REINFORCED CONCRETE

A MANUAL OF PRACTICE

CHICAGO AND NEW YORK
THE MYRON C. CLARK PUBLISHING CO.
1908

REINFORCED CONCRETE

A MANUAL OF PRACTICE

BY

ERNEST McCULLOUGH

M. W. S. E., Author of "Engineering Work in Towns and Cities;" "The Business of Contracting." "Engineering Contractors' Pocket Book," Etc., Etc.

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NOTES AND CORRECTIONS.

The formulas given in the first twenty-two pages are those developed by Professor Talbot, and credit is given on page 17. The same formulas are found in Turneaure & Maurer's "Principles of Reinforced Concrete Construction."

Page 13. The definition of the modulus of elasticity given on this page is to be found in Trautwine's Civil Engineers Pocket Book, which has been a standard since 1876. The description on this page of how the modulus is obtained is to be found in all the standard works on mechanics of materials.

Page 22. Seventh line from the top. Formula should read—

$$M = fpd'bd^2 \quad \text{or} \quad M = fpd'dbd \quad \text{or} \quad M = fAd'd$$

Page 59. Twenty-third line from top. "The maximum stress is 0.853h," should read "the maximum stress is 0.583h," etc.

Page 60. Top and second line should read "and D=internal diameter in feet."

On the same page near the bottom, the two-thirds rule for pressure on footings applies only to rectangular surfaces and not to circular. It is customary to construct rectangular footings for chimneys.

Page 107. Seven lines from bottom. The acid bond solution is generally composed of ninety parts water and ten parts chemically pure hydrochloric acid, although some men use stronger solutions.

Pages 31 and 98. The custom of stopping work in the middle of slabs and in midspan of beams instead of at the supports is spreading and results obtained seem to justify it. The joint is made where there is the least amount of steel and where consequently stops can be readily placed. Care should be taken to secure a good junction of new work to old when resuming work. This method does away with practically all the objections to T beams.

August 15, 1908.

Reinforced Concrete; A Manual of Practice. McCullough.

G. F.

To
W. A. STEVENSON, ESQ.,
of the
NORWOOD ENGINEERING CO.,
This Book is respectfully
DEDICATED.

PREFACE.

This book is intended to be what its title indicates, A Manual of Practice. The intention in the sections on design has been to keep within the usual requirements of the ordinary conservative building ordinances of American cities. This explains the use of the straight line formulas for stress and the limitations imposed by the employment of working stresses. The ambitious designer wishing to learn more of the theory of the subject and the design of higher structures can go to the larger standard treatises and have nothing to unlearn.

So far as construction is concerned, the principles stated herein as the result of personal experience, apply to all manner of work in reinforced concrete, and to this extent the book should be of some value to a large number of men. Good workmanship implies no knowledge of, or dependence upon, theory of design. It simply calls for the exercise of common sense, unremitting vigilance and care on the part of the man in charge, a willingness to learn from every intelligent workman on the job, and the kind of pride that takes honestly to heart the lessons of experience. The motto, if the manager is the sort of man who believes in mottoes, is *Do not forget!*

If the manager, however, does not know something about the theory of design, then the owner is taking great chances. The man in charge should be an engineer.

The writer feels the preface would be incomplete unless he acknowledged here the assistance given him by his son George Seymour McCullough in checking and recalculating the tables, many of them original. The work was at times irksome, and we trust no errors will be discovered.

Some assistance received from readers who followed the articles as they appeared in The Cement Era has been acknowledged in the proper places in preparing the articles for publication in book form.

Errors and omissions discovered and reported by readers will be gratefully acknowledged, and whatever the readers may do to assist in making future editions more valuable will be appreciated.

E. McC.

Chicago, Ill., U. S. A., May, 1908.

CONTENTS.

CHAPTER I.—STRENGTH OF BEAMS.............................. 9
Simple, Practical Formulas—Neutral Axis—Modulus of Elasticity—Tables of Factors—Elastic Limit—Per Cent of Steel—Design of a Beam—Strength of Concrete of Stone and Cinders—Fibre Stresses—Empirical Formulas—Beam Failures—Web Stresses—Stirrups—Weights and Areas of Steel—Double Reinforced Beams—T Beams—Allowable Stresses in Building Ordinances—Adhesion of Concrete—Floor Slabs—Continuous Beams.

CHAPTER II.—LOADS ON BEAMS.............................. 35
Reactions—Bending Moments—Deflection—Shear—Concentrated Loads—Distributed Loads—Combinations of Loads—Parallel Forces on Beams—Freely Supported Beams—Tied Beams—Cantilever Beams—Dead Load—Live Loads—Methods for Assuming Beam and Slab Calculations—Load Permitted by Building Ordinances—Highway Bridges—Sidewalks.

CHAPTER III.—COLUMNS.............................. 44
Comparison of Sizes in Different Materials—Methods of Reinforcing Columns—Ratio of Length to Thickness—Area—Tables of Column Divisors—Different Formulas for Strength and Stiffness—Increase of Strength with Added Steel—Comparison of Steel and of Steel and Concrete—Safe Fibre Stresses—Methods of Working—Pouring Columns—Foundations—Footings—Placing Vertical Steel.

CHAPTER IV.—WALLS, TANKS AND FOOTINGS.............................. 52
Plain vs. Reinforced Walls—Calculation of Pressure—Unit Pressures Surcharged Walls—Detailed Calculations for Wall—Restrained Walls—Circular Tanks—Design of Chimney—Footings.

CHAPTER V.—DESIGN AND COST.............................. 64
Examination of Reasons for Reinforced Concrete Construction—Adhesion—Shrinkage of Concrete—Connections—Use of Wire—Holes in Slabs—Comparison with Wood—Remarks on Cost and on Conduct of Work—Different Methods of Design—Forms—An Approximate Method of Estimating Costs.

CHAPTER VI.—FORMS.............................. 75
General Remarks from Practical Experience—Studded Frame Forms—Ransome Panels—Framed Panels—Board by Board Methods—Wiring—Bracing—Bolts and Ties—Spacers—Corner Forms—Beam and Slab Forms—Column Forms—Hints on Form Design and Estimating.

CHAPTER VII.—THE CONDUCT OF WORK.............................. 93
Unit of Measurement—Table of Mixtures—Hand Mixing—Machine Mixing—Measuring Aggregates—Wheelbarrows—Quantity of Water—Filling Forms—Finishing Surfaces—Dense Concrete—Tools for Good Work—Importance of Placing Steel—Pouring Concrete—Tables of Pressure of Concrete—Spacing of Wire and Bolts—Safe Loads on

Posts and Braces—Weights of Nails, Spikes and Bolts—Strength of Beams—Oiling Forms—Soap—Wetting Forms—Compressed Air—Steam for Cleaning—Joining New Work to Old—Perfect Joints—To Prevent Freezing—Protecting Work.

CHAPTER VIII.—TOOLS..111

Ordering Steel—Shears and Punches—Keeping Track of Steel—Forges—Benders for Steel—Cold Chisels—Cold Cutters—Snips—Pliers—Bars—Hammers—House Raising Jacks—Blocks and Tackles—Saws—Nail Pullers—Concrete Boxes and Doors—Hoists—Electricity—Gasoline Engines—Air Compressors—Steam—Pipe Coverings—Wheelbarrows—Concrete Carts—Dumping Buckets—Conveyor Systems—Sub-letting Work.

CHAPTER I.

Strength of Beams.

Formulas for reinforced concrete design are in reality very simple and can be used by any practical man. The majority of writers on the subject, however, write for men who retain a knowledge of the mathematics of college days and give the derivation of the formulas. Men who have forgotten their higher mathematics and men who have not gone to college look for something short and easy and use empirical formulas and easy rules rather than wade through pages of mathematical reasoning to finally reach

$$M = K\ bd^2$$

which is the formula wanted and which is not recognized when reached.

It means that the resisting moment of the beam is equal to K (a moment factor), multiplied by the breadth of the beam times the square of the depth.

Nothing can be more simple, and when tables containing values of "K" are available the application is easy.

To obtain the size of a beam change the formula to read:

$$d = \sqrt{\frac{M}{Kb}}$$

in which M=bending moment=resisting moment in inch pounds.

d=depth in inches from top of beam to center of steel reinforcement.

b=breadth in inches.

The moment, M, is found when we know the clear span and load. The breadth, b, we can generally assume at 1/20 to 1/24 the span, and K is taken from tables.

Remember that in the following pages the depth of the beam is always the distance from the top to the center of the steel reinforcement. The concrete below the steel is simply there for protection and is not relied upon to add strength to the beam.

Assume the breadth, b, at from 1/20 to 1/24 the span, for the reason that the upper part of the beam is a column and to prevent side bending, or undue stresses at its junction with the slab on top, the length should not exceed 24 times the least thickness. Therefore assume a breadth as above and solve for d. The best shaped beam is one in which b lies between ½d and ¾d. To obtain such a beam may require two or more trials, until designing experience has been gained.

There must be enough concrete surrounding the steel to permit of a good grip. Practice has shown that this should be not less than 1 inch; and when the bars are more than ⅝ of an inch in diameter or thickness, this covering should be not less than one and one-half such thickness or diameter.

When several bars are used the space between them should be at least twice their thickness or diameter. If the beam is too narrow to permit of this they should then be in two planes, staggered. Rods or bars should be of such a size that their length will exceed 50 times their thickness. On this point we will speak later.

Grip is not alone to be considered. Where danger from fire is to be feared then the least covering should be 2 inches. Experiments have proven that the adhesion between steel and concrete is impaired by continuous submersion, so walls designed to hold water should have a covering of not less than 2 inches over the steel, and should be made of very dense concrete.

At this point the writer believes it will be well to give a few reasons. Few men are satisfied with a rule until they know why it exists in the particular form given. In this respect all writers agree, but in the present instance the formula is given first and the reasons follow. The practical man wants to know the meaning of the terms "Neutral Axis," "Elastic Limit" and "Modulus of Elasticity" and why the theorist makes use of them.

Neutral Axis.

The position of the neutral axis in a rectangular beam depends upon the material of which the beam is made. Wood, wrought iron and steel being practically as strong in compression as in tension are made into beams of such shape that the neutral axis is in the middle. Cast iron being about four times as strong in compression as in tension is made into beams shaped like a letter T upside down. The neutral axis is at a point which divides the area of the beam so that the area in compression is equal to one-fourth of the area in tension.

Concrete is about ten times as strong in compression as in tension, but is so much weaker than cast iron that it cannot be fashioned the same. So it is cast into rectangular beams and steel bars are placed in the bottom to take care of the tension.

The exact location of the neutral axis is very important in the design of reinforced concrete beams and slabs; but the undue prominence given this point has been the cause of much bad work, for when touching upon this point exact writers soar into realms of mathematics where most men cannot follow. Believing in a dim way that all the x's, y's and z's are a part of the rules, men with practical rather than theoretical training use rules given in manufacturers' catalogues or picked up loosely from any source, caring nothing for the derivation, provided they look easy.

When a beam bends, the fibres in the lower part, below the neutral axis, stretch. Above the neutral axis the fibres are compressed. The neutral axis therefore lies in a plane in which the longitudinal fibres undergo no change.

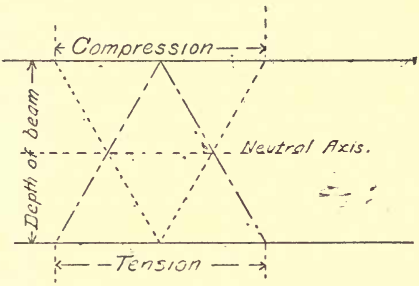


FIG. 1—DIAGRAM OF TENSION AND COMPRESSION.

Figure 1. The stretching force exerted at the bottom may be plotted as a horizontal line. Erect a perpendicular from the middle of the line, the height being equal to the depth of the beam. Connect the ends of all the lines and we have a triangle, the apex being at the top of the beam and the base being at the bottom. Through the apex draw another horizontal line, the length being equal by scale to the compressive force. Connect the ends to the middle of the base of the first triangle and we have a second triangle superimposed on the first. A perpendicular connecting the middle points in the base of each triangle, the side lines of each cross at a point proportional to the length of the bases. The neutral axis passes through the points where the side-lines cross.

The stress is proportional to the distance from the netural axis but we can see by the above description that the neutral axis is not like a fulcrum, but is located where opposing forces balance. All this preliminary explanation is necessary in order to show how "K" is obtained, that we may use the moment factor intelligently.

To some men educated in the mechanics of materials, Fig. 1 looks strange, so Fig. 3 will be more acceptable to them.

When a beam bends the assumption is made that a section plane before bending remains plane after bending; that is, a section of the beam such as would be made by a vertical saw cut. In Fig. 3 the section before bending is represented by the vertical line and the differences betwen its position and that of the inclined line represent the tensile and compressive stresses in the beam.

Representing the extreme fibre stress in the concrete by c, and the steel fibre stress by f, we have

$$fp = \tfrac{1}{2}ck$$

which will be explained in the next section.

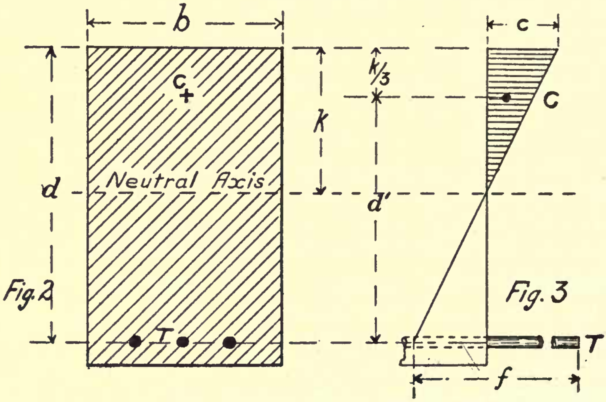


Fig. 2 *Fig. 3*

It is assumed that all tension is carried by the steel and that the unit deformation in any horizontal fibre varies directly as its distance from the neutral axis.

To illustrate this graphically look again at the two force triangles. Erase all the lines except the top one representing compression and the lower one representing tension. Join the opposite ends so there will be two small triangles with the apices at the neutral axis. Forces act through the center of gravity of bodies and the center of gravity of a triangle is one-third the distance from the base.

This explains $\frac{k}{3}$, which indicates the position of the point where the compressive strength of the concrete is concentrated.

Modulus of Elasticity.

This is a force which, if such a thing were possible, would stretch a material to twice its original length or compress it one-half. It is generally designated by the capital letter, E. In reinforced concrete work E_s is the designation for this force in steel, and E_c for concrete. The ratio $\frac{E_s}{E_c}$ is n. To determine the modulus of elasticity of a material, stretch or compress a piece of unit size, some definite amount by means of some definite force. Multiply the original length by this force. Divide the product by the area of cross-section multiplied by the amount of compression or stretch. The result is E.

E for steel is taken at from 29,000,000 pounds to 31,000,000. A value of 30,000,000 is commonly used. As the values are averages the difference between the E of high carbon and medium steel is not worth considering.

E for concrete varies with the mixture, the age and the care with which the concrete is mixed. It is not uniform throughout a piece of concrete.

It is usual in building ordinances to specify "n," and the ratios most in use are 8, 10, 12, 15 and 20.

Take beams of some unit width and some unit depth. Reinforce each with some percentage of steel and apply a certain load to each beam. Assuming some ratio of extensibility between the two materials we find that the position of the neutral axis varies with the percentage of steel and the ratio assumed between the moduli of elasticity.

Table I has been calculated by such a method. The values of "n" are shown at the tops of the columns. The letter "p" stands for percentage of steel, being the area of the steel divided by the product of the depth and breadth of the beam, the depth being distance from center of steel to top of beam.

The decimals give the values of "k" and show the distance from the top of the beam to the neutral axis, expressed in percentage of total depth, "d." The formula is:

$$k = \sqrt{\left(2\,p.\,n + p^2\,n^2\right)} - p.\,n$$

The values of "k" thus obtained, are used to calculate "K" in Table III. It is usual to assume some definite value in

TABLE I. Values of k.

