....	Double layer plaster on metal lath.....	310
149.....	25, 26, 27.....	Single layer plaster on metal lath.....	311
150.....	28A.....	2-inch limestone concrete.....	312
151.....	29, 36.....	2-inch trap-rock concrete.....	313
152.....	31, 43.....	2-inch sandstone concrete.....	314
153.....	32, 32A.....	2-inch cinder concrete.....	315
154.....	33.....	4-inch limestone concrete.....	316
155.....	34.....	4-inch granite concrete.....	317
156.....	35.....	4-inch limestone concrete.....	318
151.....	36.....	2-inch trap-rock concrete.....	313
157.....	37.....	4-inch trap-rock concrete.....	319
158.....	38.....	2-inch Joliet gravel concrete.....	320
159.....	40.....	2-inch trap-rock concrete.....	321
160.....	41.....	4-inch limestone concrete.....	322
161.....	42.....	do.....	323
152.....	43.....	2-inch sandstone concrete.....	314
158.....	47.....	2-inch cinder concrete.....	320
162.....	48, 49.....	Semifire-clay tile.....	324
163.....	50A, 51A.....	Surface-clay tile.....	325
163.....	52, 53.....	Shale tile.....	325
164.....	54, 55.....	Semifire-clay tile.....	326
165.....	56.....	do.....	327
165.....	58.....	Surface-clay tile.....	327
166.....	60.....	Semifire-clay tile.....	328
167.....	62, 63.....	Porous semifire-clay tile.....	329
166.....	68.....	2 1/4-inch surface-clay brick.....	328
168.....	69.....	3 3/4-inch surface-clay brick.....	330
169.....	71.....	Square, reinforced, trap-rock concrete.....	331
170.....	72.....	Round, reinforced, limestone concrete.....	332
171.....	74.....	Round, hooped, limestone concrete.....	333

NOTE.—The unit deformation is the unit longitudinal expansion or compression observed over a 37-inch (94 cm) gage length immediately above the midheight of the columns, and is given on the diagrams in terms of parts in 100 000 to the same scale as centigrade temperature. Thus a unit deformation of 100 parts in 100 000 is 0.001. Deformations above the zero line denote expansion and below compression. The initial compression shown at the beginning of each test is due to the applied working load. Cf. Section VIII paragraph 2*b*, Fig. 33, and Section X, paragraph 6*a*.

FIG. 146.—Average temperature and deformation, unprotected structural-steel columns, test Nos. 1 to 5

FIG. 147.—Average temperature and deformation, unprotected cast-iron and pipe columns, test Nos. 9 to 12

FIG. 148.—Average temperature and deformation, plaster on metal-lath protections, test Nos. 23 and 24

FIG. 149.—Average temperature and deformation, plaster on metal-lath protections, test Nos. 25, 26, and 27

FIG. 150.—Average temperature and deformation, 2-inch limestone concrete protection, test No. 28A

FIG. 151.—Average temperature and deformation, 2-inch trap-rock concrete protections, test Nos. 29 and 36

FIG. 152.—Average temperature and deformation, 2-inch sandstone concrete protections, test Nos. 31 and 43

FIG. 153.—Average temperature and deformation, 2-inch cinder concrete protections, test Nos. 32 and 32A

FIG. 154.—Average temperature and deformation, 4-inch limestone concrete protection, test No. 33

FIG. 155.—Average temperature and deformation, 4-inch granite concrete protection, test No. 34

FIG. 156.—Average temperature and deformation, 4-inch limestone concrete protection, test No. 35

FIG. 157.—Average temperature and deformation, 4-inch trap-rock concrete protection, test No. 37

FIG. 158.—Average temperature and deformation, 2-inch Joliet gravel concrete protection, test No. 38; 2-inch cinder concrete protection, test No. 47

FIG. 159.—Average temperature and deformation, 2-inch trap-rock concrete protection, test No. 40

FIG. 160.—Average temperature and deformation, 4-inch limestone concrete protection, test No. 41

FIG. 161.—Average temperature and deformation, 4-inch limestone concrete protection, test No. 42

FIG. 162.—Average temperature and deformation, semifire-clay tile protection, test Nos. 48 and 49

FIG. 163.—Average temperature and deformation, surface-clay tile protections, test Nos. 50A and 51A; shale tile protections, test Nos. 52 and 53

FIG. 164.—Average temperature and deformation, semire-clay tile protections, test Nos. 54 and 55

FIG. 165.—Average temperature and deformation, semifire-clay tile protection, test No. 56; surface-clay tile protection, test No. 58

FIG. 166.—Average temperature and deformation, semifire-clay tile protection, test No. 60; $2\frac{1}{4}$ -inch surface-clay brick protection, test No. 68

FIG. 167.—Average temperature and deformation, porous semifire-clay tile protections, test Nos. 62 and 63

FIG. 168.—Average temperature and deformation, $\frac{3}{4}$ -inch surface-clay brick protection, test No. 69

FIG. 169.—Average temperature and deformation, square, reinforced, trap-rock concrete column, test No. 71

FIG. 170.—Average temperature and deformation, round, reinforced, limestone concrete column, test No. 72

FIG. 171.—Average temperature and deformation, round, hooped, limestone concrete column, test No. 74

APPENDIX D.—Tables of Auxiliary Tests of Materials

Table No.	Table title	Page
5.....	Tension tests of steel.....	335-7
6.....	Tension tests of metal lath.....	338
7.....	Hardness tests of steel.....	338
8.....	Chemical analyses of steel.....	339
9.....	Transverse and tension tests of cast iron.....	339
10.....	Chemical analyses of cast iron.....	339
11.....	Tests of Portland cement by Bureau of Standards, Washington laboratory.....	340
12.....	Tests of Portland cement by Bureau of Standards, Pittsburgh laboratory.....	341
13.....	Tests of Portland cement by R. W. Hunt & Co., Chicago laboratory.....	342
14.....	Chemical analyses of sands.....	342
15.....	Physical properties of concrete sands.....	343
16.....	Physical properties of mortar and plaster sands.....	343
17.....	Mortar strength of concrete sands.....	344
18.....	Chemical analyses of coarse aggregates.....	344
19.....	Physical properties of coarse aggregates.....	344
20.....	Mineralogical composition of concrete aggregates.....	345
21.....	Compressive strength and modulus of elasticity of concrete in columns and coverings.....	346-9
22.....	Compressive strength and modulus of elasticity of concrete in head protections.....	350
23.....	Tests of lime.....	351
24.....	Tests of calcined gypsum.....	351
25.....	Compressive strength of Portland-cement and lime plaster.....	352
26.....	Compressive strength of clay-tile mortar.....	353-4
27.....	Compressive strength of gypsum mortar and plaster.....	355
28.....	Strength tests of mortar and plaster.....	356
29.....	Compressive strength of gypsum filling.....	354
30.....	Classification and description of hollow clay tile.....	357
31.....	Compressive strength of hollow clay tile.....	358-9
32.....	Transverse strength of hollow clay tile.....	360
33.....	Temperatures of vitrification and fusion of clay tile and brick.....	361
34.....	Porosity and absorption of Chicago common brick.....	361
35.....	Compressive strength of Chicago common brick.....	362
36.....	Transverse strength of Chicago common brick.....	362
37.....	Porosity of gypsum block.....	363
38.....	Compressive strength of solid gypsum block.....	363
39.....	Transverse strength of solid gypsum block.....	364
40.....	Transverse strength of gypsum wall board.....	364

NOTE.—For details of tests see Section V, pp. 58-74.

TABLE 5.—TENSION TESTS OF STEEL

Column Test No.	Specimen No.	SECTION	Location of Specimen	Yield Point, Lb. per Sq. In.	Ultimate Strength, Lb. per sq. In.	Elongation in 2 In., Percent	Reduction in Area, Percent
1	T-1	Rolled H.....	Edge of flange.....	40025	62600	41.0	57.8
1	T-1A	do.....	Junction of web and flange.....	37500	60000	37.0	61.5
1	T-1B	do.....	Center of web.....	42300	62600	40.0	52.8
14	T-14	do.....	do.....		76300	32.0	61.0
28	T-28	do.....	do.....	34850	57400	37.5	65.6
28A	T-28A	do.....	do.....	37500	68250	32.5	59.2
30	T-30	do.....	do.....	34100	57600	25.0	67.0
31	T-31	do.....	do.....	38000	58600	29.0	60.0
32	T-32	do.....	do.....	43900	69400	17.5	61.0
33	T-33	do.....	do.....	35000	59000	27.5	66.0
33A	T-33A	do.....	do.....	37200	71500	24.0	54.0
34	T-34	do.....	do.....	48050	69800	25.0	55.0
34A	T-34A	do.....	do.....	32200	62700	21.0	66.0
35	T-35	do.....	do.....	34000	66000	16.0	63.0
48	T-48	do.....	do.....	39200	60800	25.0	61.0
49	T-49	do.....	Edge of flange.....	42900	64700	40.0	60.3
49	T-49A	do.....	Junction of web and flange.....	37750	60400	40.0	68.5
49	T-49B	do.....	Center of web.....	44700	62300	35.0	54.2
67A	T-67A	do.....	Edge of flange.....	41260	62230	43.0	62.8
68	T-68	do.....	do.....	35300	56720	47.0	64.3
16	T-16	Plate and Angle.....	Edge of angle.....	42850	59800	27.0	62.0
23	T-23	do.....	Center of plate.....	35500	58900	28.5	56.0
36	T-36	do.....	do.....	32100	56600	26.5	65.0
50A	T-50A	do.....	do.....	37650	60200	28.0	53.0
51	T-51	do.....	Edge of angle.....	34200	60500	25.0	61.0
105	T-105	do.....	Center of plate.....	29450	55300	27.5	69.0
24	T-24	Plate and Channel.....	Center of channel web.....		61300	31.0	58.0
38	T-38	do.....	do.....	40500	67600	23.0	54.0
39	T-39	do.....	do.....	36600	58900	32.0	66.0
53	T-53	do.....	do.....	39000	62500	26.0	57.0
65	T-65	do.....	Edge of channel flange.....	37850	57680	39.0	58.5
65	T-65-1	do.....	Junction of flange and web.....		51600	34.0	43.1
107	T-107	do.....	Center of channel web.....	46300	63400	27.0	54.0

TABLE 5.—TENSION TESTS OF STEEL—Continued

Column Test No.	Specimen No.	SECTION	Location of Specimen	Yield Point, Lb. per Sq. In.	Ultimate Strength, Lb. per Sq. In.	Elongation in 2 In., Percent	Reduction in Area, Percent
18	T-18	Latticed Channel	Center channel web	44400	70900	23.0	55.0
40	T-40	do	do	40500	62200	28.0	63.0
19	T-19	Z-Bar and Plate	Center of web	35700	60500	25.0	55.0
25	T-25	do	do	32900	52200	28.0	65.0
32A	T-32A	do	do	35050	57200	30.5	79.0
41	T-41	do	do	31000	54400	23.0	62.0
42	T-42	do	do	44300	69700	17.5	58.0
55	T-55	do	do	43000	65000	19.0	55.0
56	T-56	do	do	37100	54200	25.0	63.0
20	T-20	I-Beam and Channel	Channel web	44800	64800	26.0	55.0
21	T-21	do	do	40500	62200	28.0	52.0
57	T-57	do	do	42100	65300	24.0	56.0
59	T-59	do	do	44000	63400	19.0	55.0
22	T-22	Latticed Angle	Edge of angle	42900	63700	18.0	63.0
46	T-46	do	do	47800	66300	35.0	64.0
8	T-8	Star	Edge of angle	39600	69300	24.0	62.0
45	T-45	do	do	43200	61100	26.5	64.0
70	70	Vertically Reinforced Concrete	1 in. sq. deformed reinforcing bar . .	*34200	59500	22.8	63.4
70	70A	do	do	27340	47860	30.3	66.7
72	72	do	do	*36000	54000	26.8 (a)	62.0
72	72A	do	do	27420	46600	31.8	67.3
73	73-1	do	do	*33500	56000	27.0	62.5
73	73-1A	do	do	*35000	55750	25.8	64.5
73	73-2	do	do	*27730	46650	33.8	68.3
111	111-1	do	do	*35000	56000	24.2 (a)	64.5
111	111-1A	do	do	*35000	56500	26.5 (a)	64.5
111	111-2	do	do	35100	52570	32.5	67.8
112	112-1	do	do	*51000	68500	22.5	62.0
112	112-1A	do	do	*50000	65000	21.9	62.0
112	112-2	do	do	20870	34750	41.0	77.2

(a) Elongation was measured in 8-in. gage length instead of 2 in.

*Tested on reduced machined sections with shouldered ends.

TABLE 5.—TENSION TESTS OF STEEL—Concluded

Column Test No.	Specimen No.	SECTION	Location of Specimen	Yield Point, Lb. per Sq. In.	Ultimate Strength, Lb. per Sq. In.	Elongation in 2 In., Percent	Reduction in Area, Percent
74	74-1	Hooped Reinforced Concrete	$\frac{3}{4}$ in. sq. deformed reinforcing bar	*40000	57500	28.3	64.2
74	74-1A	do.	do.	*36820	56700	31.0	64.6
74	74-2	do.	do.	36870	51680	35.8	67.7
74	74-3	do.	do.	*38800	54700	29.3	68.9
74	74-3A	do.	do.	40000	53500	31.0	64.7
74	74-4	do.	do.	*40850	59350	21.8	60.5
74	74-4A	do.	do.	*42300	59200	30.3	66.0
75	75-1	do.	do.	*37800	57700	29.5	64.4
75	75-1A	do.	do.	*39000	57500	29.8	65.8
75	75-2	do.	do.	37750	52870	35.8	64.6
113	113-1	do.	do.	*39800	58700	29.3	63.2
113	113-1A	do.	do.	*39800	59200	30.3	63.2
113	113-2	do.	do.	37950	54950	31.3	64.8
	Q-1	do.	$\frac{1}{4}$ in. high carbon wire hooping...	55500	86900		
	Q-2	do.	do.	49000	87600		
	Q-3	do.	do.	54000	87600		
	K-1	Ties in partly protected columns.	$\frac{1}{2}$ in. plain round bar	36680	58700	29.0	68.4
...	K-2	do.	do.	33930	54600	33.0	66.2
...	K-3	do.	do.	33000	48200	30.3	72.2
...	L-1	do.	$\frac{3}{8}$ in. plain round bar	40420	57700	27.5	68.9
...	L-2	do.	do.	40230	56950	27.3	70.5
...	L-3	do.	do.	34400	56170	28.5	70.7
...	M-1	Ties in vertically reinforced concrete columns.	$\frac{1}{4}$ in. plain round bar	41150	58500		63.5
...	M-2	do.	do.	41410	60000		67.4
...	M-3	do.	do.	41000	62500		67.8
...	M-4	do.	do.	44000	59100	35.0	70.4
...	M-5	do.	do.	43600	61200	36.0	68.6
...	M-6	do.	do.	43050	57500	38.0	71.6
...	N-1	Ties in concrete protections	No. 5 (B. & S. ga.) bright basic wire		76200		67.5
...	N-2	do.	do.		73800		67.5
...	N-3	do.	do.		75500		67.5

* Tested on reduced machined sections with shouldered ends.