Area of steel $=p$	$\frac{E_s}{E_c}=n$				
	8	10	12	15	20
.005	.25	.27	.29	.32	.35
.006	.27	.29	.31	.34	.38
.007	.28	.31	.33	.36	.41
.008	.30	.33	.35	.38	.43
.009	.31	.34	.37	.40	.45
.01	.33	.36	.38	.42	.46
.0125	.36	.39	.42	.45	.50
.015	.38	.42	.45	.48	.53

compression for concrete and some definite tensile stress for steel, and by so doing we can obtain values of "k" by this formula.

$$k = \frac{2\,p\,f}{c}$$

Where k = percentage of depth of neutral axis below top of beam.

p=percentage of steel.

f=fibre stress in steel per square inch.

c=extreme compressive stress in concrete.

Table II has been computed by means of the above formula, using values of "f" and "c" in common use.

The Chicago building ordinance will not permit a value of "c" exceeding 500 pounds per square inch. The value of "f" may be one-third the elastic limit. The value of "n" is fixed at 12. What steel can we use?

In Table I look under 12 for values of "k." In Table II we next look under 500 to get the same values, and find we can use from .7 to 1.0 per cent of steel stressed 10,000 pounds per square inch, from 0.6 to 0.9 of steel stressed 12,500 pounds, from 0.5 to 0.7 of steel stressed 15,000 pounds, or 16,000 pounds, 0.5 to 0.6 of steel stressed 18,000 pounds, or 0.5 of steel stressed 20,000 pounds. Of course for the stronger steel much smaller percentages might be used but less than 0.5 of steel makes an expensive beam, which remark will be better understood after learning how to use Table III. When a small percentage of steel is used the size of the beam is increased.

Looking in column 500 in Table III we obtain a moment factor "K" opposite the values of "k" taken from Table I and by the formula

$$d = \sqrt{\frac{M}{K\,b}}$$

the size of the beam may be computed. Narrow, deep beams are cheaper and stiffer than wide and shallow beams.

Beams should be limited if possible in width as already explained and the depth should not exceed 1/10 the span. When impossible to keep within the limits set, the internal stresses require careful investigation and provision must be made to take care of them.

TABLE II. Values of k.

Area of steel =p	f=10000				f=12500				f=15000			
	Values of c.				Values of c.				Values of c.			
	500	600	700	800	500	600	700	800	500	600	700	800
.005	.20	.17	.14	.13	.25	.21	.18	.16	.30	.25	.21	.19
.006	.24	20	.17	.15	.30	.25	.21	.19	.36	.30	.26	.23
.007	.28	.23	.20	.18	.35	.29	.25	.22	.42	.35	.30	.26
.008	.32	.27	.23	.20	.40	.33	.29	.25	.48	.40	.34	.30
.009	.36	.30	.26	.23	.46	.38	.33	.29	.54	.45	.39	.34
.01	.40	.33	.29	.25	.50	.42	.36	.31	.60	.50	.43	.38
.0125	.50	.42	.36	.31	.63	.52	.45	.39	.75	.63	.54	.47
.015	.60	.50	.43	.38	.75	.63	.54	.47	.90	.75	.64	.56

Area of steel =p	f=16000				f=18000				f=20000			
	Values of c.				Values of c.				Values of c.			
	500	600	700	800	500	600	700	800	500	600	700	800
.005	.32	.27	.23	.20	.36	.30	.26	.23	.40	.33	.29	.25
.006	.38	.32	.27	.24	.43	.36	.31	.27	.48	.40	.34	.30
.007	.45	.37	.32	.28	.51	.42	.36	.32	.56	.47	.40	.35
.008	.51	.43	.37	.32	.58	.48	.41	.36	.64	.53	.46	.40
.009	.58	.48	.41	.36	.65	.54	.46	.41	.72	.60	.52	.45
.01	.64	.53	.46	.40	.72	.60	.52	.45	.80	.67	.57	.50
.0125	.80	.67	.57	.50	.90	.75	.64	.56	×	.84	.72	.63
.015	.96	.80	.69	.60	×	.90	.77	.68	×	×	.86	.75

In Table III, "k" is the distance from top of beam to the neutral axis, $\frac{k}{3}$ is the distance down to the center of gravity of the compression triangle, d′ is the moment arm or distance from the steel to the point $\frac{k}{3}$ The moment factors K, in the columns headed 500, 600, 700 and 800 are obtained by the formula

$$K = \tfrac{1}{2}\, c\, d'\, k,$$

where c=fibre stress in most remote fibre of concrete.
d'=moment arm.
k=depth of neutral axis.

TABLE III. Values of Moment Factor K.

k	⅓k	d'	Values of c.			
			500	600	700	800
.25	.083	.917	57.3	68.8	80.2	91.7
.27	.09	.91	61.5	73.7	86.0	98.3
.28	.093	.907	63.5	76.1	88.8	101.5
.29	.097	.903	65.5	78.5	91.6	104.8
.30	.10	.90	67.5	81.0	94.5	108.0
.31	.103	.897	69.5	83.4	87.3	110.1
.32	.107	.893	71.5	85.7	100.0	114.1
.33	.11	.89	73.4	88.1	102.8	117.5
.34	.113	.887	75.3	90.5	105.3	120.5
.35	.117	.883	77.3	92.7	108.0	123.5
.36	.12	.88	79.2	95.0	110.9	126.8
.37	.123	.877	81.0	97.2	113.4	129.7
.38	.127	.873	83.0	99.5	116.0	132.7
.39	.13	.87	84.8	101.8	118.8	135.8
.40	.133	.867	86.7	104.0	121.4	138.8
.41	.137	.863	88.5	106.0	123.9	141.5
.42	.14	.86	90.3	108.3	126.3	144.4
.43	.143	.857	92.0	110.5	129.0	147.4
.45	.15	.85	95.6	114.8	134.0	153.0
.46	.153	.847	97.4	116.8	136.2	155.8
.48	.16	.84	100.8	120.9	141.0	161.0
.50	.167	.833	104.0	125.0	145.8	166.6
.53	.177	.823	109.0	130.9	152.8	174.5

Sometimes a wall or partition of concrete can be reinforced with steel and thus be strengthened to carry certain loads and relieve the footings. As the breadth and depth are fixed, assuming a certain percentage of steel, p, a stress, c, in the concrete and a depth, k, of the neutral axis, the fibre stress, f, of the steel is obtained by the formula

$$f = \frac{\tfrac{1}{2}\, c\, k}{p}$$

With the explanations given there should be no trouble in selecting a moment factor, K, when given p, c, f and n or when given two of them with the others assumed.

The theory on which the above treatment is based is deduced from experiments made at the University of Illinois, Engineering Experiment Station, under the direction of Prof. Arthur N. Talbot, but regards only what is called the straight line distribution of stresses for working loads. For ultimate loads the parabolic distribution of stresses is used.

Elastic Limit.

Nearly all materials have a certain amount of elasticity and will resume their original shape and size after being stressed. There is a point, however, beyond which they cannot be stressed without permanent distortion and this point is called the elastic limit. In steel it is from five to six-tenths the ultimate strength.

After steel has been stressed past its elastic limit it stretches faster under load and the cross-section, of course, is reduced. If imbedded in concrete, the adhesion is destroyed and as the steel stretches, more load is thrown on the concrete. Consequently the strength of the beam depends upon the elastic limit of the steel and the ultimate strength of the concrete, for concrete can hardly be said to have an elastic limit.

Up to within a year or two the claim was made that when steel having a high elastic limit was used, a considerable saving could be effected. If it took one per cent of steel having an elastic limit of 36,000 pounds per square inch then it would take only about six-tenths per cent of steel having an elastic limit of 64,000 pounds per square inch.

This is true, however, only when the question of deflection is not considered, but not true if it is. We have seen that the relative extensibility of steel and concrete must govern and that the E of high carbon and medium steel differ slightly, although one may have double the strength and nearly double the elastic limit of the other.

The steel stretches per unit of length as indicated by the division of the fibre stress in pounds per square inch by the modulus of elasticity.

The Ransome, Thacher, Kahn and Johnson bars made of medium steel have the elastic limit raised and the ultimate strength increased, by the processes to which they are subjected in preparing them for market. The drawing of wire has the same effect, in a greater degree, on the metal of which it is made.

Medium steel has been found more reliable in general structural work than high carbon steel, and it is to be supposed, is therefore best for reinforced concrete work. As the ultimate strength of a reinforced concrete beam depends upon the elastic limit and not upon the ultimate strength of the steel, for economical design, where not restricted by conservative building ordinances, a high elastic limit is desirable. This is furnished by the numerous deformed bars of medium steel and also by plain and twisted bars of high carbon steel made from old rails.

For situations where shocks may be experienced, as in water pipes, floors, floor beams, arches, etc., medium steel should be used and if high stresses are permitted in the concrete, the bars should be deformed. In retaining walls, tank walls, sewers, etc., high carbon steel may be used. As a rule the twisted, high carbon steel now advertised by many firms, will cost as much per pound as plain bars of medium steel and be much cheaper than deformed bars of medium steel. It can be usually shipped more promptly.

Per Cent of Steel.

Reinforced concrete design has not yet reached the point where formulas can be used without judgment. Seven-tenths per cent of steel, as compared with one per cent, does not always mean just what it seems.

Take for example, a beam to resist a bending moment of 100,000-inch pounds. We know that n=12, so in Table I. we find under 12 that k=.33 when 0.7 per cent of steel is used, and k=.38 when 1.0 per cent of steel is used.

It is assumed that the concrete cannot be stressed more than 500 pounds per square inch, so in Table II. we find in the column headed 500, that opposite 0.007 the fibre stress in the steel is about 12,000 pounds, corresponding to an elastic limit of 36,000 pounds, if we are held to one-third the elastic limit. One per cent of steel means a little less than 10,000 pounds or practically an elastic limit of 30,000 pounds.

We look next in Table III under column headed 500, and opposite k=.33, we find K=73.4 and the moment arm is 89% of d. Opposite k=.38, we find K=83.0 and the moment arm is 87.3% of d.

Assume a width of 8 inches and our first beam is as follows:

$$d = \sqrt{\frac{100\ 000}{73.4 \times 8}} = 13.05''$$

Allowing for concrete to cover the steel, call the total depth 15 inches. 13.05x8=104.4″x.007=.73 square inches of steel. The nearest to this will be three ½″ square bars, weighing .85×3=2.55 pounds per foot of beam. Our beam 8″x15″ contains .833 cubic feet of concrete. At $6.00 per cubic yard this means 22 cents per cubic foot or 18.3 cents per lineal foot. At 2 cents per pound the steel will cost 5 cents, making a total cost for the beam of 23.3 cents per lineal foot.

The second beam is

$$d = \sqrt{\frac{100\ 000}{83 \times 8}} = 12.2''$$

Allowing for the same depth of concrete to cover the steel, we get a beam 8x14″. It should really be 14.15″, but we will keep to the nearest half inch. By calculations similar to those just completed, we find the cost of this beam to be 23.9 cents per lineal foot. The difference is a trifle less than three per cent, due to the fact that the smaller percentage of steel goes with a larger beam.

Factor of Safety.

The statement that steel having a high elastic limit gives a better factor of safety than steel having a low elastic limit, perhaps need some explanation, since we have explained the difference between modulus of elasticity and elastic limit.

Turn to Table I. with n=12 take k=.38, corresponding to one per cent of steel. From Table III under concrete stressed 500 pounds K=83.0.

Under 600 pounds, K=83.4 and k=.31.

Under 700 pounds, K=83.1 and k=.26.

As we have taken n=12, turn again to Table I and opposite k=.31, p=0.6%. Opposite k=26, p=0.4% (about).

This value of n limits us in using Tables II and III.

Under f=10,000, c=500, k=.38, p=0.95%.

Under f=15,000, c=600, k=.31, p=0.62%.

Under f=16,000, c=600, k=.31, p=0.58%.

Under f=18,000, c=700, k=.26, p=0.5%.

Under f=18,000, c=700, k=.26, p=0.5%.

Under f=20,000, c=700, k=.26, p=0.4%.

This shows how the different percentages of steel in a beam of a specified size alter the unit stresses in both concrete and steel when a definite value is given to n, the ratio of extensibility.

If a beam is designed for a certain percentage of steel having a low elastic limit and the state of the steel market and the time limit on the job compel the use of high carbon steel, the use of an equal amount of high carbon steel insures a stronger beam.

If high carbon steel is used and the designer decides to keep to the same factor of safety, thus using less of the high carbon steel, the beam will not be so stiff as with the larger amount of steel. When a concrete beam bends innumerable cracks open in the bottom. They may be microscopic, but if in depth they extend to the steel, moisture may enter and corrode it.

Where values of c, f and n are not fixed by ordinance, the designer can exercise his judgment to secure economy, care being taken not to lean too far toward the danger line.

Therefore it is best to pay no attention to the ultimate strength of the steel but keep within a factor of safety of three, or one-third the elastic limit.

The following table gives the ultimate strength we should consider for different concrete mixtures, and the safe fibre stress should be one-fourth the ultimate strength, to correspond to one-third the elastic limit of the steel. The values given are averages for concrete 30 days old and good workmanship will put the strength above the average. Concrete grows stronger with age and the greatest strains come on beams and floors generally about one month after the concrete is poured.

ROCK CONCRETE.

1:2:4	2,450	pounds per square inch in compression.
1:2:5	2,400	" " " " " "
1:3:5	2,000	" " " " " "
1:3:6	1,900	" " " " " "
1:3:8	1,800	" " " " " "
1:4:8	1,500	" " " " " "

CINDER CONCRETE.

1 : 2 : 4	900	pounds per square inch in compression.
1 : 2 : 5	700	" " " " " "
1 : 3 : 6	500	" " " " " "

The modulus of elasticity, with usual loads, at end of 30 days is about as follows:

ROCK CONCRETE.

1:2:4	2,600,000	pounds.
1:2:5	2,450,000	"
1:3:5	2,400,000	"
1:3:6	2,300,000	"
1:3:8	2,100,000	"
1:4:8	2,000,000	"

Cinder concrete has a modulus of elasticity of about 1,040,-000 pounds.

For the above values of rock concrete n will be from 12 to 15 and for cinder concrete it will be from 25 to 30. Some men use 35 to 40 for cinder concrete.

Rock concrete weighs from 140 to 145 pounds per cubic foot, and for estimating purposes with the steel in it, the weight is generally assumed at from 145 to 150 pounds. Cinder concrete is generally assumed as weighing from 110 to 115 pounds per cubic foot reinforced.

The designer should endeavor to get just the right amount of steel in the beam. Too much steel means increased expense and also danger that the beam will be destroyed by the concrete crushing out at the top. This does not give warning enough. If the amount of steel is small the beam will deflect considerably before the concrete is over-stressed.

The foregoing formulas place the neutral axis in the position it occupies just before the beam fails, when the steel is stressed to the yield point, the elastic limit. As the formulas given are termed "Straight line" formulas, the actual position of the neutral axis is a trifle below this point, so there is an additional factor of safety up to a load which implies that the fibre stresses calculated have been reached. For ultimate loads we must use formulas based on the parabolic distribution of stress, and such formulas are not discussed in this book, as we deal only with working loads.

The only drawback to the straight line formula in the opinion of some designers, is that the amount of steel is too small for the ultimate load that can be borne by the concrete. For the most economic designing a knowledge of the parabolic theory is desirable, but for all practical work, and for designing under the limitations imposed by building ordinances, the straight line theory is perfectly satisfactory.

To obtain the fibre stresses in a beam already designed the preceding pages may be consulted. For the benefit, however, of men not used to transposing equations the following formulas are given. Representing the moment arm by $d' = d - x$ (expressed as a decimal, representing a percentage of d, precisely as k is represented as a percentage of d), then

$T =$ total tension in the steel in pounds.

$C =$ total compression in the concrete in pounds.