Five Tests of Building Columns

TABLE 6.—TENSION TESTS OF METAL LATH

Each value is the average of three tests

Specimen No.	Kind of Lath	Width of Specimen	Ultimate Strength, Lb. per In. Width
R-1	No. 24 Gage Expanded Metal.....	3 in.	150
R-2	No. 24 High-Ribbed Metal Lath.....	2½ in.	520
R-3	Woven Wire Lath.....	3 in. (7 strands)	256

TABLE 7.—HARDNESS TESTS OF STEEL

Column Test No.	Specimen No.	Section	Location of Specimen	Brinell Hardness No.
2	H-2	Plate and Angle.....	Edge of angle.....	113
17	E-17	do.....	do.....	121
37	E-37	do.....	do.....	118
51A	E-51A	do.....	do.....	118
103	H-103	do.....	do.....	121
104	H-104	do.....	do.....	125
106	H-106	do.....	do.....	131
110	H-110	do.....	do.....	126
60	H-60	Latticed angle.....	do.....	123
				Scleroscope Hardness No.
3	HC-3	Plate and Channel.....	Center of channel web.....	23
3	HP-3	do.....	Plate.....	24
38	H-38	do.....	do.....	21
107	H-107	do.....	Center of channel web.....	24
66	H-66	Latticed channel.....	Center of web.....	23
5	H-5	Z-bar and Plate.....	Plate.....	26
41	H-41	do.....	do.....	22
56	H-56	do.....	do.....	24
6	H-6	I-beam and Channel.....	Channel web.....	26
43	H-43	do.....	Center of I-beam web.....	22
57	H-57	do.....	do.....	22
58	H-58	do.....	do.....	22
7	H-7	Latticed angle.....	Edge of angle.....	25
26	B-26	do.....	do.....	20
60	B-60	do.....	do.....	30
61	B-61	do.....	do.....	22

TABLE 8.—CHEMICAL ANALYSES OF STEEL

Column Test No.	Specimen No.	Carbon	Manganese	Phosphorus	Sulphur	Silicon
1	1	0.20	0.54	0.010	0.021	0.03
1	1A	0.18	0.54	0.015	0.028	0.01
1	1B	0.21	0.54	0.012	0.025	0.04
14	14	0.16	0.72	0.023	0.034	0.03
65	65	0.18	0.40	0.011	0.036	0.04
65	65-1	0.20	0.41	0.010	0.038	0.05
65	65-2	0.21	0.42	0.008	0.029	0.04
68	68	0.12	0.57	0.010	0.022	0.02
22	Rivets	0.09		0.007	0.050	
46	Rivets	0.11		0.024	0.064	

TABLE 9.—TRANSVERSE AND TENSION TESTS OF CAST IRON

Column Test No.	Specimen No.	Modulus of Rupture, Lb. per Sq. In.	*Ultimate Tensile Strength, Lb. per Sq. In.
9	T-9	47800	23230
10	T-10	45800	29470
11	T-11	43500	23900
27	T-27	64700	26700
47	T-47		23050
62	T-62	56100	28800
**10A	1C1A		17600
114 and	1C1B	39400	18300
115	1C1C	36200	17100
do	2C1A	45500	20300
do	2C1B	46300	21700
do	2C1C	45700	21600

*Tension specimens cut from ends of transverse specimens after test.

**Specimens cut from duplicate test column.

TABLE 10.—CHEMICAL ANALYSIS OF CAST IRON

Column Test No.	Specimen No.	Carbon	Manganese	Phosphorus	Sulphur	Silicon	Graphite
9	T-9	0.61	0.47	0.57	0.090	1.86	2.66
10	T-10	0.67	0.50	0.57	0.090	1.86	2.77
11	T-11				0.087		
27	T-27				0.086		
62	T-62				0.082		

TABLE 11.—TESTS OF PORTLAND CEMENT

By Bureau of Standards' Washington Laboratory

DATE OF REPORT	September 22, 1916							Dec. 19, 1916
SAMPLE No.	1	2	3	4	5	B-1	B-2	12*
Chemical Analysis:								
Silica (SiO ₂).....	20.34%	19.46%	20.36%	20.28%	20.40%	20.46%	20.82%	20.82%
Oxide of Iron (Fe ₂ O ₃).....	2.72%	2.70%	2.60%	2.70%	2.80%	2.75%	2.52%	2.10%
Oxide of Aluminum (Al ₂ O ₃).....	6.98%	7.18%	7.24%	6.86%	6.74%	6.15%	6.38%	7.14%
Lime (CaO).....	63.68%	62.12%	63.42%	64.32%	63.24%	62.86%	64.68%	59.50%
Magnesia (MgO).....	1.13%	1.21%	0.90%	1.18%	1.12%	1.22%	1.28%	3.46%
Sulphuric Anhydride (SO ₃).....	1.34%	1.44%	1.45%	1.45%	1.46%	1.38%	1.45%	1.50%
Loss of Ignition.....	4.53%	3.43%	3.17%	3.17%	3.22%	3.40%	2.26%	4.48%
Insoluble Residue.....	0.10%	0.13%	0.17%	0.20%	0.13%	0.16%	0.13%	0.70%
Specific Gravity:								
As Received.....	3.06	3.09	3.08	3.08	3.08	3.10	3.12	
After Ignition.....	3.17	3.18	3.16	3.22	3.15			3.17
Fineness:								
Passing 110-Mesh Sieve.....	94.8%	94.4%	97.0%	96.0%	97.0%	97.4%	96.4%	99.2%
Passing 200-Mesh Sieve.....	77.6%	76.0%	77.4%	78.4%	77.0%	78.5%	78.1%	75.0%
Tensile Strength, Lb. per Sq. In.:								
Neat, 7 Days.....	434	552	551	587	680	622	736	435
	471	512	641	645	527	564	575	510
	575	610	562	604	577	670	587	425
Neat, 28 Days.....	730	690	670	690	545	715	725	740
	635	655	710	640	670	800	675	625
	585	595	730	575	705	700	665	675
1 Cement: 3 Sand—								
7 Days.....	178	227	222	182	228	121	193	186
	178	229	182	207	229	177	247	140
	169	206	206	213	243	209	238	165
28 Days.....	280	295	325	325	330	265	365	375
	310	300	310	335	340	280	380	380
	260	265	310	330	290	325	330	340
Time of Set, hr.—min.:								
Initial.....	7-30	7-15	7-30	7-10	7-20	7-30	7-25	7-0
Final.....	11-30	11-30	11-55	11-45	11-40	11-50	11-00	10-30
Soundness: Pats—								
28 Days in Air.....	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.
28 Days in Water.....	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.
5 Hr. in Steam.....	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.	O. K.
Water Used:								
Neat.....	25.0%	25.0%	25.0%	25.0%	25.0%	25.0%	25.0%	23.0%
1:3 Mortar.....	10.7%	10.7%	10.7%	10.7%	10.7%	10.7%	10.7%	10.3%

*Portland cement used in pipe column, Test No. 12.

TABLE 13.—TESTS OF PORTLAND CEMENT

By R. W. Hunt & Company

DATE OF REPORT	May 2, 1917		
SAMPLE No.	H-1	H-2	H-3
Specific Gravity:			
As Received.....	3.14	3.13	3.13
Fineness:			
Passing 100-Mesh Sieve.....	95.4%	96.0%	96.1%
Passing 200-Mesh Sieve.....	76.4%	78.0%	77.2%
Tensile Strength, Lb. per Sq. In.:			
Neat, 24 Hr.....	305	355	345
	320	330	330
	335	350	300
Neat, 7 days.....	690	625	675
	640	640	650
	700	700	695
Neat, 28 Days.....	700	700	695
	730	765	710
	765	725	725
1 Cement, 3 Sand:			
7 Days.....	310	330	300
	330	310	320
	335	345	335
28 Days.....	410	385	410
	380	395	400
	405	410	385
Time of Set, hr.—min.:			
Initial.....	5-45	5-40	5-40
Final.....	8-05	8-0	8-10
Soundness: Pats—			
5 Hr. in Steam.....	O. K.	O. K.	O. K.
Water Used:			
Neat.....	23.0%	23.0%	23.0%
1 : 3 Mortar.....	10.3%	10.3%	10.3%

TABLE 14.—CHEMICAL ANALYSES OF SANDS

SAND	Insoluble in Acid	Al ₂ O ₃ and Fe ₂ O ₃	CaO	MgO	CO ₂	*Loss on Ignition
Fox River.....	54.51	1.34	14.90	8.24	21.22	
Joliet.....	52.10	1.19	15.46	8.97	23.06	
Meramec River.....	98.66	0.58	0.14	0.06	0.11	
Long Island.....	97.98	0.96	0.04	0.18	0.25	
Pelee Island.....	64.70	1.66	17.03	1.34	15.68	
Plum Island.....	98.18	0.66	0.16	0.12	0.18	
Cambridge.....	96.93	1.63	0.17	0.90	0.25	0.97
Coarse lake.....	83.53	6.69	3.31	1.21	3.30	3.63
Fine lake.....	87.69	5.38	2.17	0.93	1.95	2.28
Beach.....	85.45	5.60	2.35	1.02	2.29	2.79
Bank.....	91.30	5.18	0.09	0.51	0.22	0.54

*Not including CO₂.

TABLE 15.—PHYSICAL PROPERTIES OF CONCRETE SANDS

SAND.	Specific Gravity	Weight per Cu. Ft. (Dry), Lb.	Computed Voids, Per cent	Granular Analysis—Percent Passing Sieve No.											
				3	4	6	8	10	14	20	28	35	48	65	100
Fox River.....	2.62	110.0	33.0	99.4	93.5	86.9	79.6	71.4	63.7	53.2	37.1	22.9	11.0	5.3	2.5
Joliet.....	2.62	96.3	41.0	98.4	95.0	89.5	82.7	73.6	64.5	52.5	33.6	15.0	3.4	0.8	0.3
Meramec River.....	2.63	100.0	39.0	99.4	98.8	96.4	92.5	87.6	83.0	78.4	71.7	56.2	22.1	5.5	0.9
Long Island.....	2.66	97.5	41.0	99.5	98.2	95.5	89.0	80.3	71.8	61.5	47.6	32.5	15.2	6.6	2.4
Pelee Island.....	2.66	112.0	33.0	100.0	100.0	93.8	82.9	69.3	58.7	48.6	37.8	27.9	15.6	8.6	3.1
Plum Island.....	2.63	92.1	44.0	100.0	99.8	99.5	98.6	96.8	89.0	69.2	34.2	7.2	0.8	0.3	0.1
Cambridge.....	2.65	109.4	34.0	91.5	88.6	85.2	81.0	75.8	70.0	60.5	46.3	32.9	16.6	9.0	5.9
Sieve Opening, In.....				0.263	0.185	0.131	0.093	0.065	0.046	0.0328	0.0232	0.0164	0.0116	0.0082	0.0058

TABLE 16.—PHYSICAL PROPERTIES OF MORTAR AND PLASTER SANDS

SAND	Specific Gravity	Weight per cu. ft. (Dry), Lb.	Computed Voids, Percent	Granular Analysis—Percent Passing Sieve No.							
				6	10	20	40	60	80	100	200
Coarse Lake.....	2.62	104.6	36.0	98.5	94.0	72.9	42.5	12.6	5.4	1.9	0.7
Fine Lake.....	2.62	100.3	39.0	99.5	98.3	94.2	76.1	42.6	16.4	10.4	0.6
Beach.....	2.63	91.7	44.0	99.0	97.0	90.7	75.5	48.8	31.3	17.5	0.3
Bank.....	2.64	99.6	39.0	99.9	99.9	99.7	98.5	89.7	47.5	20.9	
Sieve Opening, In.....				0.131	0.065	0.0328	0.015	0.0087	0.0068	0.0058	0.0029

TABLE 17.—MORTAR STRENGTH TESTS OF CONCRETE SANDS

Proportion: 1 part Tidewater Portland cement, 3 parts sand (by weight)

Test specimens stored in water. Each result is the average of three tests.

SAND	Percent Water	Age, Days	Tensile Strength, Lb. per Sq. In.	Compressive Strength, Lb. per Sq. In.
Fox River.....	10.0	7	283	3390
Fox River.....	10.0	28	437	4980
Joliet.....	12.0	7	397	1347
Joliet.....	12.0	28	379	2920
Meramec River.....	13.5	7	171	921
Meramec River.....	13.5	28	346	1540
Long Island.....	13.0	7	176	954
Long Island.....	13.0	28	303	1910
Peleé Island.....	11.0	7	245	763
Peleé Island.....	11.0	28	412	3320
Plum Island.....	12.0	7	258	1600
Plum Island.....	12.0	28	269	1621

TABLE 18.—CHEMICAL ANALYSES OF COARSE AGGREGATES

AGGREGATE	Insoluble in Acid	Silica, Soluble	Al ₂ O ₃ and R ₂ O ₃	CaO	MgO	CO ₂	SO ₃	Loss* on Ignition
Chicago limestone.....	5.00		0.44	29.16	20.40	45.35		1.00
Joliet gravel.....	21.28	0.24	0.82	23.30	16.29	35.70		1.16
Meramec R. gravel.....	96.06	0.50	2.36	Trace	Trace		None	0.92
New York trap rock.....	86.08	0.24	8.22	1.05	1.93			
Rockport granite.....	97.31a							
Cleveland sandstone.....	95.59b							
Hard coal cinders.....	90.88	0.13	0.62	0.41	0.25		0.03	6.97
Westfield trap rock.....	84.63	1.48	7.48	1.63	2.85	2.65		0.19

* Other than CO₂.

(a) Soluble content too small to warrant further analysis. Tests indicate a normal granite.

(b) Normal sandstone.

TABLE 19.—PHYSICAL PROPERTIES OF COARSE AGGREGATES

AGGREGATE	Apparent Specific Gravity	Weight per Cu. Ft. (Dry), Lb.	Computed Voids, Percent	Percent Passing Sieves Size of Openings in In.							
				1½	1	¾	½	¼	0.131	0.093	
Chicago limestone.....	2.61	86.50	46.9	100.0	98.5	76.5	33.0	6.0	0.14	0.11	
Joliet gravel.....	2.61	101.25	37.9	100.0	95.7	83.5	56.5	6.5			
Meramec R. gravel.....	2.45	95.0	38.0	100.0	98.0	91.5	62.0	19.0	6.60	3.60	
New York trap rock.....	2.87	95.6	46.6	100.0	98.3	83.3	50.8	11.8	0.20	0.10	
Rockport granite.....	2.61	87.25	46.3	100.0	100.0	100.0	86.5	29.0	7.30	1.60	
Cleveland sandstone.....	2.15	78.50	41.6	98.7	83.7	45.0	20.0	9.0	7.30	7.00	
Hard coal cinders.....	1.52	50.75	46.5	98.8	93.5	89.0	68.7	37.3	36.0	34.0	
Westfield trap rock.....	2.94	90.0	51.0	100.0	100.0	100.0	100.0	53.3	38.8	19.4	

TABLE 20.—MINERALOGICAL COMPOSITION OF CONCRETE AGGREGATES

AGGREGATE	Calcite, CaCO ₃	Dolomite, CaMg(CO ₃) ₂	Clay, Hydrous Aluminum Silicates	Limonite (Iron Rust), 2Fe ₂ O ₃ ·3H ₂ O	Chert, SiO ₂ ·XH ₂ O	Quartz, SiO ₂	Feldspars, KAlSi ₃ O ₈ NaAlSi ₃ O ₈ CaAl ₂ Si ₂ O ₈	Ferromagnesians, (Fe, Mg, Ca) Al ₂ SiO ₅ (Fe, Mg, Ca) SiO ₃	Mag- netite, Fe ₃ O ₄	Other Minerals. Amounts, Where not Given, Are Less Than 1 Percent
Fox River sand.....	6	38	(2 in calcite and dolomite)	0.5	7	32	11	4	0.5	white mica, garnet titanite
Joliet sand.....	5	42	(3 in calcite and dolomite)	trace		42	7	3	trace	garnet, cinders
Meramec River sand..	1			trace	16	83				
Long Island sand....	trace		1 (slate)			82	14	2	0.7	garnet, cinders
Pele Island sand.....	27	5	4		4	47	9	3	0.5	garnet, muscovite corundum
Plum Island sand.....	trace					83	14	2	0.3	garnet, muscovite
Chicago limestone.....		94.5	5	0.5						
Joliet gravel.....		86	(5 in dolomite)			7	5	2		
Meramec River gravel..	trace			trace	98	2				
New York trap rock...						trace	71	26	2	apatite
Rockport granite.....			trace	trace		35	59	5	trace	zircon
Cleveland sandstone..			1.5	0.5		98				
Hard coal cinders.....										85 slag and cinder 10 anthracite 5 ash

20184°—21—23

Five Tests of Building Columns

TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS

Cylinder No.	Kind of Concrete	Age, Days	Per-cent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
22-1	1:2.4 Chicago limestone.....	28		912			
22-2	do.....	29		1049			
22-3	do.....	407		1888	2,060,000	2,490,000	2,780,000
22-4	do.....	407		1780	2,390,000	2,400,000	2,310,000
28-1	do.....	28		2935			
28-3	do.....	28		3308			
28-2	do.....	438		3830			5,270,000
28-4	do.....	438		3470	4,470,000	4,620,000	4,400,000
28A-1	do.....	28		1738			
28A-3	do.....	28		1647			
28A-2	do.....	439		2370	4,300,000	3,960,000	3,550,000
28A-4	do.....	439		2213		3,900,000	3,500,000
33-1	do.....	25	12.1	1489			
33-3	do.....	25	12.1	1728			
33-2	do.....	452	12.1	1956	2,660,000	2,300,000	1,990,000
33-4	do.....	452	12.1	2273	4,000,000	3,730,000	3,350,000
33A-1	do.....	29	12.9	1255			
33A-3	do.....	29	12.9	1676			
33A-2	do.....	456	12.9	1574	2,950,000	2,680,000	2,450,000
33A-4	do.....	456	12.9	1881	3,350,000	3,100,000	2,950,000
70-1	do.....	28		2027			
70-3	do.....	28		1778			
70-2	do.....	434		3182	4,280,000	3,950,000	3,860,000
70-4	do.....	434		2743	3,750,000	3,730,000	3,490,000
72-1	do.....	29	13.1	1010			
72-3	do.....	29	12.7	1582			
72-2	do.....	521	13.1	1656	2,730,000	2,500,000	2,250,000
72-4	do.....	521	12.7	2114			2,970,000
74-1	do.....	28	12.0	704			
74-3	do.....	28	11.8	739			
74-2	do.....	523	12.0	922	1,420,000	760,000	250,000
74-4	do.....	523	11.8	935	1,760,000	1,160,000	490,000
101-5	do.....	28	12.7	1495			
101-6	do.....	520	12.7	2485	7,970,000	5,990,000	5,080,000
102-1	do.....	28	12.2	1455			
102-2	do.....	514	12.2	2363	4,050,000	3,760,000	3,740,000
111-1	do.....	28	12.5	1554			
111-2	do.....	520	12.5	2735	5,000,000	4,320,000	4,450,000
112-5	do.....	28	10.6	1493			
112-6	do.....	521	10.6	2042	3,640,000	3,460,000	3,350,000
19-1	1:3.5 Chicago limestone.....	29		2056			
19-2	do.....	29		2168			
19-3	do.....	414		2561	4,000,000	3,940,000	3,790,000
19-4	do.....	414		1880	3,350,000	3,160,000	3,120,000
35-1	do.....	32	12.8	1284			
35-3	do.....	32	12.8	1618			
35-2	do.....	505	12.8	1540	3,270,000	2,940,000	2,520,000
35-4	do.....	505	12.8	1468	4,070,000	3,760,000	3,470,000
41-1	do.....	29	13.0	1355			
41-3	do.....	29	13.4	1132			
41-2	do.....	452	13.0	1485	2,830,000	2,520,000	2,300,000
41-4	do.....	452	13.4	1313	2,590,000	2,450,000	2,200,000
42-1	do.....	28	13.3	1169			
42-3	do.....	27	12.6	1042			
42-2	do.....	450	13.3	1943	3,450,000	3,180,000	3,150,000
42-4	do.....	450	12.6	1216	2,350,000	2,000,000	1,640,000
55-1	do.....	29	13.7	718			
55-3	do.....	29	15.2	577			
55-2	do.....	483	13.7	794	1,330,000	490,000	
55-4	do.....	483	15.2	678	1,630,000		
56-1	do.....	28	13.3	811			
56-3	do.....	28	12.9	614			
56-2	do.....	492	13.3	682	1,280,000	400,000	
56-4	do.....	492	12.9	869	1,280,000	750,000	
57-1	do.....	28	12.7	912			
57-3	do.....	28	12.9	947			
57-2	do.....	483	12.7	921	1,600,000	900,000	350,000

TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS—Continued

Cylinder No.	Kind of Concrete	Age, Days	Percent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
57-4	1:3:5 Chicago limestone.....	483	12.9	1040	2,030,000	1,550,000	1,000,000
76-1	do.....	59	13.7	523	190,000		
76-2	do.....	59	13.7	688	340,000	240,000	
77-1	do.....	59	13.7	842	1,360,000	950,000	
77-2	do.....	59	13.7	508	210,000		
106-1	do.....	28	13.9	767			
106-4	do.....	29	13.9	813			
106-2	do.....	532	13.9	820	1,770,000	1,200,000	
106-3	do.....	532	13.9	946	2,230,000	1,460,000	1,060,000
16-1	1:2:4 New York trap.....	29		1023			
16-2	do.....	29		1163			
16-3	do.....	416		2512	4,750,000	4,340,000	3,860,000
16-4	do.....	416		1219	1,100,000	650,000	330,000
18-1	do.....	28		1158			
18-2	do.....	28		1764			
18-3	do.....	419		1529	1,900,000	1,570,000	1,030,000
18-4	do.....	419		1884	3,320,000	2,600,000	1,840,000
29-1	do.....	29	13.1	1494			
29-3	do.....	29	13.6	1239			
29-2	do.....	437	13.1	1807	2,570,000	2,390,000	2,210,000
29-4	do.....	437	13.6	1985	3,140,000	2,870,000	2,640,000
36-1	do.....	29	12.2	1490			
36-3	do.....	29	12.0	1548			
36-2	do.....	445	12.2	2284	2,750,000	2,740,000	2,600,000
36-4	do.....	445	12.0	2118	3,100,000	2,960,000	2,650,000
37-1	do.....	30	12.6	995			
37-3	do.....	30	12.7	1668			
37-2	do.....	504	12.6	2278	2,780,000	2,620,000	2,450,000
37-4	do.....	504	12.7	1103	820,000	530,000	350,000
40-1	do.....	28	11.2	1797			
40-3	do.....	28	11.1	1780			
40-2	do.....	502	11.2	2180	3,140,000	2,960,000	2,750,000
40-4	do.....	502	11.1	2300	4,170,000	3,450,000	3,150,000
46-1	do.....	28	13.9	1418			
46-3	do.....	29	13.9	1641			
46-2	do.....	455	13.9	1659	1,870,000	1,700,000	1,460,000
46-4	do.....	455	13.9	2082	2,730,000	2,510,000	2,300,000
60-1	do.....	27	14.6	948			
60-3	do.....	27	14.6	1028			
60-2	do.....	490	14.6	1490	2,510,000	2,140,000	1,810,000
60-4	do.....	490	13.8	1039	1,940,000	1,530,000	1,200,000
71-1	do.....	29	13.7	1572			
71-3	do.....	29	13.0	957			
71-2	do.....	451	13.7	1446	2,450,000	1,970,000	1,650,000
71-4	do.....	451	13.0	1109	1,000,000	400,000	230,000
73-1	do.....	28		2239			
73-3	do.....	28		2484			
73-2	do.....	443		2658	4,180,000	3,960,000	3,720,000
73-4	do.....	443		3224	3,900,000	4,060,000	3,830,000
75-1	do.....	28	10.0	1361			
75-3	do.....	28	10.3	1756			
75-2	do.....	462	10.0	1918	1,500,000	1,610,000	1,740,000
75-4	do.....	462	10.3	2318	2,790,000	2,770,000	2,700,000
101-3	do.....	28	13.2	1307			
101-4	do.....	541	13.2	2115	4,850,000	4,480,000	4,090,000
102-5	do.....	28	11.9	1510			
102-6	do.....	516	11.9	2692	3,020,000	3,060,000	3,070,000
104-1	do.....	32	11.1	978			
104-2	do.....	527	11.1	1542	3,900,000	3,510,000	2,350,000
20-1	1:3:5 New York trap.....	30		1254			
20-2	do.....	30		1158			
20-3	do.....	415		1109	4,950,000	940,000	450,000
20-4	do.....	415		958	8,700,000	330,000	
21-1	do.....	28		702			
21-2	do.....	28		678			
21-3	do.....	416		874	590,000	270,000	
21-4	do.....	416		986	900,000	520,000	250,000
54-1	do.....	29	12.6	790			
54-2	do.....	488	12.6	1012	1,330,000	1,850,000	530,000

TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS—Continued

Cylinder No.	Kind of Concrete	Age, Days	Per-cent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
15-1	1:2:4 Rockport granite.....	30		1478			
15-2	do.....	30		1181			
15-3	do.....	407		1773	2,800,000	2,730,000	2,600,000
15-4	do.....	407		2315	2,470,000	2,540,000	2,540,000
34-1	do.....	29	12.0	1638			
34-3	do.....	29	11.5	1308			
34-2	do.....	453	12.0	2211	2,800,000	2,570,000	2,390,000
34-4	do.....	453	11.5	1903	3,160,000	2,900,000	2,550,000
34A-1	do.....	28	12.7	853			
34A-3	do.....	28	10.7	1633			
34A-2	do.....	455	12.7	1157	2,060,000	1,750,000	1,350,000
34A-4	do.....	455	10.7	1833	2,680,000	2,670,000	2,560,000
103-1	do.....	28	13.0	993			
103-2	do.....	522	13.0	1645	3,620,000	3,100,000	2,960,000
113-1	do.....	32	12.2	1017			
113-2	do.....	527	12.2	1476		2,480,000	2,220,000
50-1	1:3:5 Rockport granite.....	28	13.5	919			
50-3	do.....	28	13.5	621			
50-2	do.....	487	13.5	1082	2,150,000	1,800,000	1,350,000
50-4	do.....	487	13.5	1063	2,380,000	1,850,000	1,280,000
50A-1	do.....	28	11.5	734			
50A-3	do.....	28	11.5	660			
50A-2	do.....	509	11.5	1255	2,400,000	1,950,000	1,650,000
50A-4	do.....	509	11.5	1073	2,750,000	2,430,000	2,220,000
51-1	do.....	29	14.0	729			
51-4	do.....	29	14.0	703			
51-2	do.....	485	14.0	924	1,480,000	1,130,000	750,000
51-3	do.....	485	14.0	688	1,480,000		
51A-1	do.....	29	14.1	683			
51A-4	do.....	29	14.1	822			
51A-2	do.....	507	14.1	877	2,170,000	1,260,000	640,000
51A-3	do.....	507	14.1	1272	2,270,000	2,230,000	2,000,000
31-1	1:2:4 Cleveland sandstone.....	30	14.0	2040			
31-3	do.....	30	14.2	1988			
31-2	do.....	501	14.0	2828	1,410,000	1,410,000	1,410,000
31-4	do.....	501	14.2	2531	1,230,000	1,230,000	1,200,000
43-1	do.....	29	14.0	746			
43-3	do.....	29	14.0	1855			
43-2	do.....	456	14.0	2128	1,420,000	1,290,000	1,230,000
43-4	do.....	456	14.0	2658	1,430,000	1,350,000	1,270,000
104-3	do.....	32	14.9	1535			
104-4	do.....	527	14.9	2720	2,270,000	2,020,000	1,670,000
44-1	1:3:5 Cleveland sandstone.....	29	14.7	1418			
44-3	do.....	29	14.7	661			
44-2	do.....	459	14.7	1662	4,130,000	3,040,000	2,050,000
44-4	do.....	459	14.7	1042	880,000	720,000	600,000
11-1	1:2:4 Joliet gravel.....	27	11.3	2365			
11-4	do.....	27	11.3	1764			
11-2	do.....	468	11.3	2190	3,030,000	3,100,000	3,120,000
11-3	do.....	468	11.3	2522	3,630,000	3,530,000	3,400,000
14-1	do.....	28		1070			
14-2	do.....	28		1192			
14-3	do.....	406		2177		3,670,000	3,470,000
14-4	do.....	406		1268	3,380,000	2,940,000	2,620,000
30-1	do.....	28		762			
30-3	do.....	28		1908			
30-2	do.....	440		898	2,250,000	1,550,000	620,000
30-4	do.....	440		2690	3,240,000	3,210,000	3,200,000
38-1	do.....	29	10.8	1080			
38-3	do.....	29	10.8	1290			
38-2	do.....	451	10.8	1872	3,350,000	3,000,000	2,840,000
38-4	do.....	451	10.8	1721	3,500,000	3,220,000	3,130,000
101-1	do.....	28	11.0	1401			
101-2	do.....	520	11.0	2408	3,810,000	3,750,000	3,600,000
102-3	do.....	28	11.0	1720			
102-4	do.....	514	11.0	2951	4,880,000	4,370,000	4,130,000