$M =$ bending moment to be resisted, in inch pounds.

then

$$T = C = \frac{M}{d'd}$$

The material deficient in amount determines the actual strength of the beam, so to find M as fixed by the steel,

$$M = fpd'dbd^2$$

in which d'd represents the moment arm in inches, and $p = \frac{A}{bd}$ the percentage of steel, bd being the area of the concrete above the center of the steel.

The strength of the beam as fixed by the concrete is

$$M = \tfrac{1}{2}c\ k\ d'd\ bd^2.$$

The fibre stress in the steel is $f = \frac{T}{A}$, where A is the area of the steel in square inches. Another formula is given under Table III.

The fibre stress in the concrete is

$$c = \frac{2fp}{k}$$

Other Formulas.

From the results of experiments in which the above formulas have been demonstrated to be correct, another set of formulas have been derived.

There are three stages in the testing to destruction of a reinforced concrete beam. Until the steel has been stressed to about one-third the yield point the neutral axis is stationary. It then rises until the steel is stressed to about one-half the yield point, after which it remains nearly stationary in position until the beam fails, when it drops.

Some men believe it is perfectly safe to design for safe loads and assume the neutral axis to be in the middle of the beam and ignore the relative extensibilities of the two materials.

Let c=stress in extreme horizontal layer of concrete per square inch.

f=fibre stress in steel.

b=breadth of beam.

d=depth from top of beam to center of steel.

x=concrete modulus.

d′=moment arm.
p=percentage of steel

$$c\frac{bd}{4} = x\ bd$$

$$\frac{x}{f} = p$$

$$M = x\ b\ d \times d'\ d = K\ bd^2.$$

Take the concrete at 500 pounds, then

$$500\ \frac{bd}{4} = 125\ bd.$$

Take f=10,000 lbs., then $\frac{125}{10{,}000}=0.0125=p.$

Assuming the neutral axis at the middle of the beam and the center of compressive forces in the concrete at one-sixth of the depth of the beam we have $\frac{5}{6}$ d=0.833d=d′, the moment arm, and 125 b d×0.833d=104b d^2=M.

Recognizing that the modulus of elasticity and the elastic limit are different factors having different influences on the strength of the beam, and that the position of the neutral axis in the middle of the beam is true only when percentage of steel is from 1.0 to 1.25 per cent, the following modification has been proposed to the above formula:

This value of K=104, is maintained as a basis, but p varies with the fibre stress in the steel= $\frac{x}{f}$

The value of K obtained as above, is maintained for p=1.0% and over. When p is less than 1.0% take $0.50+\left(\frac{0.833-.50}{p}\right)=d''$, which varies as may be seen, with p. Then

$$xbd \times d''d \times p = Kbd^2.$$

Example: Use p = 0.8% then

$$0.50+\left(\frac{0.833-0.50}{0.8}\right)=.916d=d''d$$

$$125\ bd \times 0.916d \times o.8 = 91.6bd^2.$$

The depth of the neutral axis from the top of the beam will then be k = 0.27 + 0.18 p.

By the above method tables of K may be calculated and used. The only excuse for such a formula is that it is more readily remembered than the more exact ones, but this excusevanishes when it requires modification to permit of more rational designing with varying percentages of steel.

Such a formula is useful at times when some piece of work is to be done and table books and abstruse formulas are not at hand. The rationale is simple, hence easily remembered and applied.

For all practical purposes it is first-class practice to consider the neutral axis as occupying a position in the middle of the beam and use 1.25% of steel; making the depth of the beam from 1-10 to 1-12 the span and having the breadth between ½ and ¾ the depth. It may not be the most economic beam but it will certainly be safe if shown to be able to resist the bending moment caused by the loading.

Beam Failures.

When a beam breaks by showing large cracks near the middle at the bottom and an appearance of crushing at the top, the cause generally is failure by tension in the steel. In such a case not enough steel is used for the quality of concrete obtained.

When the failure is by breaking of the concrete on top, then too much steel has been used. The beam fails by compression in the concrete.

Beams failing by showing diagonal cracks at the bottom near the supports fail by diagonal tension in the concrete.

When the width of a beam is 1/20 to 1/24 the span and is at least one-half the depth, the beam will likely fail by tension in the steel or compression in the concrete. When the beam is short and deep, it may fail by diagonal tension or the bond may be destroyed between the steel and concrete.

Let V=vertical shear on the beam.

o=circumference or periphery of bar in inches.

m=number of bars.

u=bond per square inch.

d′=moment arm.

then $u=\frac{V}{mod'}$

As the adhesion is expressed in pounds per sq. in. mo= total surface of bars per sq. in. of length.

The periphery of a half inch bar is $4 \times ½ = 2$ inches and the periphery of a one-inch bar is 4 inches. Four half-inch bars have the tensile strength of one bar one inch square but have

$4 \times 2 = 8$ inches of bond surface, or double that of the one-inch bar.

Building ordinances generally specify the bond or adhesion so the amount should be tested by the above formula. If the amount of steel has been found sufficient for bending moment but not sufficient for bond, it is plain that smaller bars may be used in order to get the increase of surface necessary.

The stresses tending to destroy the bond between the concrete and the steel are transmitted to the surrounding concrete. This gives a horizontal unit shearing stress we will call v. In value it is equal to

$$v = \frac{V}{bd'}$$

where b = breadth of beam. This v is also equal to the vertical unit shearing stress at a point just above the level of the reinforcing bars.

The formula $vb = \frac{V}{d'}$ gives the rate of vertical stress per unit of length of beam that will go into stirrups if they are found necessary.

To illustrate: Having found the bending moment, calculate the size of the beam and amount of steel necessary to resist it. Next determine the number of rods required, remembering the space between the rods should be at least equal to the thickness. Next test for bond. When this is settled test for the horizontal and vertical unit shearing stress and if it exceeds the tensile strength of the concrete, stirrups must be provided. Stirrups as a rule are necessary only in deep short beams.

By the above formula stirrups will be placed the entire length of the beam. Some designers use stirrups in every beam while others simply use them when the analysis indicates the necessity for them. Even when an analysis may show them to be really not necessary it adds considerably to the strength of the beam to provide them and the cost is not great. A sheet of expanded metal or wire fabric in the web of the beam is excellent and performs the work of stirrups.

Stirrups should be either fastened to the bottom steel or should be bent in U shape with the bottom rods going through this loop. They may be inclined at an angle of 45 degrees toward the ends of the beam, or they may be vertical. Inclined stirrups are most efficient, but they are troublesome to place.

An empirical rule given by E. L. Ransome for stirrups is to place four at each end of the beam, the first to be ¼ the depth from the end, the second to be ½ the depth from the first, the third to be ¾ the depth from the second, and the fourth to be a distance equal to the depth from the third. These stirrups are generally ¼ to ⅜-inch rods. In small beams they may be of extremely heavy wire, one stirrup hooking around each bottom reinforcing rod.

All stirrups should go to within one inch of the top of the slab on the beam and then run about six inches into the slab. In any case they should go to within about an inch of the top of the concrete and be bent to run parallel with it for about six inches. They should be used even when the reinforcement is bent upward toward the ends of the beam.

Beams having some of the reinforcing bars bent up near the ends are stronger than beams having all the bars straight. This is because of better bond, or is due to a change in direction of the stresses. Each designer seems to be at present a law unto himself as to manner of bending the bars. The writer turns up one-fourth of the rods at the quarter span point, one-third of the remainder at the sixth point and one-fourth of the remainder at the eighth point. The remainder go straight to the support and when a little past it are turned straight up at least six inches.

The bent rods go at an angle of 45 degrees to within one inch of the top. If at the end of a simple beam they bend at the top and go clear over to the support and are anchored by bending. If the beam is continuous, or fastened at the end (as common with reinforced concrete beams), the bent rods stop at the top of the beam. In the top of the beam across the supports are placed one-half as many bars as are used in the bottom. These top bars go across to the quarter span points and are there turned down at an angle of 45 degrees and terminate one inch from the bottom.

It is common now to have brackets connecting beams to walls and heavy pillow blocks under beams at posts. Diagonal tie bars go through these projections which thus act as braces and stiffen the structure in case of unequal settlement of foundation or in case of earthquake.

When bars lap past each other they should lap at least twenty-five times the thickness. No bar should be of a size that its length in a beam will be less than fifty times the thickness or diameter.

When the reinforcing steel is stressed more than 10,000 lbs. per sq. in. deformed bars should be used. Twisted bars are good enough for all practical purposes, but all the bars in the market are good, although extravagant claims are made for some.

The important fact must be remembered that it makes no difference in what form the reinforcement comes. The amount to use for strength can be determined by the formulas already given and no matter what the claims of the advertiser no smaller amounts can be used without lowering the value of the factor of safety. This remark applies to the strength of the steel to resist breaking.

Deformed bars and rods increase the hold of the concrete and thus enable the designer to utilize more of the strength of the steel than if adhesion alone is depended on. As a general rule at present prices the most economical beam contains steel stressed about 16,000 lbs. per sq. in., with the concrete stressed about 570 lbs. per sq. in. The writer would not think it safe to stress plain steel to exceed 10,000 lbs. per sq. in., so in using a higher stress he employs deformed bars or rods.

TABLE IV.

Weights per lineal foot and areas of square bars and round rods.

One cubic foot of steel weighing 490 lbs.

One cubic inch of steel weighing 0.283 lb.

Thickness or Diameter.	Weight lb. (Square.)	Area, in. (Square.)	Weight, lb. (Round.)	Area, in. (Round.)
¼ inch	.212	.0625	.167	.0491
⅜ "	.478	.1406	.376	.1104
½ "	.85	.25	.668	.1963
⅝ "	1.328	.3906	1.043	.3068
¾ "	1.913	5625	1.502	.4418
⅞ "	2.603	.7656	2.044	.6013
1 "	3.4	1.00	2.67	.7854

Table IV will be of assistance in using ordinary plain and twisted steel. The manufacturers of special reinforcing material gladly give to all inquirers pamphlets and circulars containing tables of sizes and weights of their materials.

Fabrics, whether woven, welded or expanded from sheets, so bind the concrete that the stress is distributed over a wider area than when bars are used, but not enough to safely reduce the amount of steel. The advantage claimed for fabrics is that they should cost less to put in place than it costs to put in rods or bars properly. When beams and slabs are designed the steel is assumed to occupy a definite position. To insure this all pieces should be wired at intersections. This costs a great deal and the makers of fabrics claim that by the use of their material the cost of placing steel is lessened and the certainty of getting it in the correct position assured.

Double Reinforced Beams.

There is seldom any occasion to design a beam reinforced both in the top and bottom, but occasionally it is unavoidable. Steel used, however, in the compression side of a beam is expensive, for it cannot be stressed as high as steel in the tension side.

When the bottom of the beam is stressed we know that innumerable small cracks open that can seldom be detected even with a microscope, for they are distributed because of the adhesion to the steel. For this reason we neglect the tensile strength of the concrete, as the cracks open before the steel is stressed more than three or four thousand pounds per square inch.

In the upper part of the beam we cannot exceed a certain fixed fibre stress in the concrete, so can only stress the steel in proportion to the ratio of the moduli of elasticity of the two materials. Consequently while the fibre stress in the tension steel may be as high as 16,000 pounds per square inch, the compression steel cannot be stressed to exceed 6,000 to 9,000 pounds per square inch, depending upon its depth below the top of the beam.

M. Bonna placed steel in both sides of all beams he designed and his formulas are as follows:

Let A=area in square inches of tension steel.
A′=area in square inches of compression steel.
M=bending moment in inch pounds.
d=moment arm between centers of steel reinforcement in bottom and top of beam.

Then $A=\frac{M}{fd}$, and $A'=\frac{2}{3}A$.

The compressive strength of the concrete is depended upon for a certain amount of resistance and it is not necessary to make any calculations for the position of the neutral axis. To the moment arm, d, it is simply necessary to add enough to thoroughly cover and protect the steel in each side of the beam. This total depth, therefore, will depend upon the head room wanted and the consequent depth of beam that can be permitted.

The bending moment being obtained from the span and loading and f being assumed, according to the grade of steel used,

$$\frac{M}{f} = Ad.$$

Usually, however, the problem comes to us in the form of a beam that has already been designed and which cannot be enlarged because the plans have proceeded to such a point that alterations cannot be made, yet some new ideas have arisen which call for a much heavier loading on the beam. The only way to take care of this load is to proportion the steel in the bottom to carry it, thus increasing the fibre stress on the concrete.

If the building ordinance will not allow the higher stress in the concrete, then enough steel must be placed in the compression side of the beam to balance the additional amount of steel in the tension side, without changing the position of the neutral axis as originally designed, and thereby keeping the concrete fibre stress down to where it was originally calculated.

In order that the steel and the concrete will deform equally,

$$f = nc,$$

and this cannot be exceeded.

If this ratio is used, however, it would imply that the compression steel will be placed in the top of the beam, for c is the extreme fibre stress, decreasing to 0 at the neutral axis. As the compression steel will be placed below the top of the beam, where the stress will be less than c, and in amount $= c'$ we then have, calling the compression steel stress $= f'$,

$$f' = nc'.$$

The moment arm of the beam as originally designed extended from the plane of the tension steel to the centroid of compression of the concrete. When the compression steel is added there is another moment arm added which is equal in length to the distance between the planes of the tension and compression steel. The length of this moment arm is chosen arbitrarily.

The area of the original tension steel is A and the additional steel added to that side of the beam we will call a, while the area of the compression steel will be A'. Then to find the area,

$$A' = \frac{af}{nc'}$$

That is, we multiply the fibre stress in the tension steel by the additional area of the tension steel and divide by the fibre stress of the concrete in the plane of the compression steel, multiplied by the ratio between the moduli of elasticity, in order to obtain the area of the compression steel.

The area of the compression steel being governed by its depth below the top of the beam, if a floor slab rests on the beam and is to be poured at the same time, a calculation can be made to increase the depth of the beam by considering the thickness of the floor slab as a part of it. Placing the compression steel then in the floor slab will result in some saving of steel, by materially lengthening the moment arm between the steel reinforcements.

To prevent any tendency to buckle on the part of the compression steel, small stirrups should be hung over it at intervals not exceeding twelve times the thickness of the bars or rods used, and these anchoring stirrups should go clear to the bottom steel.

The internal stresses in the beam should be carefully computed when double reinforcement is used and it may be found advisable to make the reinforcement in the top and bottom of angle iron connected by a sheet of expanded metal, or of rods fastened together by expanded metal or wire fabric, thus making the beam practically a plate girder.

T Beams.

Some designers have a great fondness for designing what are known as T beams, in which the floor slab above the beam is intended to carry all compression and the beam consequently is simply a concrete stem wide enough to carry the steel.

Many theories have been proposed for such beams, but the latest and most satisfactory are those of Prof. Talbot, in which the amount of steel is a percentage of a beam having a width equal to the width of the slab in the T section, and a depth found by the usual methods. That is, the formulas already given will do for T beams as well as for ordinary beams, but in determining the amount of steel it must be a percentage of such a large area that the writer has found no resulting economy in using such beams, especially when taken in connection with the objections mentioned in a later chapter concerning the construction work.

For any reader who may wish to investigate such beams it may be stated that the width of the slab varies from three to five times the thickness of the stem. Some men consider the width of the slab as being the clear span, thus calling on the floor for compressive strength half way on each side to the next support. T beams with wide flanges lack in stiffness.

The position of the neutral axis when determined should be such that the thickness of the flange will not be less than ⅓ k, so that the neutral axis may well be in the flange if desired. When the amount of steel is determined it should be in as large bars as possible in order to have the rib narrow, but the bond should be carefully investigated and the steel have enough peripheral area to furnish bond. Web stresses also are high and should be carefully determined, and stirrups or other web reinforcement provided. Careful attention should be paid that cross bearing steel is placed in the slab across the stem, in girders, or cross bearing beams. In the calculations the rib is not considered and its thickness depends upon the size of the steel. At its junction with the floor slab a fillet should be placed to prevent cracks because of the sharp angle.