TABLE 21.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN COLUMNS AND COVERINGS—Concluded

Cylinder No.	Kind of Concrete	Age, Days	Per-cent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
112-1	1:2:4 Joliet gravel.....	28	9.7	2004			
112-2	do.....	521	9.7	2973	4,290,000	4,160,000	4,140,000
39-1	1:2:4 Meramec R. gravel.....	28		2053			
39-3	do.....	28		1220			
39-2	do.....	437		2580	3,450,000	3,500,000	3,430,000
39-4	do.....	437		1721	3,550,000	3,500,000	3,430,000
45-1	do.....	25	11.8	2056			
45-3	do.....	25	12.3	1987			
45-2	do.....	447	11.8	2707	4,600,000	4,370,000	4,040,000
45-4	do.....	447	12.3	2374	3,870,000	3,920,000	3,900,000
112-3	do.....	28	10.7	1719			
112-4	do.....	521	10.7	2952	3,310,000	3,510,000	3,600,000
17-1	1:1½:4½ Hard coal cinders.....	28		671			
17-2	do.....	28		1307			
17-3	do.....	408		1691	1,730,000	1,500,000	1,330,000
17-4	do.....	408		2072	1,900,000	1,770,000	1,670,000
32-1	1:2:5 Hard coal cinders.....	29	22.9	495			
32-3	do.....	29	22.9	728			
32-2	do.....	504	22.9	970	1,080,000	900,000	700,000
32-4	do.....	504	22.9	977	1,250,000	1,000,000	610,000
32A-1	do.....	32	17.3	628			
32A-3	do.....	32	18.5	759			
32A-2	do.....	498	17.3	950	900,000	720,000	500,000
32A-4	do.....	498	18.5	796	750,000	530,000	
47-1	do.....	28	22.1	949			
47-3	do.....	28	22.4	907			
47-2	do.....	447	22.1	1335	1,070,000	1,050,000	970,000
47-4	do.....	447	22.4	1258	1,150,000	1,030,000	890,000
52-1	do.....	28	22.3	901			
52-2	do.....	28	22.3	914			
52-3	do.....	492	22.3	929	1,140,000	920,000	580,000
52-4	do.....	492	22.3	879	1,300,000	920,000	450,000
53-1	do.....	30	29.1	306			
53-3	do.....	30	29.1	492			
53-2	do.....	494	29.1	635	1,000,000		
53-4	do.....	494	29.1	691	1,190,000	670,000	
104-5	do.....	32	22.9	797			
104-6	do.....	527	22.9	1313	1,340,000	1,280,000	1,190,000
12-1	1:1½:3 Westfield trap.....	28		2397			
12-2	do.....	28		1312			
12-3	do.....	534		3880	3,520,000	3,370,000	3,270,000
12-4	do.....	533		4180	4,890,000	4,450,000	4,130,000

TABLE 22.—COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY OF CONCRETE IN HEAD PROTECTIONS

Cylinder No.	Kind of Concrete	Age, Days	Percent Water	Ultimate Stress, Lb. per Sq. In.	Modulus of Elasticity, Lb. per Sq. In.		
					Unit Stress		
					450	650	850
49-1	1:2:4 Chicago limestone	28	11.2	1473			
49-3	do	28	11.2	1510			
49-2	do	489	11.2	2673	8,900,000	6,000,000	6,000,000
49-4	do	489	11.2	2178	3,870,000	3,320,000	3,140,000
59-1	do	29	10.5	990			
59-3	do	29	10.5	962			
59-2	do	494	10.5	1085	2,170,000	1,370,000	830,000
59-4	do	494	10.5	1319	3,780,000	2,870,000	2,250,000
*64-1	do	27	9.8	3428			
*64-4	do	27	9.8	3119			
*64-2	do	502	9.8	4036	8,810,000	7,940,000	7,580,000
*64-3	do	502	9.8	4000	4,230,000	4,060,000	4,060,000
65-1	do	27	9.7	2120			
65-4	do	27	9.7	1951			
65-2	do	502	9.7	3157	6,500,000	5,990,000	5,830,000
65-3	do	502	9.7	2813	4,300,000	4,050,000	3,770,000
69-1	do	29	10.9	1874			
69-3	do	29	10.9	1698			
69-2	do	497	10.9	2469	4,150,000	3,880,000	3,750,000
69-4	do	497	10.9	2941	7,080,000	5,620,000	5,000,000
A-1	do	28	9.6	2058			
A-3	do	28	9.6	1947			
A-2	do	520	9.6	2969	4,040,000	4,020,000	3,810,000
B-1	do	28	11.2	1986			
B-3	do	28	11.2	2197			
B-2	do	520	11.2	3131	4,060,000	3,940,000	3,770,000
B-4	do	524	11.2	2282	3,400,000	3,420,000	3,300,000
B-5	do	524	11.2	2527	4,100,000	4,070,000	4,090,000
48-1	1:3:5 Chicago limestone	28	11.5	961			
48-3	do	28	11.5	1736			
48-2	do	489	11.5	1358	2,810,000	2,710,000	2,400,000
48-4	do	489	11.5	2397	3,750,000	3,710,000	3,570,000
62-1	do	28	11.2	881			
62-3	do	28	11.2	1324			
62-2	do	492	11.2	1731	4,580,000	4,260,000	3,730,000
62-4	do	492	11.2	1613	2,900,000	2,770,000	2,370,000
63-1	1:2:4 New York trap	27	11.1	1408			
63-2	do	27	11.1	1280			
63-3	do	496	11.1	2175	2,750,000	2,480,000	2,380,000
63-4	do	496	11.1	1872	3,090,000	2,770,000	2,430,000
*67A-1	do	28	10.3	1945			
*67A-2	do	28	10.3	1998			
*67A-3	do	493	10.3	2573		7,950,000	6,480,000
*67A-4	do	493	10.3	2406	4,890,000	4,080,000	1,780,000
49-5	1:2:4 Rockport granite	28	10.6	1395			
49-7	do	28	10.6	1509			
49-6	do	489	10.6	2212	6,650,000	4,230,000	3,640,000
49-8	do	489	10.6	2233	3,790,000	3,490,000	2,900,000
68-1	do	30	9.8	1307			
68-4	do	30	9.8	1495			
68-2	do	494	9.8	1606	3,580,000	2,950,000	2,570,000
68-3	do	494	9.8	1930	4,080,000	4,000,000	3,500,000
58-1	1:3:5 Rockport granite	28	13.4	841			
58-4	do	28	13.4	946			
58-2	do	498	13.4	1172	2,510,000	2,070,000	1,670,000
58-3	do	498	13.4	1263	2,430,000	2,030,000	1,770,000
105-5	1:2:4 Joliet gravel	28	10.5	1128			
105-8	do	28	10.5	1355			
105-6	do	538	10.5	1921	3,480,000	3,330,000	3,100,000
105-7	do	538	10.5	1723	2,800,000	2,560,000	2,130,000
106-5	do	29	10.7	1396			
106-8	do	28	10.7	1403			
106-6	do	532	10.7	1653	3,460,000	3,250,000	3,100,000
106-7	do	532	10.7	2076	3,920,000	3,350,000	3,430,000
105-1	1:2:4 Meramec R. gravel	28	9.8	1721			
105-4	do	28	9.8	1691			
105-2	do	538	9.8	2699	5,030,000	4,600,000	4,230,000
105-3	do	538	9.8	2706	3,040,000	3,090,000	3,060,000
109-5	do	28	8.7	2152			
109-7	do	28	8.7	2021			
109-6	do	509	8.7	3232	8,100,000	5,710,000	5,000,000
109-8	do	509	8.7	3095	5,060,000	4,350,000	4,190,000

*Time of mixing, 2 min. For all other cylinders time of mixing was 1 min.

Cylinder No. 64-2 failed on third application of maximum load, No. 64-3 on second application of maximum load.

TABLE 23.—TESTS OF LIME

Sample No.	Quicklime		Hydrated Lime	
	1	2	1	2
Chemical Analysis:				
Impurities (SiO_2 and R_2O_3).....	1.73%	1.70%	1.44%	1.31%
CaO.....	88.98	85.53	69.59	69.59
MgO.....	1.26	0.89	1.37	1.37
Loss on Ignition.....	8.22	8.22	27.02	27.07
CO ₂ Average.....	0.66		4.89	
Fineness:				
Residue on No. 20 Sieve, Sample Slacked and Washed Through.....	1.25			
Residue on No. 30 Sieve, Original Sample, Dry.....			4.4	
Residue on No. 200 Sieve, Sample Washed Through..			3.97	
Soundness: Pats—				
5 Hr. in Steam.....			Unsound	

TABLE 24.—TESTS OF CALCINED GYPSUM

	Western	Eastern
Chemical Analysis:		
Impurities (SiO_2 and R_2O_3).....	3.55%	2.10%
CaO.....	36.77	36.93
SO ₃	52.22	51.37
Loss Below 60°C.....	1.23	1.12
Loss Between 60°C. and Tyrrell Burner.....	6.67	8.35
Time of Set:		
Neat, 55 percent Water.....	More than 7 hr.; less than 16 hr.	24 hr.
*Tensile Strength: Lb. per Sq. In.		
Neat, 55 percent Water.....	200.0 190.0 192.0	Easily crumbles No test obtainable
1 Gypsum: 3 Ottawa Sand, 22 percent Water.....	97.0 102.0 85.0	175.0 150.0 150.0

*Mortars made up with distilled water and after setting 24 hr. in air, were dried to constant weight below 60°C. and then broken.

TABLE 25.—COMPRESSIVE STRENGTH OF PORTLAND CEMENT
AND LIME PLASTER

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	Percent Water	Age, Days	Compressive Strength, Lb. per Sq.in.
23-1	1 Portland cement.....	15. 78	28	1080
	1/10 Hydrated lime.....			
	2½ Coarse lake sand.....			
23-2	do.....	15. 78	28	1055
23-3	do.....	15. 78	512	1623
23-4	do.....	18. 23	29	1488
23-6	do.....	18. 10	29	1795
23-7	do.....	18. 10	511	2385
23-8	do.....	15. 23	28	2715
23-9	do.....	15. 23	509	3828
24-1	do.....	16. 23	28	1839
24-2	do.....	16. 23	501	2855
24-3	do.....	17. 79	29	1350
24-4	do.....	17. 79	498	2140
24-5	do.....	12. 87	29	933
24-6	do.....	12. 87	496	1669
25-1	do.....	16. 20	28	2520
25-2	do.....	16. 20	487	3956
25-3	do.....	18. 50	29	1319
25-4	do.....	18. 50	484	2480
26-1	do.....	18. 35	29	1655
26-2	do.....	18. 35	500	3035
26-3	do.....	15. 82	28	2685
26-4	do.....	15. 82	498	3744
27-1	do.....	18. 00	29	1645
27-2	do.....	18. 00	501	3191
27-3	do.....	16. 50	29	976
27-4	do.....	16. 50	499	1934
78-1	do.....	19. 52	38	818
78-2	do.....	19. 52	38	795
78-3	do.....	19. 52	38	1083
78-4	do.....	19. 52	38	640
78-5	do.....	19. 52	38	610
78-6	do.....	19. 52	38	790
78-7	do.....	21. 15	34	991
78-8	do.....	21. 15	34	583
78-9	do.....	21. 15	34	1035
110-1	do.....	21. 60	28	1455
110-2	do.....	21. 60	28	1200
110-3	do.....	21. 60	28	1225
110-4	do.....	21. 60	514	1805
110-5	do.....	21. 60	514	1595
110-6	do.....	21. 60	514	1946
110-7	do.....	17. 20	28	1913
110-8	do.....	17. 20	28	1748
110-9	do.....	17. 20	28	1716
110-10	do.....	17. 20	512	2181
110-11	do.....	17. 20	512	3408
110-12	do.....	17. 20	512	2639
110-13	do.....	19. 00	28	1789
110-14	do.....	19. 00	28	1863
110-15	do.....	19. 00	28	1831
110-16	do.....	19. 00	507	1960
110-17	do.....	19. 00	507	2990
110-18	do.....	19. 00	507	2753
110-19	do.....	19. 02	29	2084
110-20	do.....	19. 02	29	2020
110-21	do.....	19. 02	29	1710
110-22	do.....	19. 02	500	2838
110-23	do.....	19. 02	500	2758
110-24	do.....	19. 02	500	3248
77-4	1 Slaked lime.....	18. 36	60	158
77-5	2½ Fine lake sand.....	18. 36	60	149

TABLE 26.—COMPRESSIVE STRENGTH OF CLAY TILE MORTAR

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
48-1	1 Portland cement..... 1 Slaked lime..... 4 Bank or beach sand.....	24.00	28	955
48-2	do.....	24.00	28	873
48-3	do.....	24.00	499	1040
48-4	do.....	24.00	499	769
48-5	do.....	27.50	28	763
48-6	do.....	27.50	512	833
49-1	do.....	25.00	28	421
49-2	do.....	25.00	498	485
49-3	do.....	27.15	28	410
49-4	do.....	27.15	498	405
50-1	do.....	26.00	28	273
50-2	do.....	26.00	496	361
51-1	do.....	25.70	28	385
51-2	do.....	25.70	497	358
51-3	do.....	28.45	28	278
51-4	do.....	28.45	497	308
51A-1	do.....	24.45	29	303
51A-2	do.....	24.45	516	313
53-1	do.....	24.40	28	445
53-2	do.....	24.40	503	370
56-1	do.....	24.60	28	455
56-2	do.....	24.60	28	399
56-3	do.....	24.60	500	565
56-4	do.....	24.60	500	550
57-1	do.....	22.70	29	513
57-2	do.....	22.70	497	700
58-1	do.....	24.80	28	288
58-2	do.....	24.80	504	373
58-3	do.....	24.10	28	340
58-4	do.....	24.10	504	348
58-5	do.....	28.00	28	283
58-6	do.....	28.00	504	325
59-1	do.....	25.90	28	395
59-2	do.....	25.90	498	333
59-3	do.....	24.75	28	368
59-4	do.....	24.75	497	270
60-1	do.....	25.00	28	296
60-2	do.....	25.00	505	285
60-3	do.....	26.60	28	360
60-4	do.....	26.60	505	358
61-1	do.....	28.05	29	513
61-2	do.....	28.05	29	390
61-3	do.....	28.05	524	473
61-4	do.....	28.05	524	628
61-5	do.....	23.50	29	415
61-6	do.....	23.50	524	480
62-1	do.....	25.30	28	475
62-2	do.....	25.30	433	551
63-1	do.....	28.25	28	313
63-2	do.....	28.25	505	350
68-1	do.....	29.50	28	310
68-2	do.....	29.50	503	270
68-3	do.....	24.10	28	285
68-4	do.....	24.10	503	323
69-1	do.....	21.80	516	463
69-3	do.....	24.45	29	318
69-4	do.....	24.45	516	338
69-5	do.....	21.45	28	853
69-6	do.....	21.45	515	940
69-7	do.....	22.65	28	653
69-8	do.....	22.65	28	663
69-9	do.....	22.65	493	773

**TABLE 26.—COMPRESSIVE STRENGTH OF CLAY TILE MORTAR—
Concluded****Test specimens 2-in. cubes stored in air**

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
69-10	1 Portland cement.....			
	1 Slaked lime.....	22.65	493	708
	4 Bank or beach sand.....			
76-1	do.....	23.75	60	495
76-2	do.....	23.75	60	571
76-3	do.....	23.75	60	588
77-1	do.....	23.75	60	380
77-2	do.....	23.75	60	385
77-3	do.....	23.75	60	360
52-2	1 Portland cement.....			
	2 Slaked lime.....	29.40	38	228
	6 Bank sand.....			
52-5	do.....	30.30	29	100
54-5	do.....	26.60	28	159
54-6	do.....	26.60	498	216
54-1	1 Portland cement.....			
	1½ Hydrated lime.....	20.90	28	303
	6 Bank sand.....			
54-3	do.....	20.90	28	395
54-4	do.....	20.90	499	626

TABLE 29.—COMPRESSIVE STRENGTH OF GYPSUM FILLING**Test specimens 8 by 16 in. cylinders**

Cylinder No.	MATERIAL	Proportions, Parts by Loose Volume of Materials as Used	*Per-cent Water	Age, Days	Ultimate Strength, Lb. per Sq. In.
66-1	Eastern gypsum filling.....	1 Eastern calcined gypsum.....			
		1 Finelake sand.....	54.70	31	15
		4 Broken gypsum blocks.....			
66-5	do.....	do.....	61.20	29	31
66-6	do.....	do.....	61.20	483	95
67A-5	do.....	do.....	60.15	29	22
67A-6	do.....	do.....	60.15	29	17
67A-7	do.....	do.....	60.15	489	105
67A-8	do.....	do.....	60.15	489	85
109-3	Specimen collapsed on removal	of mould 12 days after placement..	52.20		
109-4	Broken during storage.....		52.20		
108-1	Western gypsum filling.....	1 Western calcined gypsum.....			
		1 Finelake sand.....	63.30	28	27
		4 Broken gypsum blocks.....			
108-2	do.....	do.....	63.30	28	33
108-3	do.....	do.....	63.30	513	42
108-4	do.....	do.....	63.30	513	32
109-1	do.....	do.....	63.10	28	29
109-2	do.....	do.....	63.10	509	40

* Based on total weight of dry materials in mixture.

TABLE 27.—COMPRESSIVE STRENGTH OF GYPSUM MORTAR AND PLASTER

Test specimens 2-in. cubes stored in air

Cube No.	Proportion, Parts by Loose Volume of Materials as Used	*Percent Water	Age, Days	Compressive Strength, Lb. per Sq. In.
64-1	1 Western calcined gypsum			
	3 Fine lake sand	23.40	30	220
64-3	do	23.40	30	145
64-4	do	23.40	523	118
64-5	do	23.40	523	94
65-1	do	22.20	29	233
65-2	do	22.20	29	275
65-3	do	22.20	29	275
65-4	do	22.20	522	202
65-5	do	22.20	522	214
66-1	1 Eastern calcined gypsum			
	3 Fine lake sand	23.50	29	43
66-2	do	23.50	29	13
66-3	do	23.50	29	15
66-5	do	23.50	504	35
67-1	do	23.10	28	193
67-2	do	23.10	28	105
67-3	do	23.10	28	213
67-5	do	23.10	520	152
67-6	do	23.10	503	160
67A-1	do	22.85	31	215
66A-2	do	22.85	31	130
67A-3	do	22.85	31	188
67A-6	do	22.85	497	83
108-1	do	22.60	27	189
108-2	do	22.60	27	201
108-3	do	22.60	489	83
108-4	do	20.60	27	240
108-5	do	20.60	489	139
108-6	do	20.60	489	172
109-1	do	22.20	28	70
109-2	do	22.20	28	86
109-3	do	22.20	515	75
109-4	do	22.60	515	139
109-5	do	22.60	28	200
109-6	do	22.60	515	222
76-4	1 Fibered gypsum plaster			
	3 Fine lake sand	17.20	60	443
76-5	do	17.20	60	468
76-6	1 Calcined gypsum			
	2 Hydrated lime	61.00	60	958
77-6	do	61.00	60	1029

*Based on total weight of dry materials in mixture.

TABLE 28.—STRENGTH TESTS OF MORTAR AND PLASTER

Test specimens, 1-in. briquettes for tension tests and 2-in. cubes for compression tests made in laboratory. Each value is the average of three tests.

Mortar or Plaster	Proportion, Parts by Loose Volume of Materials as Used	*Per-cent Water	Method of Storage	Age, Days	Strength, Lb. per Sq. In.	
					Ten-sile	Com-pressive
Clay tile mortar.....	1 Portland cement.....		1 day in damp closet, remainder of period in water			
	1 Slaked lime.....	25.0	do	7	67	257
	4 Bank sand.....		do			
do	do with beach sand.....	25.0	do	7	72	236
do	do with Ottawa sand.....	25.0	do	7	65	174
do	do with bank sand.....	25.0	do	28	97	425
do	do with beach sand.....	25.0	do	28	114	496
do	do with Ottawa sand.....	25.0	do	28	91	369
do	do with Ottawa sand.....		do	28	161	848
do	do with bank sand.....	25.0	1 day in damp closet, remainder of period in air	28		328
do	do with Ottawa sand.....	25.0	do	28		190
do	do with bank sand.....	25.0	do	365		442
do	do with Ottawa sand.....	25.0	do	365		259
Portland cement plaster.....	1 Portland cement.....		1 day in damp closet, remainder of period in water			
	1/10 Hydrated lime.....	16.7	do	7	144	418
	2½ Coarse lake sand.....		do			
do	do with Ottawa sand.....	16.7	do	7	114	363
do	do with coarse lake sand.....	16.7	do	28	197	1105
do	do with Ottawa sand.....	16.7	do	28	154	721
do	do with coarse lake sand.....	16.7	1 day in damp closet, remainder of period in air	28		1010
do	do with Ottawa sand.....	16.7	do	28		616
do	do with coarse lake sand.....	16.7	do	365		1297
do	do with Ottawa sand.....	16.7	do	365		963
Western gypsum block mortar.....	1 Western gypsum.....	22.0	1 day in air, then dried to constant weight below 60° C.	2	126	
	3 Fine lake sand.....		do			
do	do with Ottawa sand.....	22.0	do	2	95	
Eastern gypsum block mortar.....	1 Eastern gypsum.....	22.5	do	2	202	
	3 Fine lake sand.....		do			
do	do with Ottawa sand.....	22.5	do	2	158	
Western gypsum block mortar.....	1 Western gypsum.....	22.0	In air for whole period.	28		491
	3 Fine lake sand.....		do			
do	do with Ottawa sand.....	22.0	do	28		343
do	do with fine lake sand.....	22.0	do	365		647
do	do with Ottawa sand.....	22.0	do	365		517
Eastern gypsum block mortar.....	1 Eastern gypsum.....	22.5	do	28		786
	3 Fine lake sand.....		do			
do	do with Ottawa sand.....	22.5	do	28		415
do	do with fine lake sand.....	22.5	do	365		1180
do	do with Ottawa sand.....	22.5	do	365		698

*Based on total weight of dry materials in mixture.

TABLE 30.—CLASSIFICATION AND DESCRIPTION OF HOLLOW CLAY TILE

Tile No.	Nominal Thickness, In.	Type of Clay	District	Hardness	Color	Average Outside Dimensions, In.			Thickness of Side Shells, In.	Cross Webs		Scoring		
						Thickness	Width	Length		Number	Av. Thickness, In.	Number	Shape	Dimensions, In.
A	2	Surface clay....	Chicago, Ill.	Soft.....	Light red.	2.06	12.00	12.00	0.80	4	0.80	22	Rectangular	0.12 by 0.50
A	4	do.....	do.....	Soft.....	Light red.	4.09	12.20	12.00	0.79	4	0.71	22	do.....	0.12 by 0.50
B	2	do.....	Boston, Mass.....	Medium	Red.....	2.10	6.10	12.00	0.65	3	0.68	12	Half round..	0.30 diam.
B	4	do.....	do.....	Medium	Red.....	4.03	12.30	11.70	0.83	5	0.75	24	do.....	0.40 diam.
C	2	Semi-fire clay..	Waynesburg, O...	Hard.....	Buff.....	2.07	12.02	11.90	0.55	4	0.50	24	Rectangular	0.12 by 0.45
C	4	do.....	do.....	Hard.....	Buff.....	3.98	11.84	11.85	0.53	4	0.46	28	do.....	0.12 by 0.45
D	2	Shale.....	Cleveland, O.....	Hard.....	Red.....	2.01	11.90	11.65	0.52	4	0.50	26	do.....	0.02 by 0.25
D	4	do.....	do.....	Hard.....	Red.....	3.96	11.62	11.85	0.60	4	0.56	40	do.....	0.10 by 0.30
E	2	Semi-fire clay..	Perth Amboy, N. J	Medium	Buff.....	2.00	7.96	12.20	0.70	4	0.82	22	Half round..	0.20 diam.
E	4	do.....	do.....	Medium	Buff.....	3.85	11.87	11.87	0.67	4	0.64	38	do.....	0.30 diam.
F	2-in. curved	do.....	do.....	Soft.....	Light red.	2.09	10.56	12.00	0.85	4	0.96	11	do.....	0.30 diam.
G	2-in. curved	do.....	do.....	Soft.....	Light red.	2.02	14.00	12.00	0.83	4	1.45	16	do.....	0.35 diam.
H	2-in. curved	do.....	do.....	Soft.....	Light red.	2.55	10.94	12.00	0.76	4	0.67	11 3	Rectangular	0.10 by 0.37 0.40 by 2.74

Fire Tests of Building Columns

TABLE 31.—COMPRESSIVE STRENGTH OF HOLLOW CLAY TILE

Tile units are 12 in. wide and 12 in. long, except as noted

Sample No.	Kind of Title		Porosity, Percent of Volume	Absorption, Percent of Dry Weight	How Tested	Area under Load, Sq. In.	Maximum Load	
	Nominal Thickness, In.	Clay					Total, Lb.	Lb. per Sq. In.
A-1	2	Surface clay, Chicago district....	40.3	29.0	On end.....	19.6	44300	2260
A-2	2		44.4	28.5	do.....	19.6	46860	2390
A-3	2		43.1	27.3	do.....	19.6	44650	2280
Average....			42.6	28.3			45277	2310
A-4	2	do.....	44.5	28.5	On edge.....	16.2	18920	1170
A-5	2		43.7	27.7	do.....	16.2	32110	1980
A-6	2		41.9	26.2	do.....	16.2	22340	1380
Average....			43.4	27.5			24456	1510
A-16	4	do.....	34.4	27.6	On end.....	25.2	53000	2100
A-19	4				do.....	25.2	57280	2270
A-20	4				do.....	25.2	63460	2520
Average....							57913	2296
A-11	4	do.....	44.2	28.1	On edge.....	16.0	29320	1830
A-12	4		42.9	29.5	do.....	16.0	20250	1270
A-14	4		41.8	26.0	do.....	16.0	33190	2070
Average....			42.9	27.5			27588	1723
*B-6	2	Surface clay, Boston district	28.8	15.0	On end.....	9.1	52830	5800
*B-8	2		27.3	15.0	do.....	9.1	48400	5310
*B-10	2		27.9	14.2	do.....	9.1	62450	6800
Average....			28.0	14.7			54560	5990
*B-1	2	do.....	30.0	16.0	On edge.....	11.7	52450	4480
*B-2	2		28.6	15.0	do.....	11.7	58150	4970
*B-4	2		31.0	16.6	do.....	11.7	58040	4900
Average....			29.8	15.8			56213	4796
B-12	4	do.....	30.8	16.4	On end.....	28.4	134350	4740
B-14	4		32.4	17.6	do.....	28.4	111400	3920
B-19	4		25.6	12.8	do.....	27.2	160930	5910
Average....			29.6	15.6			135560	4856
B-11	4	do.....	28.8	15.0	On edge.....	15.3	68170	4460
B-13	4		28.0	14.6	do.....	15.3	69760	4560
B-17	4		29.8	15.6	do.....	14.1	56710	4020
Average....			28.8	15.1			64880	4346
C-1	2	Ohio semi-fire clay	15.5	7.3	On end.....	14.1	98350	6980
C-5	2		15.4	7.0	do.....	14.1	123500	8770
C-6	2		22.5	11.2	do.....	14.7	94280	6410
Average....			17.8	8.5			105376	7386
C-2	2	do.....	18.3	8.8	On edge.....	11.2	68870	6150
C-4	2		18.5	8.8	do.....	11.2	47040	4200
C-8	2		16.9	7.9	do.....	11.2	46920	4190
Average....			17.9	8.8			54276	4846
C-13	4	do.....	18.5	9.0	On end.....	16.7	123610	7400
C-14	4		17.6	8.6	do.....	17.2	104850	6090
C-19	4		18.5	8.9	do.....	16.7	96050	5760
Average....			18.2	8.8			108170	6416
C-15	4	do.....	16.0	8.4	On edge.....	9.6	58180	6060
C-18	4				do.....	10.0	36780	3680
Average....							47480	4870

*Nominal width of tile, 6 in.