Common values assigned to concrete and steel by ordinances are as follows:

Steel.—Fibre stress not to exceed one-third the elastic limit in tension.

Concrete.—Tension no value. Compression, 500 lbs. per sq. in. for beams, 350 lbs. per sq. in. for columns.

Shear.—50 lbs. per sq. in. (as internal stress).

Adhesion to steel.—75 lbs. per sq. in. of surface of steel.

General rules for lengths of reinforcing bars and rods have been given. The following table gives the matter in a little more complete form. When a certain fibre stress is assumed for the steel and a certain value given per square inch for adhesion, we obtain the length of steel necessary for bond by ascertaining the area of the cross section of the steel, multiplying this by the fibre stress and dividing the result by the adhesion per square inch, multiplied by the circumference or perimeter of the rod or bar.

TABLE V.

Minimum lengths (minimum clear span of beam) required to secure rods and bars against slipping under stated fibre stresses in the steel. Adhesion 75 lbs. per sq. inch of surface.

Diameter or Thickness	Adhesion per lineal inch		Values of f.			
			10,000	16,000	18,000	20,000
	Bars	Rods	Lengths			
¼ in.	75.0	59.3	1 ft. 5 in.	2 ft. 3 in.	2 ft. 7 in.	2 ft. 10 in.
⅜ in.	112.5	88.5	2 ft. 3 in.	3 ft. 4 in.	3 ft. 9 in.	4 ft. 2 in.
½ in.	150.0	117.5	2 ft. 7 in.	4 ft. 7 in.	5 ft. 2 in.	5 ft. 8 in.
⅝ in.	187.5	147.0	3 ft. 7 in.	5 ft. 8 in.	6 ft. 4 in.	7 ft. 1 in.
¾ in.	225.0	177.0	4 ft. 2 in.	6 ft. 8 in.	7 ft. 6 in.	8 ft. 4 in.
⅞ in.	262.5	206.0	4 ft. 11 in.	7 ft. 10 in.	8 ft. 10 in.	9 ft. 10 in.
1 in.	300.0	236.0	5 ft. 7 in.	8 ft. 11 in.	10 ft. 1 in.	11 ft. 2 in.

This gives half the length of the steel in the beam (for there are two, opposite and equal, pulls) and multiplied by two gives the minimum clear span to be used in connection with the rods or bars. The table shows this, and when a beam has been figured, the size of the steel can be taken readily from this table in order to be safe as respects bond. One-half the lengths given in the table will be sufficient to run the steel into walls or other supports or to lap pieces past each other. Of course, as great a length can be used as the designer wishes, but the lengths given are minimum. When lapping steel reinforcement, the pieces should not be spliced together with wire so they touch throughout the whole length, but should preferably be so spliced that a space be left between equal to their thickness or diameter. This is best accomplished by bending each piece so the pull will be in a direct line and putting small spacers at intervals of a few inches. The connection of these lapping pieces, however, should be strong and certain in order that the concrete be not unduly stressed to give the required adhesion. The spacers should be first-class metal clamps, similar to those used for cables. The ideal lap is one where each end is so bent that when fastened in place the shape will be a loop similar to a turnbuckle, with strong clamps at the end connections and several spacers across the loop. Another good connection is a screwed connection. It is really the best if properly made. It should always be used in columns.

The writer is a firm believer in reinforcing all beams on top across supports. Unless means are especially provided to localize the stresses, the top of all beams will exhibit more or less cantilever action, for the connection causes it. In fact, it would do no harm to design all beams in two ways: First, as a plain beam, not allowing for contraflexure; second, as two cantilever beams, joined in the middle of the span. Place reinforcement in the beam accordingly, turning it down or up for web reinforcement when it passes the point where it is required for bending moment in the top or bottom. Such a beam will be safe, no matter how loaded, and if a fire consumes the contents of the room below and should happen to destroy the concrete protecting the under steel, the steel in the top of the beam will carry the load. The writer believes too many chances have been taken with reinforced concrete designing in the past, and too many chances are taken today.

Floor Slabs.

Owing to the monolithic character of reinforced concrete work there is a continuous action at supports, but it is bad practice to lessen the amount of bending moment developed at the center of beams or girders because of it. Negative stresses set up at supports is all we can provide for and this has been touched upon. Therefore, we should take care of the bending moment of a simply supported beam at the center by the formula, $M = \frac{wl^2}{8}$ and the negative bending moments at the supports by placing enough steel in the top to take care of the bending moment as given by the formula, $M = \frac{wl^2}{12}$.

With floor slabs, however, it is usual to compute the bending moment by the formula, $M = \frac{wl^2}{10}$, although some designers use 12 instead of 10, in which formula w is the load per square foot and l the span in inches, the moment, M, being in inch pounds. The span is always the shortest span when the floor panel is not square. If the panel is square the formula is $M = \frac{wl^2}{20}$ and the reinforcement runs both ways. That is, when this formula is used it is figured that half the load goes to two sides and the other half to the other two sides. Then two layers of steel will be used for reinforcement, at right angles to each other. To the thickness obtained by the formula, $d = \sqrt{\frac{M}{Kb}}$ must be added sufficient concrete to protect the steel underneath and also the extra thickness caused by the second layer of steel. This additional concrete to allow for the two layers of steel is often overlooked by designers, with the result that the moment arm is shortened and the slab deflects unduly.

Mr. Robert B. Hansell, C. E., Baltimore, Md., has asked the writer to insert the following table in order to make it clear that in the case of a uniformly loaded beam supported at several equidistant points the portion of the load bearing on each support is as follows:

TABLE VI.

Number of supports.	Number of each support.					
	1st.	2d.	3d.	4th.	5th.	6th.
2	½	½.	.....	.....	.....	.....
3	⅜	10/8	⅜	.....	.....	.....
4	4/10	11/10	11/10	4/10	.....	.....
5	11/28	32/28	26/28	32/28	11/28	.....
6	15/38	43/38	37/38	37/38	43/38	15/38

It is seldom that more than three supports will occur in practice, but the above table will be useful in helping distribute the load to all the beams under floor and roof slabs. The load per square foot on the slab will be the assumed floor loading, and the load per lineal foot on the beams and girders under the slab will be determined by the reactions shown in the above table.

To space the rods equally both ways in a square slab is not correct, for if we consider a 12″ strip as the unit and the floor consists of a number of 12″ strips side by side we know that the diagram for bending moment will be a parabola. The steel should therefore be closer together in the middle running both ways. To determine this draw the beam and the parabola with the vertex as M. Then dividing the beam into a number of 12″ sections determine the bending moment on each. Reinforce each for its bending moment. While this is a little labor and there may be some difficulty in securing the right spacing of the steel it is more correct than the usual method. When a floor slab freely supported at all four sides, designed for a bending moment expressed by $\frac{wl^2}{8}$ develops so much strength that the use of 10 as a divisor seems to fit better than 8, part of the reason is in the excess of steel in the outer edges of the slab.

In this connection see end of Chapter II.

CHAPTER I.—(Continued.)

Parabolic Theory of Stress.

It has been mentioned in the preceding pages that for economy in design the formulas there given do not give the best results. In fact for the ideal beam the cost of the steel should exactly balance the cost of the concrete above the center of the steel. Certain obvious considerations prevent the attainment of this ideal but by using what is known as the Parabolic Theory of Stress, it can be more closely approximated than by the straight line theory given under Fig. 2 on page 12.

When we study the stress strain curve of a reinforced concrete beam it is easily seen that the material does not act so that the unit deformation in any horizontal fiber varies directly as its distance from the neutral axis. In fact the tensile strength of the concrete is gone by the time the steel is stressed up to about one-fourth the usual working stress, and this is shown by innumerable small cracks in the bottom under the steel. By the time the steel is stressed to the elastic limit it is believed the cracks extend, almost, if not quite, to the neutral axis.

Neglecting therefore the tensile strength of the concrete and concentrating the entire tensile stress in the line of steel near the bottom the straight line theory for the steel is correct, so that no matter what formula may be used to determine the resisting moment as determined by the concrete, the resisting moment as determined by the steel is always

$$M = f\ p\ d'\ b\ d^2 = f\ A\ d'd$$

which is the formula given on page 22, which unfortunately contains a typographical error.

By testing reinforced concrete beams to destruction and plot-

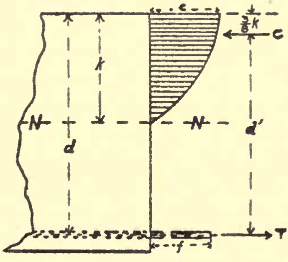


FIG. 3A.

ting the results we obtain a stress strain curve for the concrete in compression closely approximating a parabola. The agreement is so remarkable that the parabola has been chosen as representing more nearly the actual stress strain curve than the triangle. This is shown in Fig. 3A.

To illustrate the difference this makes, the following formulas are used to give the exact stresses in the beam.

The average abscissa of a triangle equals one-half the greatest but the average abscissa of a parabola equals two-thirds the greatest so that the equation on page 12 becomes, for the parabolic formula

$$fp = 2/3\ ck$$

The formula for determining the position of the neutral axis in the straight line formula is given on page 13, and this becomes, for the parabolic formula,

$$k = \sqrt{3pn + (\tfrac{3}{2}pn)^2} - \tfrac{3}{2}pn$$

In a triangle the center of gravity is one-third the height from the base, which gives k/3, as shown on page 13. In a parabola, however, the distance is 3/8, so that, for the parabolic formula,

$$d' = 1 - 3/8\ k$$

It is easy to remember the parabolic formulas for ultimate loads by simply substituting 3/8k for 1/3k and by substituting 2/3 for 1/2 when dealing with the stresses in concrete. The moment of resistance for the beam as determined by the concrete is

$$M = 2/3\ k\ d'd\ bd^2$$

Another very important difference must be pointed out. The material deficient in area determines the strength of the beam. If the steel is deficient in area the parabolic formulas for ultimate loads cannot be used for they are based upon the possibility of utilizing the full strength of the concrete. It is only when the steel "balances" the concrete that the resisting moment for the steel gives the exact resisting moment for the beam. With parabolic formulas we should not use less than one per cent of steel and some beams require nearly two per cent. With straight line formulas we can use as small a per cent of steel as desired. This statement, of course, must be modified by the character of concrete used, for a very small per cent of steel will develop the full strength of some concretes.

Using straight line formulas we secure rigidity and a certain excess of concrete. The advantage is that when a beam is underreinforced (has too small an area of steel to balance the concrete) it gives warning long before it breaks. A "balanced" beam goes suddenly. This partly accounts for the general use of straight line formulas in building laws. Another reason they are greatly used is that the formulas in general use up to within a year or two were empirical and in this form, so that the majority of designers used them. Another reason is that until within the same time few engineers or architects could procure simply written books on reinforced concrete design. The greater part of the information was given free by companies selling steel. To prevent the charge of trying to sell too much steel they all used straight line formulas and advocated small percentages of reinforcement. Now, however, that it is generally known "the more steel the less concrete" and that the concrete costs about three times as much as the steel in a beam, the steel companies are generally using parabolic formulas.

With straight line formulas we are limited to the use of working stresses in both steel and concrete with vague ideas as to the actual factor of safety. With parabolic formulas the beam is supposed to contain enough steel to develop the full strength of the concrete by the time the steel stress equals the elastic limit.

The straight line theory is therefore known as the Fiber Stress Method and the parabolic theory is known as the Factor of Safety Method. As we have here two materials differing in many respects, joined together in a structure the factor of safety method seems most logical to use for economic design. Therefore, instead of designing to obtain a resisting moment equal to the bending moment, with safe fiber stresses in the two materials, the bending moment produced by the load should be multiplied by the desired factor of safety to get the ultimate bending moment. Then instead of a safe fiber stress in the concrete the actual or assumed, ultimate strength is used. Instead of a safe fiber stress in the steel the elastic limit is used. The resisting moment obtained is equal to the ultimate bending moment.

To secure the maximum economy in design when a test load is prescribed some ambitious designers assume a load equal to the test load plus the estimated dead load of the beam or slab. To obtain the resisting moment they assume a strength for the concrete of about two-thirds the actual or assumed strength. For the steel they assume a stress about five per cent less than the elastic limit. This insures the beam against failure under the test load. It would hardly do to require the test load to stress the concrete to the ultimate and the steel to the elastic limit or beyond, yet it is too often done in competitive designing.

Straight line formulas give a certain stress in the concrete under load. The stress is actually about fifteen per cent less when we calculate it by parabolic formulas. Thus there is really a larger factor of safety under working loads than is apparent.

A table like Table I can be calculated for parabolic formulas by using the formula for k already given; that is,

$$k = \sqrt{3\,pn + (\tfrac{3}{2}\,pn)^2} - \tfrac{3}{2}\,pn$$

To calculate a table like Table II to ascertain the value of "k" in connection with unit stresses, use the formula

$$k = \frac{3pf}{2c}$$

in which

k = depth to neutral axis;
f = elastic limit of steel;
c = ultimate strength of concrete;
p = ratio of steel to concrete.

A table of moment factors "K," like Table III can be calculated by the formula

$$K = 2/3c\ d'\ k$$

which is similar to the formula given on page 16, using 2/3 instead of 1/2.

Analysis of Beams and Slabs.

When a beam or slab shows signs of failure and it is possible to ascertain the area of the reinforcement, parabolic formulas are used to analyse the failure if the steel is shown to be in excess of or balances the concrete. If the steel is confessedly inadequate then use straight line formulas. We do not care about the actual stress in the material in excess.

Instead of straight line formulas, however, the flexure formulas of Professor Talbot may be used. They are parabolic also and we can use them practically as straight line formulas but instead of taking a definite fiber stress, a fraction "q" is used.

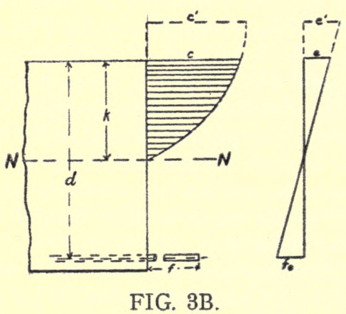


FIG. 3B.

In Fig. 3B the diagram on the left shows c′ which is the ultimate strength of the concrete in compression and "c" the strength the concrete reaches at the time the strength of the steel (the elastic limit) is reached. Instead of assuming the strength of the steel at the elastic limit any assumed strength can be given to it, thus approximating the safe fiber stress if desired. The diagram on the right is the deformation diagram in which "e′" is the total deformation in the concrete at the time of failure, and "e" the deformation reached when the steel has reached the strength assumed. At the bottom of the diagram "f" is the steel stress and "f_e" the deformation in the steel. The fraction "q" stands for $\frac{e}{e'}$. For example, when "q" = 1/4 the concrete is strained to one-fourth its limit of compression, etc.

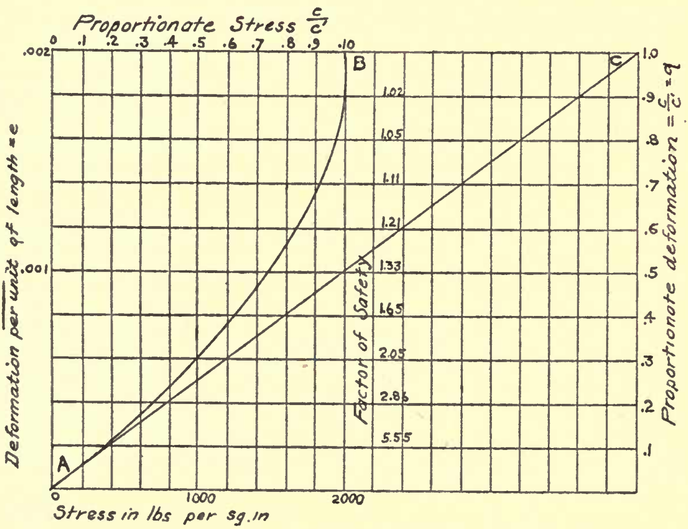


FIG. 3C.