**TABLE 31.—COMPRESSIVE STRENGTH OF HOLLOW CLAY TILE—
Concluded**

Tile units are 12 in. wide and 12 in. long, except as noted

Sample No.	Kind of Tile		Porosity, Percent of Volume	Absorption, Percent of Dry Weight	How Tested	Area under Load, Sq. In.	Maximum Load	
	Nominal Thickness, In.	Clay					Total, Lb.	Lb. per Sq. In.
D-1	2	Ohio shale..	8.1	3.5	On end.....	14.3	114350	8000
D-3	2	do.....	22.1	10.8	do.....	14.1	112600	7990
D-9	2	do.....	14.3	7.2	do.....	14.5	109680	7560
Average....			14.8	7.1			112210	7850
D-2	2	do.....	16.2	7.4	On edge.....	11.1	43300	3900
D-5	2	do.....	12.2	5.5	do.....	10.7	61980	5800
D-8	2	do.....	8.2	3.6	do.....	11.1	81450	7340
Average....			12.2	5.5			62243	5680
D-11	4	do.....	12.0	5.2	On end.....	18.9	193550	9720
D-16	4	do.....	11.3	4.9	do.....	18.1	151770	7950
D-17	4	do.....	12.2	5.5	do.....	18.9	177940	8930
Average....			11.8	5.2			174420	8866
D-13	4	do.....	16.3	7.4	On edge.....	11.7	95840	8190
D-14	4	do.....	13.8	6.5	do.....	11.7	61130	5220
D-19	4	do.....	9.6	3.8	do.....	11.5	117790	10230
Average....			13.2	5.9			91686	7880
*E-5	2	Semi-fire	28.3	17.2	On end.....	12.1	53240	4400
*E-6	2	clay,	30.4	17.1	do.....	12.1	51040	4210
*E-8	2	New Jersey district			do.....	12.1	55620	4600
Average....							53313	4402
*E-1	2	do.....	33.2	19.8	On edge.....	15.6	33670	2160
*E-2	2	do.....	33.5	19.6	do.....	15.9	29000	1820
*E-10	2	do.....	35.8	22.0	do.....	15.9	23100	1450
Average....			34.1	20.4			28590	1810
E-15	4	do.....	25.3	14.2	On end.....	21.6	95650	4440
E-17	4	do.....	33.6	20.6	do.....	21.6	74660	3460
E-19	4	do.....	29.5	17.2	do.....	21.6	86250	4000
Average....			29.4	17.3			85520	3966
E-12	4	do.....	29.0	16.2	On edge.....	12.6	39000	3170
E-13	4	do.....	33.2	19.8	do.....	12.6	24920	1980
E-14	4	do.....	33.9	20.4	do.....	12.6	20050	1590
Average....			32.0	15.4			28290	2246
†F-5	2	Porous semi-	37.6	23.2	On end.....	18.9	83270	4410
†F-8	2	fire clay,	41.8	27.0	do.....	19.9	54200	2720
†F-10	2	New Jersey district	45.3	30.4	do.....	18.3	72630	3970
Average....			41.5	23.8			70033	3700
†G-2	2	do.....	49.1	27.6	do.....	24.6	38060	1550
†G-3	2	do.....	50.2	39.1	do.....	23.9	33370	1400
†G-4	2	do.....	40.2	26.4	do.....	23.9	53440	2230
Average....			46.5	31.0			41623	1726
†H-1	2½	do.....	51.2	39.0	do.....	19.9	27150	1360
†H-2	2½	do.....	46.9	34.3	do.....	18.0	32110	1780
†H-4	2½	do.....	41.4	27.7	do.....	20.8	40300	1940
Average....			46.5	33.3			33086	1693

* Nominal width of tile, 8 in.

† Curved tile.

TABLE 32.—TRANSVERSE STRENGTH OF HOLLOW TILE

Tile tested flatwise on side with center load and supports 10 in. apart.
 Tile units are 12 in. wide, except as noted.

Sample No.	Kind of Tile		Maximum Load, Lb.	Calculated Maximum Stresses, Lb. per Sq. In.	
	Nominal Thickness, In.	Clay		Bending, Outer Fiber	Shear, Center of Webs
A-7	2	Surface clay, Chicago district.....	1120	365	158
A-8	2	do.....	1050	370	148
A-9	2	do.....	830	292	117
A-10	2	do.....	1100	387	155
Average.....			1025	361	145
A-13	4	do.....	2600	254	146
A-15	4	do.....	1500	147	84
A-17	4	do.....	2500	244	140
A-18	4	do.....	1890	185	106
Average.....			2122	207	119
*B-3	2	Surface clay, Boston district.....	1000	625	202
*B-5	2	do.....	1520	950	307
*B-7	2	do.....	1280	800	258
*B-9	2	do.....	1240	775	250
Average.....			1260	787	254
B-15	4	do.....	5600	547	253
B-16	4	do.....	6900	675	312
B-18	4	do.....	5100	499	231
B-20	4	do.....	5250	513	237
Average.....			5712	558	283
C-7	2	Ohio semi-fire clay.....	1220	477	236
C-10	2	do.....	2040	798	394
Average.....			1630	637	315
C-20	4	do.....	2120	321	223
D-4	2	Ohio shale.....	2530	945	415
D-6	2	do.....	1630	608	267
D-7	2	do.....	3200	1194	525
Average.....			2452	916	402
D-12	4	do.....	4530	564	320
D-15	4	do.....	6250	778	441
D-18	4	do.....	4910	611	346
D-20	4	do.....	3280	408	231
Average.....			4742	590	335
†E-3	2	New Jersey semi-fire clay.....	1410	705	155
†E-4	2	do.....	1100	550	121
Average.....			1255	627	138
E-11	4	do.....	2010	272	142
E-16	4	do.....	1900	257	134
E-18	4	do.....	1330	180	94
E-20	4	do.....	2570	347	182
Average.....			1952	264	140

*Nominal width, 6 in.

†Nominal width, 8 in.

TABLE 33.—TEMPERATURES OF VITRIFICATION AND FUSION OF CLAY TILE AND BRICK

No.	CLAY	Temperature of Vitrification, Deg. C.	Temperature Producing Softening, Deg. C.	Temperature of Fusion, Deg. C.
A	Surface clay, Chicago district.....	1120	1200	1240
B	Surface clay, Boston district.....	1120	1160	1180
C	Ohio semi-fire clay.....	1180	1450	1460
D	Ohio shale.....	1120	1380	1400
E	New Jersey semi-fire clay partition tile.....	1200	1480	1500
F	New Jersey porous semi-fire clay, 2 by 8-in. curved tile.....		1450	1470
G	New Jersey porous semi-fire clay, 2 by 12-in. curved tile.....		1510	1530
H	New Jersey porous semi-fire clay, 2½ by 11-in. curved tile.....		1450	1470
J	Chicago common brick.....			1145

TABLE 34.—POROSITY AND ABSORPTION OF CHICAGO COMMON BRICK

Brick No.	Specimen No.	Porosity, Percent of Volume	Absorption, Percent of Dry Weight
1	1	35.4	20.2
1	2	31.9	17.8
1	3	32.5	18.0
1	4	31.5	17.6
1	5	24.5	13.1
1	6	36.3	20.9
1	7	26.5	15.4
1	8	24.3	12.9
1	Average.....	30.4	16.9
2	1	43.1	23.8
2	2	42.1	26.9
2	3	40.1	24.4
2	4	39.9	24.4
2	5	40.3	24.7
2	6	40.9	25.1
2	7	40.3	24.6
2	8	40.6	24.9
2	Average.....	40.9	25.1

TABLE 35.—COMPRESSIVE STRENGTH OF CHICAGO COMMON BRICK

Sample No.	How Tested	Dimensions, In.			Area Under Load, Sq. In.	Maximum Load	
		Width	Thick-ness	Length		Total, Lb.	Lb. per Sq. In.
J-1.....	On end.....	3.45	2.20		7.59	32900	4330
J-2.....	do.....	3.75	2.25		8.44	14900	1760
J-4.....	do.....	3.50	2.25		7.88	27550	3500
Average.....							3200
J-3 $\frac{1}{2}$ brick.....	On edge.....		2.25	2.9	6.52	7550	1160
J-5 do.....	do.....		2.15	3.35	7.20	26000	3610
J-7 do.....	do.....		2.35	5.40	12.69	15680	1240
J-8 do.....	do.....		2.35	4.10	9.64	16800	1740
J-9 do.....	do.....		2.25	3.40	7.67	15150	1980
J-10 do.....	do.....		2.25	4.55	10.24	20700	2020
Average.....							1960
J-3 $\frac{1}{2}$ brick.....	On side.....	3.70		4.10	15.17	46010	3030
J-5 do.....	do.....	3.50		4.60	16.10	55800	3460
J-6 do.....	do.....	3.60		7.80	28.08	57730	2050
J-7 do.....	do.....	3.50		2.60	9.10	16490	1810
J-9 do.....	do.....	3.60		4.60	16.56	67910	4100
J-10 do.....	do.....	3.65		3.70	13.50	32900	2440
Average.....							2815

TABLE 36.—TRANSVERSE STRENGTH OF CHICAGO COMMON BRICK

Brick tested on side with center load and supports 7 in. apart

Sample No.	Dimensions, In.		Maximum Load, Lb.	Modulus of Rupture Lb. per Sq. In.
	Width	Thickness		
J-3.....	3.70	2.25	940	530
J-5.....	3.50	2.15	2700	1750
J-8.....	3.70	2.35	460	240
J-9.....	3.60	2.25	1540	890
J-10.....	3.65	2.25	1580	900
Average.....				862

TABLE 37.—POROSITY OF GYPSUM BLOCK

Sample No.	Kind of Block		Porosity, Percent of Total Volume
	Thickness	Gypsum	
K-4.....	4-in. hollow.....	Western.....	63.7
K-5.....	4-in. hollow.....	do.....	63.7
L-4.....	4-in. solid.....	do.....	63.7
L-5.....	4-in. solid.....	do.....	63.9
M-4.....	2-in. solid.....	do.....	63.6
M-5.....	2-in. solid.....	do.....	63.6
Average.....			63.7
N-4.....	4-in. solid.....	Eastern.....	58.6
N-5.....	4-in. solid.....	do.....	58.4
O-4.....	2-in. solid.....	do.....	62.6
O-5.....	2-in. solid.....	do.....	62.2
Average.....			60.4

TABLE 38.—COMPRESSIVE STRENGTH OF SOLID
GYPSUM BLOCK

All blocks tested on edge

Sample No.	Gypsum	Dimensions, In.			Area Under Load, Sq. In.	Maximum Load	
		Thick- ness	Length	Width		Total, Lb.	Lb. per Sq. In.
L-1.....	Western.....	4	30.1	12	120.5	61100	507
L-2.....	do.....	4	30.1	12	120.5	44300	367
L-3.....	do.....	4	30.0	12	120.0	49030	409
Average.....							428
M-1.....	do.....	2	29.9	12	59.8	38250	640
M-2.....	do.....	2	29.9	12	59.8	23760	396
M-3.....	do.....	2	30.0	12	60.0	23840	398
Average.....							478
N-1.....	Eastern.....	4	25.7	16.5	103.0	67540	656
N-2.....	do.....	4	25.7	16.5	103.0	51340	498
N-3.....	do.....	4	26.2	16.5	105.0	52860	504
Average.....							553
O-1.....	do.....	2	28.1	16.5	52.2	23950	458
O-2.....	do.....	2	28.1	16.5	52.2	19205	365
O-3.....	do.....	2	28.1	16.5	52.2	21635	415
Average.....							413

TABLE 39.—TRANSVERSE STRENGTH OF SOLID GYPSUM BLOCK

All blocks tested flatwise on side with center load

Sample No.	Gypsum	Dimensions, In.			Span, In.	Weight of Block, Lb.	Maximum Load, Lb.	Modulus of Rupture, Lb. per Sq. In.
		Thickness	Length	Width				
L-7.....	Western.....	4	30.2	12	24	42.9	689	132
L-8.....	do.....	4	30.2	12	24	47.4	652	125
L-9.....	do.....	4	30.2	12	24	46.2	562	109
Average..								122
M-7.....	do.....	2	29.8	12	24	22.7	266	206
M-8.....	do.....	2	29.9	12	24	22.1	260	201
M-9.....	do.....	2	29.9	12	24	22.8	294	227
Average..								211
N-7.....	Eastern.....	4	26.1	16.5	20	58.9	1520	175
N-8.....	do.....	4	26.1	16.6	20	56.2	1438	165
N-9.....	do.....	4	26.0	16.6	20	58.2	1503	169
Average..								170
O-7.....	do.....	2	25.9	16.4	20	24.4	305	143
O-8.....	do.....	2	26.2	16.3	20	27.7	245	117
O-9.....	do.....	2	25.9	16.4	20	25.7	311	146
Average..								135

TABLE 40.—TRANSVERSE STRENGTH OF GYPSUM WALL BOARD

Samples, 18 in. long, were tested with center load and supports 16 in. apart

No.	Width, In.	Thick-ness, In.	Weight, Lb.	Condition of Board	Direction of Loading Bar	Maximum Load, Lb.
A-1.....	11.9	0.38	3.1	Dry.....	Parallel with grain of paper..	37.0
A-2.....	12.0	0.39	3.1	Dry.....	do.....	39.0
A-3.....	12.0	0.39	3.1	Dry.....	do.....	37.5
Average..						37.8
B-1.....	12.0	0.39	3.1	Dry after having been sat- urated.....	Parallel with grain of paper..	39.0
B-2.....	11.9	0.38	3.0		do.....	35.5
Average..						37.2
C-1.....	11.9	0.40	3.8	Saturated.....	Parallel with grain of paper..	10.0
C-2.....	12.0	0.38	3.7	Saturated.....	do.....	6.0
Average..						8.0
D-1.....	12.0	0.38	3.1	Dry.....	Perpendicular to grain of paper.....	90.5
D-2.....	12.0	0.34	3.2	Dry.....	do.....	93.0
D-3.....	12.0	0.39	3.2	Dry.....	do.....	92.0
Average..						92.8
E-1.....	12.0	0.38	3.0	Dry after having been sat- urated.....	Perpendicular to grain of paper.....	89.5
E-2.....	12.0	0.38	3.1			87.0
Average..						88.2
F-1.....	12.0	0.38	3.7	Saturated.....	Perpendicular to grain of paper.....	15.5

APPENDIX E.—PREVIOUS INVESTIGATIONS

	Page
1. Bauschinger's tests.....	365
(a) First series.....	366
(b) Second series.....	366
2. Tests by Möller and Lühmann.....	367
3. Hamburg tests.....	369
(a) First series.....	370
(b) Second series.....	370
(c) General results.....	371
4. Fire test of column at Vienna.....	371
5. New York tests.....	372
6. Waite's tests.....	373
7. Tests by McFarland and Johnson.....	373

The first experimental investigations on the fire resistance of building columns were made abroad, principally in Germany. When iron came into use as a structural material in that country, it was thought that the possibility of constructing truly fireproof buildings was realized, for iron is not combustible. Extensive fires showed, however, that, while the structural framework did not burn, the building collapsed suddenly and without warning. It was early recognized that some wooden structures offered greater resistance to fire than those built of unprotected iron, a result that occasioned no little comment when first observed. In fact, iron as a structural material was for a time in considerable disrepute. At one time the use of unprotected cast-iron columns under main bearing walls was forbidden in Berlin but wrought-iron columns permitted. Subsequent to large fires in Berlin and Hamburg the reverse was true. Similar changes in opinion were evident as to whether cast and wrought iron columns should be given fire-protective coverings and whether such coverings should be removable or permanent.