Fig. 3C is a diagram taken from the bulletins from the Illinois Experiment Station containing the reports of tests made under the direction of Professor Talbot, with some additional notes.

These flexure formulas are rather unwieldly and are therefore a method of analysis rather than for designing. By using them to construct diagrams they can be of great service but for every day work the straight line formulas are all right for partial loads, below the ultimate.

The idea in view in developing these flexure formulas for loads below the ultimate is to keep the stress in the steel below the yield point (elastic limit) and yet ascertain the real stress in the concrete. By assuming different values for "q" the actual stress in the concrete for any assumed stress in the steel can be found, irrespective of the percentage, or area, of reinforcement. Thus it may be seen that the factor of safety method and the fiber stress method are practically combined.

To show how the foregoing simple formulas are complicated by the introduction into them of "q" the expression on page 12, becomes

$$fp = \frac{kc\,(3 - q)}{3\,(2 - q)}$$

and that at the bottom of page 13 becomes

$$k = \sqrt{2 - \frac{3\,pn}{3 - q} + \left(\frac{3\,pn}{3 - q}\right)^2} - \frac{3\,pn}{3 - q}$$

Stress Strain (Stress Deformation) Curve.

In analytical mathematical work every line described by its relation to an origin, that is by ordinates and abscissas, is termed a curve, even when it is a straight line.

Stress is a force applied. Strain is a deformation resulting from that stress. Deformation is amount of change of form or shape.

On a sheet of squared paper let the vertical graduations upward from zero, represent the deformation and the horizontal graduations from left to right, represent the stress. When points are plotted showing the deformation for any stress and all the points are joined we have a stress strain curve. As already mentioned the stress strain curve for the concrete in compression in a reinforced concrete beam approximates a parabola in form.

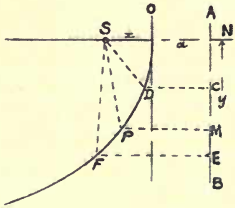


FIG. 3D.

Fig. 3D shows a parabola, which is a curve such that any point on the curve is equi-distant from a given point and a given straight line. The given point, S, is called the focus and the given straight line, A—B, the directrix.

$$CD = DS;\ MP = PS;\ EF = FS.$$

The point O is called the vertex and when the directrix touches the vertex, O, the equation for the parabola is

$$y^2 = 4x$$
$$y = \pm 2\sqrt{x}$$

The general equation for the parabola with any other position of the directrix, is

$$y^2 = 4ax$$
$$y = \pm 2\sqrt{ax}$$

In Fig. 3C, the position in which the parabola is drawn is that in which it is generally drawn when plotted as a stress strain curve of the concrete in compression. The stress is represented by "x" and the deformation by "y". In this diagram for convenience the ultimate strength of the concrete is assumed at 2000 lbs. per sq. in. and the deformation per unit $= 0.002 = e$.

Ratio of Deformation.

Over the modulus of elasticity much discussion has been waged and as concrete has no true modulus of elasticity the ratio known as "the ratio between the moduli of elasticity of the concrete and steel" should really be termed "the ratio of deformation." No one objects to the statement that some ratio of deformation exists but decided objection is made to that ratio being termed the ratio between moduli of elasticity when one of the materials has no such modulus.

This ratio of deformation is expressed in terms of a tangent to the parabola representing the stress strain curve of the concrete in compression. The stress is the tangent of an angle between the deformation and the tangent to the parabola; essentially the co-tangent.

When the concrete commences to deform the stress strain curve is almost straight but gradually changes in shape, becoming more nearly vertical until when the limit of deformation is reached it has assumed a parabolic form as shown in Fig. 3C.

The line representing the initial modulus of elasticity is represented by the line A—C and is tangent to the parabola at its origin. This origin is not that usually assumed as the origin of the parabola but is the origin of the stress strain curve of compression in the concrete.

In Fig. 3E let x and y represent respectively the abscissa and ordinate of the parabola. PT represents the tangent at the point where intersected by y, and a is the angle of the tangent. dx is an infinitesimal increment of x and dy is an infinitesimal increment of y.

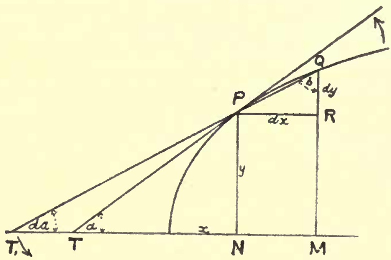


FIG. 3E.

Plotting them as shown we obtain the triangle P, Q, R in which to find the angle P = da. Then dy/dx gives the tangent of the angle P.

Conceive the quantities dx and dy as being so small that the line P Q extended to T, will assume the position P T, swinging as indicated by the arrows.

At this point $\frac{y}{\sqrt{x}}$ which represents the slope of the curve, being coincident with dy/dx, which represents the slope of the line, gives the following equation for the tangent of the parabola

$$\text{Tan } a = y/2x$$

The angle b is one used to express the value of E_c. In Fig. 3D it is marked A and the angle for which we have just found the equation for the tangent, is marked C.

$$\text{Let } \frac{2x}{y} = \text{Tan b} \qquad \text{Tan b} = E_c$$

$$x = c \qquad \text{Then } \frac{2c}{e} = E_c$$

$$y = e \qquad c = \tfrac{1}{2} E_c e$$

When different values of n are used they of course represent the E_c obtained from different points of tangency but instead of the line being tangent to the parabola at the point fixed by the selected concrete fiber stress, it is parallel to said tangent and starts from the point of origin of the stress strain curve.

As a matter of fact the line forming the side of an angle, the tangent of which represents the modulus of elasticity of the concrete cannot be tangent to the parabola at a point where the concrete stress = 0. It must be a tangent at some definite value of the stress in the concrete.

CHAPTER II.

Loads on Beams.

This chapter is intended as a review for the reader and an aid for quick reference when some useful and necessary formula is forgotten.

A moment is the product of a force multiplied by the distance at which it acts. For example, a load suspended from the extreme end of a cantilever beam causes it to bend more than if placed near the fast end. That is, the bending moments vary with the distance from the load to the support. The bending moment is opposed by a resisting moment in the beam.

When the resisting moment is less than the maximum bending moment the beam fails. When one is equal to the other the beam has no factor of safety. The factor of safety is represented by the fraction $\frac{Rm}{M}$, where

$M =$ maximum bending moment.

$Rm =$ resisting moment of beam.

When a beam is designed to carry a load four things are to be considered:

1st. Reactions. This is the name given to the proportion of the load carried by each support. The name is given because action causes reaction and it is convenient to assume that there is an upward push at the supports to balance the downward push of the beam with its load.

2d. Bending moment.

3d. Deflection. While a beam may be strong enough to carry a certain load it may deflect, or bend, enough to crack plastering. Consequently there are cases where deflection is of more importance than absolute strength. Formulas for deflection will not be here considered in connection with reinforced concrete beams. If the design calls for a stress in the concrete not exceeding 600 lbs. per sq. in. and not exceeding 16,000 lbs. per sq. in. in the steel the deflection will not exceed 1/360 of the distance between supports. This is a limit beyond which it is hardly wise to go if the under side of the beam is to be plastered.

4th. Shear. This is a term applied to the action tending to cut the beam vertically and is caused by the downward push of the load resisted by the upward push of the supports. 1-2-4 concrete can safely stand 300 lbs. per sq. in. of such action.

What is termed shear in a beam of reinforced concrete is really diagonal tension. When a beam bends the bottom fibres stretch and the upper fibres are compressed. At the neutral axis there is a tendency to shear, and this grows less toward the top and bottom of the beam, where one set of stresses, of course, overcomes the effect of the other set. Shear in this case should not exceed 60 lbs. per sq. in.

Having, then, two sets of forces acting at continually changing angles with each other, we have resultants to these forces acting at right angle to the tangent of the curve formed. The resultant acts as a tensile stress in the concrete and stirrups are placed at right angles to it.

This explains why stirrups should be fastened in some way to the bottom rods, or hooked under them, and why they should go farther than the top of the beam; for as put in many beams, the stirrups are too short for bond. See Chap. I.

Let $W =$ total load in pounds, uniformly distributed, including the weight of the beam $= wl + w'l$.

$w =$ load per lineal foot on beam.

$w' =$ weight per lineal foot of beam.

$P =$ concentrated load on beam.

$B =$ total weight of beam $= w'l$.

$l =$ length in inches of beam. This is usually assumed as equal to the clear width between supports plus a bearing at each end, as it gives a better margin of safety than when the clear span alone is used.

$M =$ maximum bending moment in inch pounds.

Case A. Beam supported at both ends (freely resting on supports) and loaded uniformly.

$M = \frac{Wl}{8}$, at middle of beam.

Maximum shear at points of support $= \frac{W}{2}$.

When the shear acts upward on the left side of a section, or downward on the right side, it is termed positive. The reverse case gives negative shear. As all beams are subjected to positive and negative shear there is a point where the sign changes, and this is where the maximum bending moment occurs.

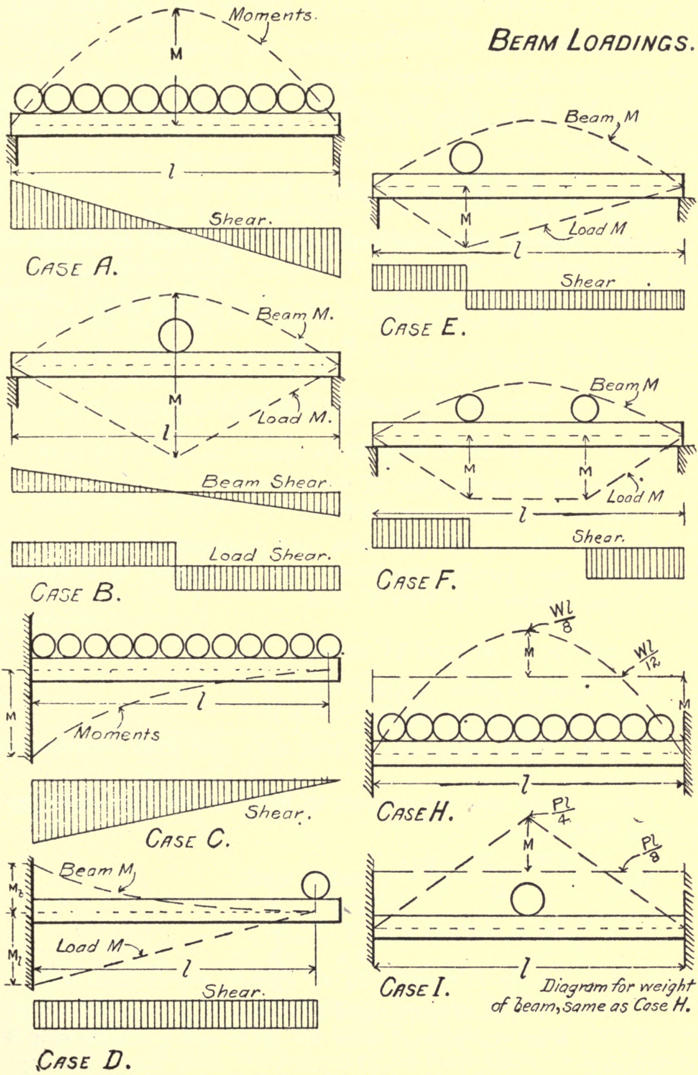


FIG. 4—BEAM LOADINGS.

On the left half, the shear is positive and on the right half negatime, and 0 at middle of beam.

Reaction at each support $= \frac{W}{2}$.

Case B. Beam supported at both ends with load concentrated at the middle.

$M = \frac{Pl}{4} + \frac{Bl}{8}$, at middle of beam.

Max. shear at points of support $= \frac{P + B}{2}$ and at middle of beam $= 0$.

In this case (B) the shear at all points on the beam $= \frac{P+B}{2}$

Reaction at each support $= \frac{P + B}{2}$.

Case C. Cantilever beam uniformly loaded.

$M = \frac{Wl}{2}$ at point of support.

Maximum shear at point of support $= W$.

Reaction at point of support $= W$.

Case D. Cantilever beam with load concentrated at any point.

In this case $l =$ distance from point of support to a vertical line through center of gravity of load.

$M = Pl + \frac{Bl}{2}$ at point of support.

Max. shear at point of support $= P + B$.

Reaction at point of support $= P + B$.

Case E. Beam supported at both ends with load concentrated at any point.

Call distance from left support to load, a, and from right support to load, b.

$M = \frac{a\,(2\,Pb + Bl - Ba)}{2l}$, under load.

Max. shear at support $a = \frac{Pb}{l} + \frac{B}{2}$.

" " " " $b = \frac{Pa}{l} + \frac{B}{2}$.

Reaction at each support equal to shear.

At this point the general rules for shear and reactions may be introduced.

The weight of the beam being a distributed load, one-half goes to each support in addition to the proportionate part of the loads on the beam. In shear one-half is plus (the left) and one-half is minus (the right).

Rule for reactions for combinations of loads.—Multiply each load by its distance from one support. Add the products and divide the sum by the span. This gives the reaction on the opposite support. The other reaction is obtained by subtracting the reaction thus found from the sum of the weights of the loads. The sum of the reactions is always equal to the sum of the loads.

Rule for shear for combination of loads.—The maximum shear will equal the greater reaction. The shear under each load is found by setting down either reaction with its plus or minus sign and adding, algebraically, to it successively the weights of the loads, commencing with the one nearest the reaction chosen.

Case F. Beam supported at both ends with two loads equally distant from the ends.

Call the distance from each end, a.

$M = Pa + \frac{Bl}{8}$ at middle of beam.

Max. shear at points of support $= \frac{2P + B}{2}$.

Reaction at each support $= P + \frac{B}{2}$.

Case G. Beam supported at both ends with several concentrated loads.

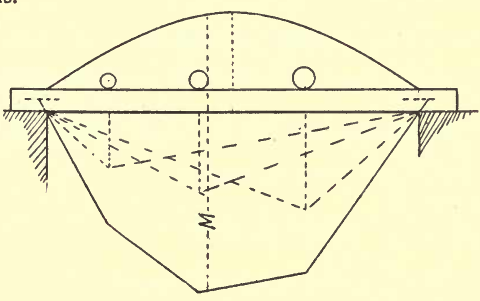


FIG. 5—CASE G.

The most simple method for this case is to make a scale drawing, showing by a single horizontal line the length of the beam. Calculate each M as shown for case E. Draw vertical lines downward under each load, the lengths of the lines under each load representing the bending moment caused by the load at that point. Connect the lower ends of these lines to the ends of the beam. This gives a number of triangles equal to the number of loads, and each vertical line will be divided into the same number of sections.

Add the lengths of these sections together to obtain the bending moment at each point. In other words, the total bending moment at any point produced by the weights of all the loads is equal to the sum of the moments at that point produced by each of the weights separately.

Under each load extend the vertical line until it is equal in length to the bending moment at that point caused by all the loads. Connect the ends of these lines and the ends of the beam. Above the beam draw a parabola with an extreme height $=\frac{Bl}{8}$. The position and amount of the maximum bending moment will be found by scaling the longest possible vertical line intercepted by the parabola above and the irregular figure below the beam.

Case H. Beam fixed at both ends and loaded uniformly.

A beam supported at the ends bends downward when loaded and is concave on top. When fixed at the ends it is convex on top at each end and concave in the middle. When uniformly loaded the points of contraflexure are 0.2113 the length from each support.

$M=\frac{Wl}{12}$ at points of support.

$M=\frac{Wl}{24}$ at middle of beam.

Max. shear $=\frac{W}{2}$ at points of support.

Case I. Beam fixed at both ends with concentrated load in middle.

Points of contraflexure=0.25 the length from each support.

$M=\frac{Pl}{8}+\frac{Bl}{12}$ at points of support.

$M=\frac{Pl}{8}+\frac{Bl}{24}$ at middle of beam.

Max. shear $=\frac{P+B}{2}$ at points of support.

Parallel Forces on Simple Beams.