1. BAUSCHINGER'S TESTS³

In 1884-1886 Prof. J. Bauschinger, of Munich, Germany, made two series of fire and water tests on building columns that were loaded in a horizontal testing machine and heated by wood fire in a wrought-iron trough placed under them. Water was applied to the top surface. Temperatures were measured by alloys of tin, lead, and silver, having computed melting points of 300, 400, 500, and 600° C, that were attached to rods and held against the surface of the column at the middle of the sides to obtain the average temperature. Deflections were measured in the vertical and horizontal directions by indicators attached to wires that were fastened to the column at the middle of its length. The columns were loaded to what were considered safe working loads and subjected to three successive fire and water tests, the surface temperatures it was aimed to attain when water was applied being generally 300° C, 400 to 600° C, and red heat above 600° C. These tests were made with the column ends fixed or restrained. If failure did not occur, a final test at red heat with the column ends unrestrained was made in most cases. The method of testing was largely determined by the fact that opinions differed as to whether damage to cast-iron columns in building fires was caused by the fire or by the application of water to the red-hot metal.

During the first part of each fire test the column deflected downward toward the fire, due to the unequal heating, but with increase of temperature greater uniformity obtained and the column straightened somewhat. The application of water to the upper surface caused another sharp deflection downward, which became less as the

³ Mittheilungen aus den Mech. Tech. Lab. d. k. Tech. Hochschule, München, Heft 12, 1885; Heft 15, 1887.

column cooled on all sides, the final deflection being in some cases upward. The test conditions and effects approximate to some extent those pertaining to unprotected columns under exterior walls.

(a) **FIRST SERIES.**—In the first series tests were made on six cast-iron, three wrought-iron, and 15 columns of other building materials, including Portland-cement mortar, brick, and several kinds of building stone.

The cast-iron columns had been rejected for building purposes because of uneven wall thickness, "cold shuts," and other defects, and were more or less ornamental in form, the types varying from the plain cylindrical and slightly tapering shafts to the form having an ornamental base for about one-third of the length and a tapering shaft, which in one case was deeply fluted. They had plain or highly ornamental capitals and were from 11 feet to 13.8 feet in length and from 5.8 to 7.6 inches in outside diameter, as measured at the midheight of the column, with average wall thicknesses of 0.40 to 1 inch, the thickness varying considerably within each column. Working loads of 3400 to 5800 pounds per square inch were applied, the stress depending on the slenderness ratio of the column.

As tested with restrained ends the cast-iron columns supported their full load in the fire and water tests, although cracks developed in some cases at the higher temperatures when water was applied. Maximum vertical deflections of $3\frac{1}{2}$ inches were observed after application of water on columns of nearly uniform wall thickness. With columns of uneven wall thickness the deflections were larger. In the tests with unrestrained ends, on application of water the resulting deflection made the columns unable to support full load and caused some of them to break.

The wrought-iron columns were about 13 feet (4 meters) in length and consisted of one welded tube 5.04 inches outside diameter and 0.24 inch thick, and two built-up box columns, one of two 7-inch channels and two plates fastened together by bolts spaced about 16 inches on centers, and the other of two 7-inch I beams and two plates fastened with bolts spaced about 7 inches apart. The loads were 6200, 7600, and 6700 pounds per square inch, respectively.

The welded tube took a large deflection before the temperature had reached 600° C and failed to sustain full load. A slight application of water increased the deflection and caused it to fall out of the machine. The two other columns were heated to 300 and 400° C, followed in each case by water application, their behavior being similar to that of the cast-iron columns, only the deflections were larger. On heating to 600° C and applying water the deflections became so large that the full load could not be carried, and some of the bolts were sheared off.

Of the tests on other building materials that with Portland-cement mortar was made on a column about 12 inches (30 cm) square and 10 feet (3 m) long. The proportions of the mixture was 1:5 Portland cement and coarse sand. The column was tested at the age of 6½ months under a working load of 95 pounds per square inch. It was heated for 1¾ hours until a temperature of 600° C was attained at the middle of the sides, when water was applied. No apparent injury resulted, and when cold it was loaded to failure at 920 pounds per square inch.

The brick column was about 12½ inches (32 cm) square and 6.6 feet long with brick laid in Portland-cement mortar and the outside covered with a layer of Roman-cement plaster about 0.60 inch (1.5 cm) thick. It was exposed to fire for one hour attaining a temperature of 600° C on the sides, when water was applied. No apparent damage other than slight cracking and flaking of the plaster resulted, the column being subsequently loaded to failure at load of 520 pounds per square inch.

(b) **SECOND SERIES.**—Objections raised against Bauschinger's tests, in particular that the loads applied to the cast iron-columns were too small and that the component

parts of the wrought-iron columns were bolted together at intervals too far apart to secure a rigid section led Bauschinger to conduct a second series of tests in 1886 on two cast and five wrought iron columns. The cast-iron columns were about 13 feet (4 m) long; one had outside diameter of 7 inches and average wall thickness of 1.1 inch, and the other had outside diameter of 6.1 inch and average wall thickness of 1 inch, the applied working loads being 8400 and 7100 pounds per square inch, respectively. The columns had fairly uniform wall thickness and were apparently of better quality than the cast-iron columns of the first series. They were tested in a manner similar to those in the first series, and although large deflections occurred when heated to red heat and suddenly cooled by water application, they sustained their full working load. No cracks due to water application developed.

The wrought-iron columns were about 19 feet long and consisted of two plate and channel box sections and three starred-angle sections. The former were built up of two 5.7-inch channels and two $\frac{5}{8}$ by 8 inch (20 cm) plates with rivets spaced 3 to 5 inches on centers, and were loaded to 6300 pounds per square inch. The starred-angle sections were of two sizes, two being built up of four $3\frac{1}{2}$ by $3\frac{1}{2}$ by $\frac{7}{8}$ inch angles spaced $2\frac{1}{4}$ inches back to back and riveted together rigidly at the ends and at two intermediate points about 6 feet apart. The angles of the third column of this type had $3\frac{1}{8}$ by $3\frac{1}{8}$ by $\frac{3}{8}$ inch (1 cm) angles spaced 1.4 inches back to back and were riveted at the ends and at five intermediate points about 3 feet apart. The working loads for the two types were 6300 and 5300 pounds per square inch, respectively.

All five columns were heated red hot, and water applied until they were cold. The two box columns carried their full load throughout the test, although large permanent deflections were produced. The angle columns deflected downward rapidly and were unable to carry their full load at 600° C. Water application increased the deflection and caused a further decrease in the sustained load. The rivets were distorted, but none were sheared off.

2. TESTS BY MÖLLER AND LÜHMANN⁴

In 1887, M. Möller, a Government architect, and R. Lühmann, an engineer, obtained a prize offered by the (German) Society for the Promotion of Industrial Progress for the best essay on the subject of the "Resistance of Iron Columns when Subjected to High Temperatures," submitting to that end a paper in which were described tests conducted by them at Hamburg, Germany.

From the remarks accompanying the announcement of the competition it is evident that the principal interest centered in the relative resistance to fire and water of unprotected cast and wrought iron columns, although the behavior of masonry piers at high temperatures was also considered of importance.

In planning their investigation, Bauschinger's work was carefully considered by Möller and Lühmann, who concluded that in both the first and second series of tests by the former, the wrought-iron columns were overloaded, resulting in relatively greater bending as the columns deflected under the unsymmetrical heating and giving greater extreme fiber stress than the columns could withstand. It was held that a comparison of cast and wrought iron columns should be made with specimens identical in length, lateral dimensions, and moment of inertia.

The specimens tested by Möller and Lühmann were 3.28 feet (1 m), 6.56 feet (2 m), or 13.12 feet (4 m), in length. The shortest length was chosen to give approximately the compressive strength of the material, the intermediate length was in about the

⁴ *Verhandlung des Vereines zu Beforderung des Gewerbflusses*, 66, p. 573 and 701; 1887.

same ratio to the lateral dimensions as of columns in actual use, and the longest was taken as representing unusually large values of that ratio.

The cast and wrought iron specimens, all approximately 9.8 square inches in cross-sectional area, comprised solid and hollow cylindrical forms, the former $3\frac{1}{4}$ inches in diameter, the latter about 6 inches in outer diameter with wall thickness slightly less than $\frac{5}{8}$ of an inch; a hollow, cast-iron, ornamental, fluted column of approximately the same diameter and area as the other hollow sections; and a riveted form of rectangular section (6 by 7 inches) built up of four angles united on two sides by plates and on the other sides by lattice bars, the component parts being rigidly bolted and riveted together. The cast iron columns had fairly uniform wall thickness.

As adding somewhat to the resistance to fire and water, one of the wrought-iron and two of the cast-iron columns were filled with cement, a 2-inch gas pipe being placed concentrically in one of the latter to maintain alignment of the pieces when the column broke. Quite full protection was given one each of the hollow cast and wrought iron specimens by a covering of 1:3 cement mortar about $2\frac{1}{2}$ inches thick, interlaced with wire. Also, one of the riveted angle columns was protected by a circular wood mantle about $1\frac{1}{4}$ inches thick, covered with sheet iron.

There were further included oak and fir columns 6 inches square and columns of brick 9 inches square set in cement mortar.

Lengths other than 6.56 feet were used only in the unprotected cylindrical cast and wrought iron specimens. Report was made on about 40 tests, of which approximately two-thirds were fire and water tests, the rest being load tests at normal temperature.

The testing apparatus was very similar to that used by Bauschinger, the columns being loaded in a horizontal hydraulic press and heat applied by a coke fire in a U-shaped trough placed beneath the column. Temperatures of 330, 440, and 600° C were measured by metals and alloys placed in contact with the outside of the columns. Vertical deflections were measured at the mid-point of the length, using a lever with one of its ends resting on the column.

The loads were applied with the column axis uniformly 0.39 inch (1 cm) below that of the machine, the authors holding that central loading of columns does not occur in buildings except by chance, and that the above eccentricity is about what may be expected in practice.

In general the tests were conducted with the column ends unrestrained, the machine being fitted with spherical bearing blocks, but to determine the difference a few columns were loaded between fixed parallel bearing plates. Some of the columns were loaded at ordinary temperatures, and the safe load, as judged apparently by the limit of permissible deflection, and the maximum load were determined. The same quantities were determined for the high temperature condition, in general after 330 and 600° C had been attained on the upper and lower sides of the column, respectively. The load determinations at high temperature were generally made when the maximum deflection had been induced, which occurred at the time of water application.

On application of maximum load while hot, the short hollow columns (1 m long) deflected upward due to yielding of the metal at higher temperature on the lower side of the column. The solid columns of this length, in common with all of the longer columns, deflected downward due to the uneven heating, the deflections increasing on application of water and load. The following tabulation gives a summary of the results obtained with unprotected hollow and solid columns of cast iron and wrought iron:

Form and material	Effective length	Slender-ness, ratio $\frac{l}{r}$	Fire and water test		Test at normal temperature	
			Maximum load	Deflection	Maximum load	Deflection
	Feet		Lbs./in. ²	In.	Lbs./in. ²	In.
Hollow cast iron.....	4.2	27	16 500	0.98	43 700	1.02
Hollow wrought iron.....	4.2	27	13 500	1.97		
Hollow cast iron.....	7.5	47	23 000	1.77	37 200	.39
Hollow wrought iron.....	7.5	47	10 800	1.97	23 000	.78
Solid cast iron.....	4.2	57	9220	1.17	39 300	.33
Solid wrought iron.....	4.2	60	9100	.28	22 100	.63
Solid cast iron.....	7.5	101	6640	1.93	10 900	.98
Solid wrought iron.....	7.5	107	5000	1.97	13 500	.18
Hollow cast iron.....	14.0	89	2650	4.75	15 300	1.77
Solid cast iron.....	14.0	190	1890	6.80		
Solid wrought iron.....	14.0	201	1620	4.56		

The decrease in strength with length was marked, particularly for the columns loaded after fire exposure and water application. This was due to the larger lateral deflections caused by unsymmetrical heating, cooling, and loading induced in the longer columns, test conditions that can hardly be said to duplicate typical fire conditions even for unprotected exterior wall columns. In the shorter lengths, the cast iron sustained higher average unit loads than the wrought iron, the difference becoming smaller with increase of length or slenderness ratio. The load reported as safely carried after fire and water application varied from a little over one-half of the maximum load finally sustained to nearly the full value of the latter. Failure was due in all cases to bending produced mainly by uneven temperature distribution over the column section. The tests developed no cracking of the hot cast iron due to water application except in the case of the longest column, in which case the crack could have been incidental to failure as caused by the large deflection.

The columns tested between fixed parallel plates withstood higher loads than those tested with spherical end blocks, although the tests were too few in number to afford definite comparison.

The filled columns sustained slightly higher loads under the same test conditions than the corresponding unfilled columns. The metal of the columns protected by mortar was only moderately warm after the column had been in the fire about $1\frac{1}{2}$ hours, and that of the column protected by wood was 120° C after the covering had been removed subsequent to a 63-minute fire exposure. The strength of the brick columns was reduced about 50 per cent after 40 to 45 minutes' fire exposure. The oak column failed due to reduction of area and strength of the wood after 8 minutes under load of 1240 pounds per square inch, as computed on the original area, and the fir column after 18 minutes in the fire under load of 1110 pounds per square inch.

3. HAMBURG TESTS⁶

Two series of fire tests on loaded building columns were undertaken at Hamburg, Germany, in the period 1893 to 1895, by representatives appointed by the Hamburg

⁶ Vergleichende Versuche über die Feuersicherheit von Speicher-stützen. Commissions—bericht Hamburg, 1895.

Vergleichende Übersicht über die Feuersicherheit gusseiserner Speicher-stützen. Hamburg, 1897.

Zeitschrift d. Verein Deutch Ing., 40, p. 159; 41, p. 1007; 42, p. 183.