All the forces acting on a beam may be determined graphically and thus the foregoing formulas proven. Combinations can be worked out without the labor of going through the calculations and the work will prove itself.

Let A B represent a beam resting on supports as shown by the arrows pointing upward at C and D, these arrows representing the reactions. At the left the line A′ B′ is the sum of all the weights drawn to some scale, each weight being represented by the amount between the figures, e. g., from A′ to 1 is force 1; from 1 to 2 is force 2, etc. The line T O is perpendicular to the line A′ B′ and is of any length, generally taken at some even number of hundreds or thousands of pounds, depending upon the scale to which A′ B′ is drawn. From O draw lines to 1, 2, 3, etc., and to the ends. With triangles or parallel ruler transfer these lines to form the polygon a, 1, 2, 3, 4, b, under the beam. Connect the points a and b and transfer the line so it will be represented in direction by the line O S.

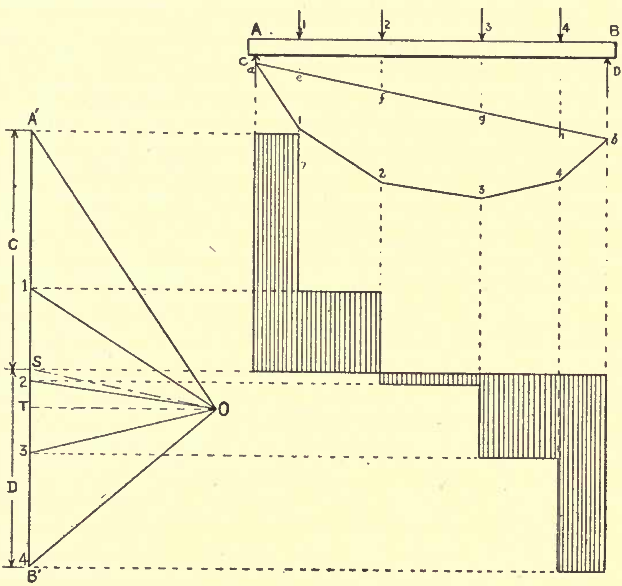


Fig. 6—Parallel Forces on Beam.

Then the reaction under A is shown by C = A′ S, and the reaction under B is shown by D, = B′ S, to scale.

The bending moment at any point is equal to the length of OT multiplied by the ordinate of the polygon at that point. For example, the bending moment at 2 is equal to OT multiplied by f2. If our scale is in hundreds of pounds and the length OT is equal to one unit of the scale, then the ordinate is equal to the bending moment.

Shear is obtained by the third diagram with the shaded rectangles. The construction of this figure needs little explanation. Simply produce the horizontal and vertical lines as shown and hatch the rectangles thus formed. The shear at any point on the beam is equal to the length of the longest vertical line at that point. The diagram shows that the point of maximum bending point is the point where the shear equates to zero.

To consider the weight of the beam, which is uniformly distributed, it will be well to make a separate diagram and afterward take the sums of the moments, shears and reactions, caused by this distributed load and the concentrated loads shown by the arrows. This is shown in other figures.

In calculating a beam, the load per lineal foot is used as explained in the formulas. In calculating a floor slab, consider it as a number of beams 12 inches wide lying side by side. It is thus necessary to figure only one 12-inch beam in order to ascertain thickness of slab and amount of steel. Retaining walls may be calculated as a series of horizontal beams, one on top of another, or as a series of vertical beams standing edge to edge on end. To secure proper intervals in slabs, it is customary to have reinforcement at right angle to the principal reinforcement, to which the latter will be connected by wiring at intersections. This crossing steel is generally assumed as being equal to one-third of 1 per cent of the area and spaced accordingly. Where the extremes of temperature are not great, even less than this amount may be used. Bars of the same size are used, which would mean that when the reinforcement in the line of the beam is 1 per cent, the bars are spaced six inches apart; then the crossing bars will be 18 inches apart. Nothing is settled about this point yet, and two considerations enter into the question. One is temperature and the necessity for some provision to avoid temperature cracks. The other is shear in the concrete between adjacent beams of which the floor is assumed to be composed.

The following loads are those allowed in different cities of the United States, in addition to the dead load. The load of the structure is termed the dead load. The load imposed on the floor by the materials stored there is termed the floor loading and is often spoken of as the live load. Strictly speaking, however, a live load is a moving load and has double the effect of the same weight in pounds of a stationary load. For example, a live load of 200 lbs. per sq. ft. might be calculated as if it were a steady load of 400 lbs. for a bridge but as 200 lbs. in a building.

Floor loads. Dwellings, apartment houses, hotels, tenement houses, or lodging houses, 70 lbs. per sq. ft.; office buildings, first floor, 150 lbs. per sq. ft.; above the first floor, 100 lbs.; schools and places of instruction, 80 lbs.; stables or carriage houses, 80 lbs.; building for public assembly, 150 lbs.; ordinary stores, light manufacturing and light storage, 120 lbs.; stores for heavy materials, warehouses and factories, 150 to 250 lbs.; roofs, pitch less than 20 deg., 50 lbs.; pitch more than 20 deg., 30 lbs.

Live load on slabs to be used for highway purposes should be about 100 lbs. per sq. ft. (equal to dead load of 200 lbs.) and the fibre stress in the concrete should not exceed 400 lbs. per sq. in. with $n=15$. For spans not exceeding 50 feet a highway bridge can be calculated in the same manner as given for floor slabs. Sometimes considerable economy can be effected by calculating two beams about 18 inches thick, to carry the load, and using them as parapets. The floor slab across will be connected to these beams a little below the neutral axis, but this, of course, depends upon the head room required. Such parapets should have double reinforcement and be designed that way. The steel in the floor slab should be turned up into the beam far enough to provide for hanging (reaction). Abutments must be rigid and go to a good foundation. See bulletin No. 15, "Concrete Bridges," American Association of Portland Cement Manufacturers, Land Title building, Philadelphia, Pa.

Sidewalks in Chicago are calculated for 300 lbs. per sq. ft., including weight of slab; in St. Louis and some other cities, for 300 lbs. per sq. ft. in addition to weight of slab.

CHAPTER III.

COLUMNS.

Columns of reinforced concrete are used because they cost less in place than columns of steel or iron. They are also used because the majority of men who design in reinforced concrete like to use as much of the material as possible. To them it seems a profanation to use a column of steel or iron and have the walls and floors of concrete.

Concrete columns occupy more space than columns of other materials commonly used for such purposes. Taking for example a load of fifty tons to be supported by a column 18 feet high. the sizes are as follows:

Reinforced concrete	18 x 18 inches
White pine or spruce	13 x 13 inches
Oak	12 x 12 inches
Yellow pine	11 x 11 inches
Cast iron (hollow and round)..............	8 in. diameter
Steel (2 6-inch latticed channels)..........	6 x 8 inches

In a floor having twenty columns the space occupied by round cast iron columns will be less than six square feet. The space occupied by the same number of steel columns will be less than eight feet, while the space occupied by the same number of reinforced concrete columns will be over forty square feet. When space is rented by the square foot, or, rather, the value of space is based on its rental value, the economy of reinforced concrete columns is sometimes questionable. In situations where the space occupied is of little consequence and where durability is the chief thing sought, the reinforced concrete column has a place. Even in such a place, it may not be as good as columns of other materials when the cutting off of the light is also considered.

Two methods are used for reinforcing columns. In one all the reinforcement is vertical. It is tied at intervals or is wrapped with wire, but such tying or wrapping is empirical and is more to keep the reinforcement in place than for any other purpose. The other method is to have simply enough vertical rods to tie the wrappings to and to reinforce by spiral wrappings of wire or steel rods.

There are two modifications that are hardly entitled to be called reinforced concrete, but should rather be styled "concrete protected" columns. In one the column must be of steel sufficient in size to carry the load and with concrete to protect it and add the necessary stiffness and factor of safety. In the other the reinforcement is built as a column large enough to carry all the dead loads and the construction loads. The concrete, when added, makes it strong enough to take care of all live loads. This last form is coming rapidly into use, together with built-up reinforcement for beams. It admits of certainty in placing the reinforcement, while at the same time lessening the time occupied in construction. In the opinion of the writer, the building ordinances in all cities will gradually insist upon such construction.

When columns are reinforced with vertical rods, the few experiments made do not show results that are entirely satisfactory. They do show, however, that this method is good if the concrete is well proportioned, well mixed and thoroughly compacted when placed. The steel should be as straight as possible and rest upon bed plates at the bottom. The total load should not exceed 350 or 500 pounds per square inch.

Generally building ordinances require that reinforced concrete columns have a ratio not exceeding $\frac{l}{d} = 12$. In other words, the least thickness or diameter will be in inches equal to the clear height in feet. It is plain that, as the steel rods carry part of the load, they might bend enough to destroy the concrete. In a long column, flexure might be caused by a heavy load, although in the tests made up to date no bending seemed to have been developed in columns having a ratio of less than 25. Some designers use a ratio of 20 with $c = 350$ and a ratio of 12 with $c = 500$.

Columns reinforced by spiral wrappings can be loaded up to about 1,000 pounds per square inch, because the concrete is encased in the wrappings as in a steel cylinder. As the load comes upon it the tendency is to bulge outward. This, being resisted by the spirals, enables greater strength to be developed than by the use of longitudinal bars or rods alone.

A spirally wound column is not very stiff, and if such reinforcement is used in long columns it must be assisted by longitudinal rods. The place for these rods, of course, is inside the spiral. One objection to the use of spirally reinforced columns is that there must be some settlement before the bulging is sufficient to call into play the strength in the spirals. Within a fibre stress of 1,000 pounds per square inch, if the column is well made, this need not cause anxiety. The writer, however, prefers to use lower stresses and longitudinal reinforcement.

Recent experiments seem to indicate that when vertical rods are used in spirally wound columns they are an element of danger. As already stated, the ultimate strength of the column is increased, but there is nothing gained when loads are light, as they are generally. That is, if the steel is used in the form of spiral wrappings it will generally be found to be expensive reinforcement unless high stresses are used. When some of it is in the form of longitudinal rods and the stress is so high that the column settles enough to call the spiral wrapping into service, then the steel is apt to be stripped from the longitudinal rods, thereby dividing the body of the column. When this is done and each rod is free to act alone, it buckles and the column is rapidly destroyed. Therefore when a column is wound there should be as little vertical steel as possible.

A volume of steel in the form of a large wire or small round rods wrapped spirally has 2.4 times the effect that the same volume of steel would have if disposed in vertical reinforcement. That is, provided it is wrapped with a pitch of from one-sixth to one-seventh the diameter.

A knowledge of this fact permits one of two alternatives. The column size may be retained and a smaller amount of steel used than if vertical reinforcement is adopted, or the size of the column may be reduced by using a stress in the concrete 2.4 times greater than the stress ordinarily considered safe. This means that we can use in a spirally wound column a stress of 840 pounds per square inch instead of the 350 pounds used in a vertically reinforced column, or we may use 1,200 pounds per square inch instead of 500 pounds. The placing of vertical reinforcement is cheaper and it is easier to get the reinforcement right than to wind the steel. Compilers of conservative building ordinances so far do not look kindly upon spirally wound columns except when they are fabricated in shops and brought to the building. Neither do they like high stresses.

The tables of column divisors are used as follows: Assume a value of $\frac{l}{d}$ and thus get the area of the cross section of the column. Perhaps the building ordinance fixes this ratio, in which case use it and obtain the area. Divide the weight in pounds to be carried by the column by the area. Under the value of "n" fixed by the ordinance, find the column divisor. On the same horizontal line will be found the percentage of steel.

If a certain percentage of steel is assumed and a value of "n" selected, the column divisor in the "n" column on the same line as the steel percentage will be used as a divisor of the weight in pounds. The result will be the area of the column in square inches.

The tables, as may be seen, have been calculated with $c = 350$ and $= 500$ pounds per square inch, with values of "n" ranging from 8 to 20 and with from 1 to 10 per cent of steel.

TABLES OF COLUMN DIVISORS

for longitudinally reinforced-concrete columns.

Table VII. c = 350 lbs. per sq. in.

Per cent of steel	n = 8	10	12	15	20
	f = 2800	3500	4200	5250	7000
	—DIVISORS—				
1	374.5	381.5	388.5	399	416.3
2	399	413	427	448	483
3	**423.5**	**444.5**	**465.5**	497	549.5
4	448	476	504	**546**	**616**
5	472.5	507.5	542.5	595	682.5
6	497	539	581	644	749
10	595	665	735	840	1015

Table VIII. c = 500 lbs. per sq. in.

Per cent of steel	n = 8	10	12	15	20
	f = 4000	5000	6000	7500	10000
	—DIVISORS—				
1	535	545	555	570	595
2	570	590	610	640	690
3	605	635	665	710	785
4	**640**	680	720	780	880
5	675	**725**	**775**	850	975
6	710	770	830	**920**	1070
10	850	950	1050	1200	**1450**

While the percentages of steel are given as though no limit is considered, there is a practical limit depending on the cost of the steel and the ease of getting it in the form of rods and bars as compared with getting structural shapes and using concrete protected columns instead of reinforced concrete columns. The heavy lines are drawn where the same amount of steel in a steel column would carry the entire load without the concrete.

The tables were calculated by the following formula:

$P = c\ (A_c + nA_s)$, in which

P = total load in pounds or compressive strength of section.

c = compressive load per square inch permitted on concrete.

$\left.\begin{array}{l} A_s = \text{area of steel} \\ A_c = \text{area of concrete} \end{array}\right\}$ in cross section.

n = ratio of moduli of elasticity.

To understand and use this formula, assume a unit value for steel and concrete, when it becomes

$P = c\ (1 + n1)$,

and by multiplying the quantities in the parenthesis by an assumed value of c (for example, 350) and of n (for example, 12), we have

$P = 350\ (1 + 12\ (1)\ ) = 350 + 12 \times 350.$

The following example illustrates the method of calculation, taking the values already assigned and starting with a 1 per cent reinforcement of steel:

$350 \times 12 \times 0.01$ (the per cent of steel)............ =	42.0
350×0.99 (the per cent of concrete)............. =	346.5
Column divisor	388.5

The total load in pounds diivded by 388.5 will give the area in square inches of the column section for a reinforcement of 1 per cent of steel, when "c" is 350 and "n" is 12.

The factor "n" is very important in column formulas. Professor Talbot says it is apt to be misleading to use it, and to term it the ratio between the moduli of elasticity is indefinite and undesirable. He therefore uses a formula having a different ratio, as follows:

$P = A_c\ (1 + (n - 1)\ p)$, in which

P = total load in pounds or compressive strength of section.

A = total area of section (including concrete and steel) in square inches.

c = compressive load per square inch allowed on the concrete.

n = ratio of stress in steel to stress in concrete.

p = percentage of steel or ratio which area of steel bears to the area of the concrete.

The tables of column divisors obtained by the first formula were checked by the second.

The ratio of the moduli of elasticity has been considered of great importance in column design, assuming the adhesion or bond between steel and concrete to be sufficient. When a load comes upon the column, part is supposed to be taken by the concrete and part by the steel. Each must, therefore, be compressed precisely in the ratio of the relative extensibility. If they do not, then the adhesion will be destroyed, after which each material will act independently, making certain the destruction of the column. When the adhesion is destroyed, each rod becomes a long and extremely slender column.

To call "n" the ratio between the moduli of elasticity is not correct. The term "ratio of stress in steel to stress in concrete" gives a clearer idea. The ratio is sometimes as high as 20, but the safest value to use when a factor of safety of from 4 to 5 is wanted in the concrete, lies between 12 and 15.

Calling A the total area of the column section (including the steel) and "p" the percentage of reinforcement, the area of the steel will be

$$pA$$

the unit stress in the steel will be

$$nc$$

and the area of the concrete will be

$$A\ (1-p)$$

The total compressive stress in the steel will be

$$pAnc$$

and in the concrete will be

$$A_c\,(1-p)$$

How much strength does the longitudinal steel add to the concrete? The formula:

$$P = A_c\ (1 + (n-1)\ p)$$

indicates that each addition of 1 per cent of steel adds (n—1) % of strength to the concrete.

The same formula enables us to find when the steel in the usual structural shapes will carry the load without the concrete.

Let $s =$ the ordinary stress used in steel columns. Usually 16,000 pounds per square inch.

$A' =$ area of steel in steel column (cross section).

$pA =$ area of steel reinforcement in reinforced concrete column

Then $A_c\ (1 + (n-1)\ p) = sA'$

$$pA = sA'$$

$$p = \frac{c}{f - c\ (n-1)}$$

That is, the safe fibre stress allowed in the concrete is divided by the usual fibre stress in steel structural work minus the safe concrete stress multiplied by $(n-1)$, to get the percentage of steel that will carry the load without the concrete, provided it is in the usual structural form.

It is then simply a question of the balancing of costs. If the concrete column containing reinforcement can be built at less cost, or if the steel can be obtained in the form of rods and bars within less time than it can be obtained in shapes that can be worked into the usual form of steel column, then the reinforced column may be best.

The steel should be so disposed in the column that it will be protected by at least one inch of concrete, and so the concrete can flow readily between the bars. Large bars should be used in preference to small. They may be so placed that the column may be square, round or octagonal in section. The writer believes it is well to allow a part of the area for fire protection of the steel. In the examples given at the beginning of this chapter the area of a reinforced concrete column is given at 18″ x 18″ = 324 square inches. Using the tables above, the exact dimension will be found nearer 16 inches square, and the additional size is given for the thorough protection of the column.

In this connection it may be stated that the logical position for the steel is in the center of the column, but it should be nearer the outside, because the concrete should be in as large a body as possible and be not too much cut up. The writer believes that the body of concrete inside the steel should make a column of clear concrete having a ratio of $\frac{l}{d}$ equal to three-fourths the ratio of the whole column. That is, if calculations show the column should be 16 inches square, the steel can be arranged to fit inside a 12-inch square. This rule is wholly empirical, and by some designers may not be considered good. It should not be adhered to if it results in an interior section of less than 8 x 8 inches, and does not need to be followed closely if a larger inside square can be obtained for the steel and still leave a thickness of at least one inch of concrete protection. In columns liable to exposure to fire it will be a safe precaution to increase the dimensions of the column by at least 2 inches.

When the reinforcement calls for four large bars, a good plan is to use four angles of equivalent area and make of them a latticed column. Otherwise use square bars wrapped with No. 8 or No. 10 plain wire, tying it with No. 16 or No. 18 wire at each intersection, merely to preserve the pitch, which should be equal to half the thickness or diameter. The wrapping should be commenced at one corner and go around in as long strands as possible. When spliced, it should be by hooking the ends and making long twisted joints. Four rods generally suffice, except when their size would render them difficult to handle, when a larger number of smaller rods can be used.

Owing to the shrinkage stresses developed in concrete, which may stress the steel unduly before the load comes on the column, it is good practice to use plain bars or rods in columns and pour about five diameters in height at a time, leaving an interval of about two or three hours between pourings. The columns should be completed before the pouring is commenced for the floors and beams they support. Concrete in columns usually settles considerably in setting, and for this reason it is well to allow about three or four hours' time to elapse before pouring the beams connecting to the top of the columns.

This rule for pouring columns does not always work well, for some engineers have told the writer that they noticed small cracks that appeared in the concrete below when it was set rather hard but not quite hard enough to bear the weight of the fresher concrete poured above. It is quite likely they waited too long between pourings. The idea of having a little time elapse is to permit of thorough settlement in small masses and to allow a somewhat uniform setting thereby.

When vertical rods used in columns are shorter than the column, pieces put on to lengthen them should rest on top instead of being wired side by side, and the joint should be made in a sleeve of pipe just fitting the steel. Alongside each joint and about four times as long as the sleeves, which should be twenty-four diameters long, should be set a half-inch bar as a joint stiffener. It should not be wired to the column reinforcement, but should be a separate piece, entirely surrounded by concrete.

Mr. W. A. Hoyt, C. E., structural engineer with the Corn Products Company, informs the writer that he does not set his vertical rods until ready to go on with the columns. It often happens in construction work that the column footings are completed long before the columns are poured. The long steel rods are a problem to handle. Some men place in the footings short sections of pipe and set the steel in the pipe when ready to go up with the work. In spite of all care, more or less dirt will collect in the pipes. Mr. Hoyt uses instead short sections of steel which project a foot or two above the footing. When ready to go ahead a sleeve is placed around them and the vertical rods are set in this sleeve.

CHAPTER IV.

Walls, Tanks and Footings.

Retaining walls fail by sliding forward, by overturning, or by breaking across at a point approximately one-third of the height up from the bottom.

Until the advent of reinforced concrete all walls were designed to have enough bulk to prevent failing by any of the above ways. It was realized that if masonry could be made with enough cohesion to stand bending, walls could be sunk far enough in the earth so they would not slide forward; that they could be anchored in some way to prevent overturning, but that all this involved the production of strains that would cause the wall to break. Therefore, the strong masonry was needed. This is apart from the fight between the men who designed walls by theoretical formulas and those who stood faithfully by empirical formulas, or, as some called them, "rule-of-thumb methods."

There was never any reason for the latter fight, for walls that were designed according to theory, coupled with judgment, always stood. It was when a man's passion for pure theoretical reasoning overcame his better judgment that walls fell, except in the very few exceptional cases where the circumstances were such that even walls designed by empirical rules, supposed to embody the best judgment in exceptional cases, would have failed. Empirical rules simply set a thickness for a wall according to the height and practically independent of the character of the backing, except when a man liked to add a guess of his own.

When walls commenced to be built of reinforced concrete all the carefully treasured empirical rules as to weight and thickness of walls had to go; for whereas in the case of bulk the pressure was of little or no moment, so long as certain procedures were adhered to, when it came to the designing of a wall in which the minimum of material was the goal, pressures had to be taken into account. Reinforced concrete walls are in shape like a capital letter L or like an inverted capital T. The weight of the wall is small and the weight of the backing upon the rear leg has to be taken into consideration. Therefore, bending moments develop which cannot be withstood in old style masonry having joints, so are taken care of by the reinforcement.

Considering a vertical strip 12″ wide on a wall the total pressure is found by the formula:

$$P = y \times d^2$$

in which d is the depth in feet from the top of the wall to the point at which the pressure is wanted, and the material is no higher than the wall.

The following table gives values of y for different materials:

TABLE IX.

Water	y = 31.25
Fine dry sand	y = 15.7
Dry loose gravel	y = 12.1
Dry loose earth	y = 8.8
Moist earth	y = 5.6
Dense, natural earth	y = 6.2

Considering a horizontal strip 12″ wide; at any depth the pressure, w (or load), per square foot, is as follows.

$$w = 2y \times d$$

A surcharged wall is one that supports a sloping fill. The pressure on any foot will be

$$w = 3y \times (2d - 1)$$

and the total pressure will be

$$P = 1.5y \times d^2$$

The depth is the depth from the top of the wall as already explained, and while the formula is not exact, it is close enough for all practical purposes.

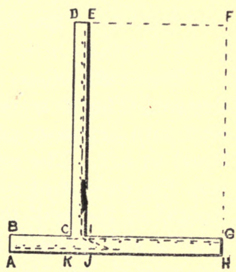


FIG. 7.

Reinforced concrete retaining walls may be designed as cantilevers, in which case the formula for P is used, or they may be designed with counterforts, in which case the formula for w is also used. Fig. 7 represents a wall which is a compromise between the L and the inverted T. In order that the system of lettering used may apply to both designs, DC or EI represent the height, which will hereafter be designated by h. The pressure P is applied at h/3, to obtain the bending moment at CI, which will determine the thickness at that point. The pressure is applied at a point one-third the height for the reason that the area of pressure is a triangle and forces act always through the center of gravity of bodies; the center of gravity of a triangle being one-third up from the base.

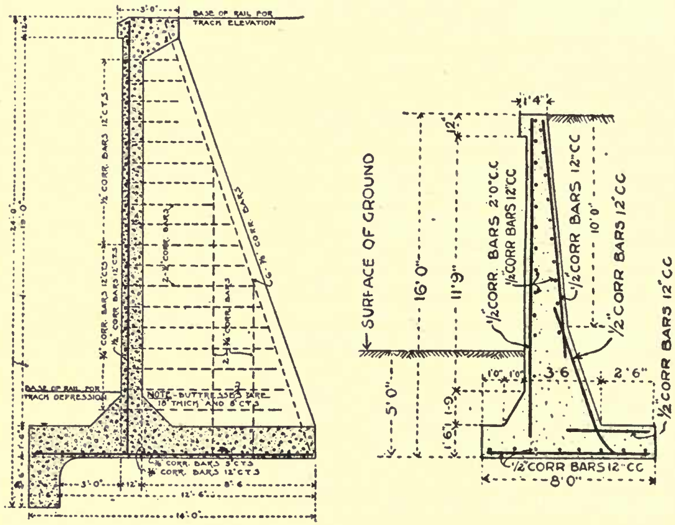


FIG. 8—TWO TYPES OF RETAINING WALLS.

Two calculations will always be made to obtain the economic design. The first calculation will not take the weight of the wall into account, and the base AH will be taken at a length equal to h/2. The area DFHK in square feet, multiplied by 100 lbs., the weight of one cubic foot of earth, will be taken as the weight of the wall to resist the pressure, P. Find the center of gravity and through it vertically pass a line to represent to scale the weight just found. Through it horizontally pass a line to scale representing the pressure. Complete the parallelogram and draw the resultant. The position of this resultant is of importance, for in order that the maximum pressure on the base be not greater than twice the average, and that there be no tension on the back side of the foundation, the distance from the resultant to the middle point of the base must not exceed 1/6 the base.

To find pressure on base:

Let $F =$ pressure on base in pounds per sq. ft. at A.

Let $W =$ weight in pounds of the wall and backing (area DFHK).

Let $b =$ length of base, AH.

$d =$ distance in feet from point of intersection of resultant with bottom of base, to the nearest extremity of the base.

Then $F = \frac{2W}{3d}$, when d is equal to or less than b/3.

And $F = \frac{4W}{b^2}(b - 1.5d)$, when d is equal to or greater than b/3.

Or the following general formula can be used:

$$F = \left(\frac{b - d}{b \times d}\right) W$$

This pressure, F, on the base is the maximum pressure which it is estimated can be borne by the earth on which the wall is built. If the foundation will not stand such a pressure the base may be lengthened or piles can be driven under the toe, AB.

The bending moment at CK will be practically equal to $M = \frac{3F}{4} \times BC \times .55BC$; when $BC = \frac{AH}{2}$, or for shorter toe

$$M = \left(F - \frac{BC}{2AH} F\right) \times BC \times .6BC$$

The bending moment at IJ is, $M = \frac{W \times IG}{2}$

As the pressure against the wall is at right angle to the surface pressed, it brings the resultant a little nearer the center if the pressed surface is sloping instead of vertical. This increases the stability and decreases the bending moment in the case of water, but makes practically no difference for earth.

The dotted lines indicate how the steel will be placed to take care of the bending moments developed. As the shearing stress on CK will be great the thickness should be made double what the bending moment would demand, when $BC < \frac{IG}{8}$ thus using the calculation for bending moment simply to obtain the steel area. The thickness at IJ should be sufficient to develop the bond strength of the steel rods in the wall. Before determining this, however, the thickness, IC, should be obtained, and the thickness of the wall at regular intervals, in order to obtain the correct shape, which will approximate a curve. Three points should be enough, and they may be connected by straight lines, the front of the wall being vertical, or if desired may have a slight batter. The last slope of the back towards the bottom may be such that the steel parallel with it may extend into the base far enough to develop bond, or at its connection with the base the angle may be divided and the steel given a different slope in order to secure length of embedment without unduly thickening the base.

The thickness, IJ, will be used to determine the amount of steel, but the actual thickness, as seen, depends upon the bond embedment of the steel in the wall. All the steel must be run far enough past the points of maximum moment to develop sufficient bond. (See Table V, Chapter I.).

The steel in the base will run from the front to the back (perpendicular to the length of the wall), and the steel in the wall will be vertical. All of it need not run to the top of the wall, but it may be reduced as shown by the calculations for bending moment. Longitudinal rods should be placed in the wall and in the base at regular intervals to take care of temperature stresses and to assist in preserving the intervals between the main reinforcing rods. The steel should be well wired together at all intersections.

Before figuring the steel, however, a second calculation should be made after the dimensions of the wall have been fixed as shown. The wall should be drawn to scale and a new center of gravity found, representing the compound section made up of the wall weighing 150 lbs. per cu. ft. and the earth filling on the slab weighing 100 lbs. per cu. ft. To allow of some reduction in size it may be best to use 120 lbs. for the concrete portion. With this calculation, which may result in some changes being made in sizes, the work can stop and the steel be calculated for the wall. The longitudinal steel should be equal to about one-third of one per cent in area of cross section. The edges, BA, GH and DE may be of any thickness sufficient to give adequate protection to the steel, the slope being gradual and thus effecting some saving in material.

As walls designed in this manner are more apt to be heaved by frost than the usual type of gravity wall, the bottom slab must be placed deep enough to avoid any danger of frost action, but as this is a case where moments enter in the following formula may be used, in which $f = $ depth below surface in feet.

$f = .0007\ F$, which gives a minimum value for the depth.

Reinforced concrete walls are often designed with counterforts back of them, tieing the face slab to the bottom slab in the rear. This is a simple wall to figure and easy to build, but on account of more form work, by reason of the counterforts, may be more expensive at times than the cantilever wall, although generally requiring less material.

First determine the spacing between the counterforts. Then design the vertical wall as a slab between the counterforts, with horizontal reinforcement. Calling the distance between counterforts, in inches, L; the bending moment on each horizontal strip 12″ wide will be:

$$M = \frac{wL^2}{8}$$

in which $w = 2y \times d$ (load per sq. ft. at depth, d.)

The total load on any horizontal strip will be wL, and one-half of this at each end is reaction, thus indicating how far the steel must run back into the counterfort for bond. In addition to this horizontal reinforcing steel there should be vertical steel of approximately one-third of one per cent area used to space the horizontal rods and going to the top of the wall. At the bottom it should turn gradually into the slab at the back, going into it to help tie the face and base together.

The rear slab may be lifted by the tendency of the wall to overturn because of the pressure. In this case w is equal to the weight of a column of earth 12″ square, with a height=h, at the back edge of the slab, but is zero at the wall. Using then this value for w, which is different for each 12″ strip parallel with the wall,

$$M = \frac{wL^2}{8}$$

for each strip, thus the reinforcement will be spaced at greater intervals nearer the wall and the slab should really be thinner. The thickness, however, is maintained for the benefit of the weight given and to furnish bond for the vertical rods and the rods from the toe. The total load on each parallel strip is wL, and one-half of this at each end is reaction, showing how far the steel must project up into the counterfort for bond.

The counterfort will have a width at the bottom equal to the base and at the top will run to the wall or may end in a beam along the top of the wall. Its thickness will depend almost wholly on the thickness required to protect the rods embedded in it. It is calculated as a cantilever having a load $= PL$ and the bending moment at the bottom is, $M = \frac{PLh}{3}$. With this value of M, calculate the steel required and make the counterfort wide enough to take it. This steel will run along the back edge of the counterfort and into the top of the wall, where it must be embedded for bond to take up $\frac{PL}{3}$. At the bottom the steel must be bent back into the base for anchorage and be embedded so it will stand a pull $= \frac{2\,PL}{3}$.

Usually it will be found that the comparative cost of plain and reinforced concrete retaining walls depends so largely upon local costs of materials and labor that, for heights less than sixteen feet the plain wall may be far cheaper. This is owing to the generally wider base of the reinforced wall which calls for more excavation, the cost of form work and of the steel and labor in placing steel, and extra labor involved in pouring thin walls.