Stahl und Eisen, 18, p. 696.

senate from the building commission, the fire department, the warehouse association, and the insurance interests.

(a) **FIRST SERIES.**—In the first series tests were made on wrought-iron columns built up of four angles united by latticed bars and plates to form rectangular sections. A few columns were tested unprotected, two were filled with cinder concrete, and nine were protected by coverings from $\frac{3}{4}$ to 2 inches in thickness, consisting of concrete (Monier) blocks, gypsum and magnesite blocks or boards, pressed cork, and asbestos cement. Some of the coverings were protected by a sheet-metal mantle. In the series were also included three wooden columns 11.8 inch square, one of which was covered by sheet metal.

The question of the relative fire resistance of the several column materials was still of interest, and, although fire-protective coverings had come to be employed, their use had not gained such headway as in America, where hollow tile, at that date not manufactured in Germany, was the almost universal covering material. The importance of simulating in the tests actual loading and fire conditions was recognized.

The columns were loaded by a hydraulic ram in upright position within a framework in which were inserted two platforms 11½ feet apart, to which the test column was attached. Heat was applied over a length of about 4 feet in the middle portion of the column by a gas-fired oven built in halves which swung together about the column. Temperatures were measured by Seger cones, fusible alloys, and a thermoelectric pyrometer, and lateral deflections with rods passed through holes in the furnace walls. Working load was applied, centrally or eccentrically, and heating continued until the column was unable to sustain it, when water was applied over the heated length. The furnace temperature rise varied considerably between different tests and was at times quite rapid, 1200° C being sometimes attained in 2 hours and 1400° C in 4 hours.

The unprotected wrought-iron columns of the first series failed after 17 to 59 minutes' fire exposure, depending on the rate of temperature rise, the metal temperature at failure being given at about 600° C, and the load sustained 14 200 pounds per square inch (1000 kg per sq. cm). The filled columns proved to be only slightly more resistive than the protected and unfilled columns. The protected wrought-iron columns failed after 1¾ to 4 hours, with maximum furnace temperature of 1200 to 1300° C, the load applied being apparently the same as for the unprotected columns. The temperatures inside of the column were not known, except that they were over 412° C.

The application of water resulted in injury to the concrete coverings, and destruction of the other coverings, except where covered with sheet metal.

The wooden columns failed after a furnace exposure averaging a little over 1 hour for the three tests, the reduction in cross section from combustion of the wood being about 40 per cent.

(b) **SECOND SERIES.**—Tests were made on about 24 columns of cast iron of 10.8 inch outside diameter, 22 of which had wall thicknesses of 1.18 inch, and two 0.47 inch, the columns being cast in vertical position. Of the total number 17 were protected by coverings 1½ to 2 inches (4 to 5 cm) thick consisting of the materials used in the first series, to which were added coverings of tufa stone and asbestos-

kieselguhr. A number of the coverings were encased in sheet metal. The effect of an air space between the covering and column and of free convection within the column were studied, as, also, the difference between permanent as distinguished from removable coverings, some members of the commission holding that coverings should be removable to permit examination of the column from time to time on account of danger from rust.

The method of testing was substantially the same as in the first series. The unprotected cast-iron columns failed in from 35 to 59 minutes, depending on the intensity of the fire, under load of 7100 pounds per square inch (500 kg per sq cm), and temperature of the column reported as 800° C. Increasing the column load to 10 700 pounds per square inch (750 kg per sq cm) caused failure to occur with column temperature of 700° C. The thin-walled column gave a little lower resistance than those of greater thickness. The application of water to unprotected columns that had failed in the fire test and suffered large deformation generally caused cracking.

The protected columns failed after 3 to 5½ hours, with maximum furnace temperatures from 1200 to 1500° C, except the one covered with 2-inch thickness of asbestos-kieselguhr, which did not fail after a fire exposure of 7 hours. The air space between the column and the covering increased the fire resistance a little and the provision for free convection within the column delayed failure by nearly one hour. The action of water on the coverings was the same as in the first series.

(c) GENERAL RESULTS.—The Hamburg tests were the first to determine the strength of wrought iron and cast iron, applied as columns, under central load and symmetrical heating. The column temperatures reported, regarded as those of the metal at failure, were without doubt higher than the actual temperatures, due to the methods of measurement employed. As column tests they are open to objection in that only about one-third of the column length was heated. The tests on protected columns while giving little information on the effectiveness of covering materials now in use, proved conclusively the great gain in fire resistance attainable by protecting the metal with materials of low heat conductivity.

With regard to water application on cast-iron columns at high temperature, the results are not conclusive, since in almost all tests the columns had sustained large deformations by failure in the fire test before water was applied.

4. FIRE TEST OF COLUMN AT VIENNA⁶

In 1893 the Building Department of Vienna conducted a fire and water test on a single wrought-iron column 11½ feet long, built up of two 5½-inch channels connected by lattice bars. The column was protected by brickwork 5½ inches thick, laid in fire clay mortar, the interior of the column being unfilled.

The furnace was of brick 8 by 12 feet, and with height equal to the column length. Wood piled 3 to 4 feet high was used for fuel, and the temperatures of furnace and column were indicated by alloys. Load was applied to the column section, using a long lever. After a fire test of 2½ hours duration, wherein furnace temperatures evidently much in excess of 400° C were attained, a hose stream was applied to one side of the column.

The only apparent damage caused by the combined test was considerable spalling of the bricks at the corners, cracking of bricks in the upper half of the covering, and washing away of some of the mortar. The maximum temperature indicated inside of the column was 65° C.

⁶ Engineering News, 82, p. 184; Sept. 6, 1894. Translation of article in Zeitschrift des Oestereichischen Ingen. u. Arch. Verein.

5. NEW YORK TESTS¹

Fire tests of unprotected columns, two of structural steel and two of cast iron, and a fire and water test of one unprotected cast-iron column, were made in 1896 under the direction of a committee appointed by the Tariff Association of New York, the Architectural League of New York, and the American Society of Mechanical Engineers.

The steel columns were 14 feet long, one being a box section built up of two 10-inch channels and two 12 by $\frac{1}{4}$ inch plates, and the other a Z-bar section consisting of four 4 by $\frac{5}{8}$ inch Z-bars riveted to a plate $6\frac{1}{2}$ inches wide, other details of design being in accord with accepted standards of practice. The cast-iron columns were 13 feet long, of 8-inch outside diameter and 1-inch wall thickness, and were cast in horizontal position with dry-sand core, the ends being flanged and end bearings machined.

The columns were tested in vertical position within a furnace chamber about 12 feet square and 14 feet high, the fuel used being producer gas admitted by burners extending through the floor of the furnace. Air was supplied through openings in the floor near the gas inlets. Arrangements were made for intensifying the fire when necessary by injecting a naphtha spray into the gas main supplying the burners. Load was applied to the column within a restraining frame of structural steel by a hydraulic ram placed beneath the column, filler blocks of cast iron transmitting the load to the column.

In the fire tests it was intended to subject the columns to working loads, and for one test each of steel and cast-iron columns to use a slow and a rapid furnace temperature rise. Furnace temperatures were measured with an Uhling-Steinbart transpiration pyrometer. Temperatures on the metal of the columns were not measured in any of the tests.

In the test of the plate and channel column, using a slow temperature rise, a furnace temperature of about 650° C was attained in one hour, which temperature maintained for about 20 minutes caused the steel to show red color and fail under a load of 6400 pounds per square inch, trouble with the loading equipment preventing the application of the full working load.

The Z bar column, tested under full working load of 12 000 pounds per square inch and rapid furnace temperature rise, failed 25 minutes after the beginning of the test. A maximum furnace temperature of about 750° C was indicated at 14 minutes, which fell off to 600° C at failure.

The cast-iron columns were tested under working load of 7700 pounds per square inch. In the test with slow temperature rise the column began to show color after 65 minutes' fire exposure, the furnace temperature being near 600° C. After another 18 minutes, during which the furnace temperature averaged 640° C, the gas was shut off and the furnace door opened for 9 minutes, which disclosed the test column to be decidedly red and bent. The door was closed and the test continued at lower furnace temperature for 28 minutes, the column sustaining its load, although greatly bent. No cracks developed. The permanent lateral deflection was $3\frac{1}{2}$ inches.

The cast-iron column tested with rapid temperature rise had deflected a visible amount at 35 minutes and began to show color at 39 minutes, the furnace temperature at this time being 730° C. It failed at 43 minutes with complete fracture across the section near the middle.

In the fire and water test, water was applied at four successive times following fire exposures of increasing intensity. Before the third application the furnace temperature was 580° C, and the column showed color. Immediately before the last water

¹ Trans. Am. Soc. of Mech. Eng., 18, p. 24, 1896-1897.

application a furnace temperature of 700°C was indicated, and the column was red and bent. No cracks developed. The permanent lateral deflection was $3\frac{1}{4}$ inches.

The tests made apparent that unprotected metal columns will fail in fires of moderate intensity after a comparatively short exposure, the steel columns being less resistive than those of cast iron under the respective unit loads for which the two types are generally designed. Injury to cast-iron columns due to water application while hot is shown to be improbable with columns of the given proportions. The tests give little information on the temperature in the metal at failure or on definite time resistance under comparable fire conditions.

6. WAITE'S TESTS

In 1903 the Guy B. Waite Co., of New York, conducted some fire and water tests on floors and partitions, under the supervision of the Manhattan Bureau of Buildings, constructing to that end a reinforced-concrete test house, in which were placed, for the purpose of studying incidentally the behavior of column coverings, a reinforced-concrete column made with stone aggregate, proportion of mixture, 1 : 2 : 4, and two cast-iron columns, one of which was covered with $1\frac{1}{2}$ inches of poured cinder concrete of proportion 1 : 5, Portland cement, and hard-coal cinders, the other with a gypsum and cinder composition.

The reinforced-concrete column was considerably pitted by water from a $1\frac{1}{8}$ -inch nozzle at 60 pounds' pressure after it had been subjected for 4 hours to a test fire with average temperature of about 930°C . The cinder-concrete covering was in excellent condition after it had gone through the same test, and, in addition, three tests of 1-hour duration with water application at the end of each. The gypsum covering appeared in good condition at the end of a single fire test, but on water application it was partially washed away.

7. TESTS BY MCFARLAND AND JOHNSON⁸

Tests to determine the effect of fire and water on the strength of reinforced-concrete columns were made in 1906 by H. B. McFarland and E. V. Johnson at the Chicago laboratory of the National Fireproofing Co.

The columns, three in number, about $10\frac{1}{2}$ inches square and 12 feet long, were made of 1 : 2 : 4 limestone concrete and reinforced with $\frac{3}{8}$ -inch rods placed near the corners. Two columns were tested in compression at normal temperature at age of 2 months and $3\frac{1}{2}$ months, respectively, and sections 5 to 6 feet long cut from them outside of the region of failure for use in the fire test. One of these specimens was covered with 3-inch solid porous clay tile and the other was tested unprotected, and after the fire test subjected to water application. The third column was cut in two, one part for use in the fire test and the other for a comparable compression test at normal temperature. The age of all columns was 23 months at the time of the fire test.

The three specimens were placed on end in a wood-fired furnace and subjected under no load to a 3-hour fire test. Furnace temperatures, as indicated by a Bristol thermoelectric pyrometer, ranged from 800 to 1000°C for the greater portion of the period. On the day following the fire test they were tested in compression.

The section protected by clay tile was little affected by the test and developed compressive strength of 3127 pounds per square inch. An 18-inch long specimen cut from the same column but not subjected to fire test developed 3558 pounds per square inch. The specimen to which water was applied after the fire test failed in the compression test at 674 pounds per square inch. The column from which it was cut had

⁸ Engineering News, 56, p. 316; Sept. 20, 1906.

developed 2116 pounds per square inch at age of $3\frac{1}{2}$ months, the fire and water treatment having apparently caused a decided decrease in strength. A similar effect was indicated in the case of the third specimen, its strength being 711 pounds per square inch, against 2565 pounds per square inch for the section of the same column that was not exposed to fire.

The large reduction in strength sustained by the unprotected columns can be ascribed, in part, to their small size, larger columns being subject to smaller percentage reduction in strength due to surface damage from fire exposure.

WASHINGTON, June 4, 1920.

APPENDIX F

CENTIGRADE AND FAHRENHEIT CONVERSION TABLE

C.°	0	10	20	30	40	50	60	70	80	90
	F.°	F.°	F.°	F.°	F.°	F.°	F.°	F.°	F.°	F.°
0	32	50	68	86	104	122	140	158	176	194
100	212	230	248	266	284	302	320	338	356	374
200	392	410	428	446	464	482	500	518	536	554
300	572	590	608	626	644	662	680	698	716	734
400	752	770	788	806	824	842	860	878	896	914
500	932	950	968	986	1004	1022	1040	1058	1076	1094
600	1112	1130	1148	1166	1184	1202	1220	1238	1256	1274
700	1292	1310	1328	1346	1364	1382	1400	1418	1436	1454
800	1472	1490	1508	1526	1544	1562	1580	1598	1616	1634
900	1652	1670	1688	1706	1724	1742	1760	1778	1796	1814
1000	1832	1850	1868	1886	1904	1922	1940	1958	1976	1994
1100	2012	2030	2048	2066	2084	2102	2120	2138	2156	2174
1200	2192	2210	2228	2246	2264	2282	2300	2318	2336	2354
1300	2372	2390	2408	2426	2444	2462	2480	2498	2516	2534
1400	2552	2570	2588	2606	2624	2642	2660	2678	2696	2714
1500	2732	2750	2768	2786	2804	2822	2840	2858	2876	2894
1600	2912	2930	2948	2966	2984	3002	3020	3038	3056	3074
1700	3092	3110	3128	3146	3164	3182	3200	3218	3236	3254
1800	3272	3290	3308	3326	3344	3362	3380	3398	3416	3434
1900	3452	3470	3488	3506	3524	3542	3560	3578	3596	3614
2000	3632	3650	3668	3686	3704	3722	3740	3758	3776	3794

C.°	F.°	F.°	C.°
1	1.8	1	.56
2	3.6	2	1.11
3	5.4	3	1.67
4	7.2	4	2.22
5	9.0	5	2.78
6	10.8	6	3.33
7	12.6	7	3.89
8	14.4	8	4.44
9	16.2	9	5.00
10	18.0	10	5.56
		11	6.11
		12	6.67
		13	7.22
		14	7.78
		15	8.33
		16	8.89
		17	9.44
		18	10.00

Examples:

515° C. = 950° F. + 9° F. = 959° F.
2039° F. = 1110° C. + 5° C. = 1115° C.