For comparison use for the plain wall the common rule that the breadth of base will be one-third the height. With the gravity wall, the strength being in the weight and solidity, a very much cheaper concrete can be used than for a reinforced wall. Comparative estimates should be always made before deciding upon the type of wall, remembering also that with a gravity wall not reinforced, the liability of error in the computations is relatively small.

Restrained Walls.

Some walls are designed as restrained at the ends, of which an example was given above in designing the slabs between counterforts. Occasionally, however, a wall is designed that is restrained at the top and bottom, as, for example, foundation walls around basements, pressing against the basement floor and first floor, and tanks having floors and roofs into which to tie the walls.

In such cases P is found as before. The reaction at the top is $\frac{P}{3}$ and at the bottom is $\frac{2P}{3}$, thus showing how far the steel must run into the slabs, or beams, to which the wall is connected, for bond, or the pressure which a basement wall will exert against the floors at its top and bottom.

If the wall is connected to slabs at top and bottom, the reinforcement in those slabs must run into the wall far enough for bond, in addition to the wall reinforcement running into them. The steel, however, that ties the slabs to the wall must be in addition to the steel required in the slabs, as it is sufficiently stressed, by reason of it being reinforcement. Some designers do use the reinforcement steel for tying the walls, but when this is done it is dangerous.

Sometimes instead of a slab at the top, the tank is open and a beam is run along the upper edge, being designed for a uniformly distributed load $= \frac{P}{3}$. This gives a finish as a coping, the span of the beam being from one end of the tank to the other, or sometimes being designed as a continuous beam with intermediate supports determined by steel rods running to the opposite side, or by counterforts.

Sometimes the thickness of the tank wall is determined in advance and the reinforcement is horizontal. At intervals, determined by the strength of this wall, are run vertical beams, projecting each side of the wall, as pilasters. The strength of the vertical beams being ascertained, the resisting moment of the beam is equated to P for different lengths and at the points thus fixed steel rods will be used to tie the opposite sides of the tank together through these beams.

In a wall restrained at top and bottom, the maximum stress is 0.853h from the top, and, $M = \frac{Ph}{7.8}$

For a 12″ vertical strip on a wall designed to resist water pressure,

$$M = 4h^3 \text{ in inch lbs., when h is in inches.}$$

The wall will be of uniform thickness from top to bottom, the reinforcement being vertical with the usual horizontal bearing steel.

Circular Tanks.

For circular tanks the steel resists all the tension and the concrete serves only to protect the steel. The mixture should be fairly rich and well mixed so it will be dense and water tight. All the rods should be bent to circles and ends firmly fastened together. They should be either in the center of the concrete, or not to exceed twice the thickness of the steel from the outer face.

The pressure, w, on the side is the same as for the pressure against a vertical wall. The tension in the sides, to be taken by the steel is, $T = \frac{wD}{2}$, in which w represents the unit load, or

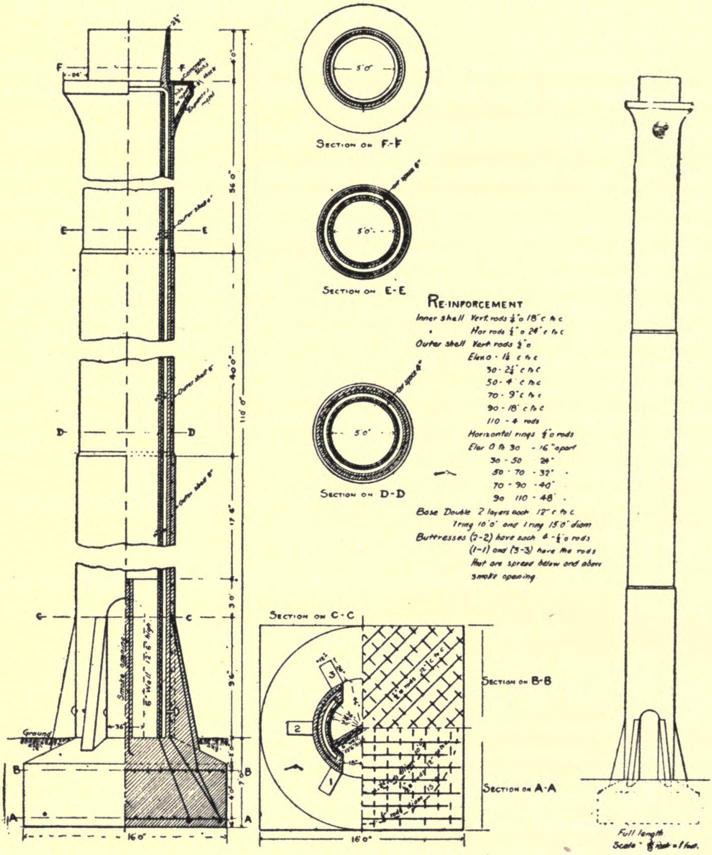


Fig. 9—Reinforced Concrete Chimney.

pressure per sq. ft., at any definite depth, and $D =$ internal diameter in inches. $T =$ total tension on 12″ width.

The area of steel to take care of this tension is found by dividing the tension by the safe unit stress in the steel. By calculating the tension for each foot in width the closest economy may be obtained in design.

The least thickness of concrete should be four inches to a depth of about six feet, but for deeper tanks the bottom thickness in inches should be equal to h/2 in feet, and the thickness at a point six feet from the top can be four inches, remaining at that to the top, while the remainder of the wall gradually increases to the maximum thickness at the bottom.

There should be some vertical steel to help preserve the intervals between the horizontal rings, and if the tank is a very high one this vertical steel should be proportioned to resist stresses caused by wind against the tank as a circular hollow cantilever beam. This will now be discussed, as it is also of value in designing chimneys.

For chimneys the depth of the foundation is from 1/10 to 1/6 the height. If the ratio of base to height is small the foundation must be spread, and it is often made in the form of a truncated cone or pyramid. When reinforced concrete is not used a common rule is to make the width of the foundation equal to the width of the chimney, or tank, plus one-tenth. The bottom of the foundation is one and one-half times that width. When a reinforced concrete base is used the upward reaction on the base is the action of a load on a cantilever beam having a length equal to the distance from the edge of the tank or chimney to the edge of the foundation, and pressure per sq. ft. at edge is,

$$F = \left(\frac{b - d}{b \times d}\right) W$$

The pressure of the wind will be 50 lbs. per sq. ft. on a plane equal in height to the chimney, or tank, and having a width equal to the outside diameter. Through the center of gravity of the structure drop a vertical line and make it equal in length to the weight. Through this point draw the total amount of the wind pressure and obtain the resultant. If it falls outside the middle third of the base then it requires reinforcement. Having the wind pressure the bending moment at the bottom is found and it is required to find the steel to resist it.

To find the resisting moment of the steel we have to find the section modulus of a hollow steel cylinder and to find the resisting moment of the concrete we have to find the section modulus of a hollow concrete cylinder. The thickness of the concrete in this case, for a high structure, will be found by taking a minimum thickness at the top and maintaining it until a depth is reached where the load is about 200 lbs. per sq. in. The thickness will then be increased from time to time so the pressure, due to the weight of the concrete, will at no time exceed 200 lbs. per sq. in. This will then give the thickness of the shell at the bottom, which is the thickness we will use in the calculations.

The section modulus for a hollow beam is $S = 0.0982\left(\frac{d^4 - d_1^4}{d}\right)$ in which $d =$ external diameter in inches, and $d_1 =$ internal diameter in inches.

When $M =$ bending moment in inch lbs
$S =$ section modulus in inches.
$f =$ fibre stress in lbs. per sq. in.

$$M = Sf; \qquad S = \frac{M}{F}; \qquad f = \frac{M}{S}.$$

To find the section modulus for the steel reinforcement consider it as a thin sheet arranged in a cylindrical form, and, putting the two diameters, d and d_1 in inches, find S. Divide the overturning moment in inch lbs. by the fibre stress and get a second value of S. Take the area in sq. ins. of the steel in one circumferential foot of the structure as assumed by the thin sheet, and multiply by the second value of S, obtained by dividing the moment by the fibre stress in the steel. Divide the quotient by the first value of S, found by the foregoing formula, and the result will be the number of inches of steel (area of vertical rods) required in each foot.

To find the maximum intensity of compression in the concrete is the next step. Having already obtained diameters for the concrete, find S by the formula. To get the second value of S we take one-third of the first value of S for the steel and multiply it by n (the ratio of the moduli of elasticity) and add to it this first value of S of concrete. Dividing the bending moment by this second value of S for the concrete we get the pressure per sq. in. on the windward side of the structure. To this must be added the unit weight of the structure, of course, and we thus get the total compression in the concrete at the base on the windward side. If it exceeds 350 lbs. per sq. in. the thickness at the base must be increased.

The steel rods are vertical and must extend into the base far enough to have sufficient bond.

Footings.

The area of the footing under a wall must be sufficient to keep the load within the permissible amount. In the case of a wall the footing must project a certain number of feet on each side so that it will really be like two cantilever beams, each carrying half the load. The width of each beam will be 12 inches and the length will be the quotient found by dividing half the load by the permissible load per square foot on the foundation.

The shear at the edge of the wall will be equal to the load on the beam at that side and we may assume not more than 300 lbs. per sq. in. for direct shear. This will fix a minimum thickness for the slab. The calculation for bending moment will give a thickness and the greater thickness is to be selected. The thickness given by the bending moment, however, will be used to proportion the steel. The steel rods must run clear across from one edge to the other, under the wall, near the bottom of the footing slab. They must be of a size that will furnish sufficient area for bond and then the amount of steel for stirrups should be calculated to take care of internal stresses. These stirrups will be close together just under the edges of the wall. (See pages 25, 31 and 36.)

Footings under columns will have enough square feet in them to spread the load sufficiently and the steel will be proportioned on the theory of the column standing on two beams crossing under the column, each equal in length to the width of the slab. There will then be four cantilever beams, each having a width equal to the thickness of the column and a length equal to their projection beyond the column. To help bind the base together, diagonal rods, equal in number to the rods in each of the beams mentioned, will run to the corners. Sometimes rods are placed around the outside as well to connect the ends of the cross rods. Usually steel is placed both ways like a network, or expanded metal or wire fabric are used in addition. Investigations must be made for shear and bond, and stirrup reinforcement provided where found necessary.

In putting down footings it is well to place three or four inches of concrete and lay the steel on this; in the footing put steel plates on which to rest the vertical steel.

CHAPTER V.

Design and Cost.

Do not approach the design of a reinforced concrete structure with the idea that the material is miraculously endowed with wonderful properties and has utterly changed its character because of the reinforcement.

A noted Frenchman, M. Consideré, made some experiments on beams by loading them until they broke. He sawed slices from the bottom underneath the reinforcement, and claimed to have discovered that concrete beams, when reinforced, stretched ten times as much in the bottom as plain beams, *without injury to the concrete.*

So far as an ordinary mortal could see, the concrete had the appearance of ordinary concrete. There was nothing to distinguish it except the fact that the professor said it was different. Strange to say, when separated from the steel, it was exactly as strong in tension and compression as before.

Professor Turneaure, of the University of Wisconsin, was one of the unbelievers in this miracle, so made tests of his own. He placed beams in a testing machine with the reinforced side on top. Instead of resting them upon supports and having the load applied as weight, he applied pressure at the bottom, and the supports were on top at the ends. As the pressure was applied, the beams bent upward, and when water was poured on the surface fine dark lines appeared. When the beam was removed from the testing machine and pieces sawed from it for testing, it was discovered that when cut between two dark lines there was no apparent change in the general properties of the concrete. When a piece, however, included one of the fine dark lines it fell apart, thus proving the line to be a crack.

This set of experiments showed that the concrete, when reinforced, was the same old concrete. Instead of having miraculous properties developed by the reinforcement, it was the reinforcement that stretched. The concrete simply developed innumerable fine cracks. If the adhesion is good, these cracks are uniformly distributed and are not visible to the eye. In fact, some may require a remarkably good microscope to discover them.

A very few men dispute the experiments of Professor Turneaure as being not conclusive, but by the majority of engineers they are accepted as establishing the true action of concrete in the tensile side of a reinforced beam. If this is the fact, then it is proven that failure begins in a reinforced concrete beam from the moment the load begins to act. Cracks form just as soon as the load exceeds the tensile strength of the concrete, and these cracks continually enlarge until the beam fails, either by the steel parting or by the concrete crushing at the top, alone or in combination with breaks caused by internal stresses in the concrete.

Assuming that cracks open in the bottom under the steel, the importance of having the steel protected by plenty of mortar underneath is seen. A very small crack will admit moisture and corrosion commences. This is a warning not to attempt to use too small a percentage of reinforcement, nor too small a factor of safety. Deflection must be considered, for when a beam deflects it bends, and when it bends the concrete underneath the steel cracks.

Strictly speaking, there is no modulus of elasticity for concrete, for the reason that concrete is not a uniform material. It is no more uniform than stone, so far as mere strength is concerned, and every one knows that stone varies in strength in the same specimen. The variations in concrete come from differences in proportioning the mixture and also in mixing the aggregates. This being the case, the term "factor of safety" is a misnomer. There is no factor of safety with concrete as there is with steel.

What we know about concrete, that renders it suitable for use in building when combined with steel is:

That owing to the presence and even distribution of the cement, it is more durable than the best of stone when exposed to the atmosphere;

That when carefully made, with the aggregates proportioned and manipulated as experience has shown to be best, we know that a stress of 500 pounds per square inch in compression may be used with a reasonable certainty that the ultimate strength of that particular concrete may be five or six times the stress allowed. As steel is a carefully made, homogenous material, such a ratio can be assumed as exact through every portion, and can be termed a factor of safety. With concrete, however, the factor of safety is entirely an assumption which applies to the concrete as a whole. It may be large for some portions and be small for other portions of the mass.

That the strength of concrete depends upon the character of the aggregates. Thus cinder concrete is stated to be one-half as strong as stone concrete. Sandstone concrete is not so strong as concrete made of basalt or granite. Limestone concrete is very strong and satisfactory under certain conditions, and not so reliable when conditions are not right.

That the strength of concrete increases and its resistance to atmospheric influences is greater the older it is, for the cement protects the other aggregates.

That in setting concrete shrinks, and consequently it grips fast all steel, or anything else of an impervious nature that is enclosed within its mass. This gripping action is entirely physical and not chemical. It is termed adhesion, and this adhesion is said to be impaired by continuous vibratory shocks, and is known to be impaired by continuous submersion in water. This impairment may never proceed to the point of destruction of the adhesion, but if high stresses are used in the steel it is well to have the additional safeguard of mechanical bond, obtained by the use of deformed rods and bars.

On the point of adhesion it is well to remember that there is no special affinity between steel and concrete, any more than there is between oil and water. To test this, place concrete on a smooth steel surface and let it set. When dry, kick it off, which can be done easily. Set a steel I beam on edge and plaster the space on one side with concrete, or, better, set a board alongside as a form and pour concrete into the space. When perfectly set, hit the other side of the I beam with a hammer and see the concrete drop away.

Put a flat steel bar on edge across a shallow box, the one edge resting on the bottom, the other level with the top, thus making a partition. Fill the box with concrete and allow the concrete to set. When dry, remove the sides of the box, and, after the material is thoroughly set, see how slight a blow will cause the concrete to separate from the steel.

The shrinkage proceeds from the outside toward the interior. The special fact that enables us to use concrete and steel in combination is that each material expands and contracts in almost identical degree under the influence of changes in temperature. If it were not so, then changes in temperature would cause each to move in a different degree, and the adhesion would soon be destroyed.

When it is known that the adhesion is entirely a gripping action, the value of small rods and bars for reinforcement is proven.

In fact, when discussing adhesion, the values given should be given to half-inch bars. Bars of less diameter will have proportionately greater and those of larger diameter proportionately smaller adhesive value. This difference probably varies as the cube root of the diameters, although there is no absolute basis yet for stating the difference to exist in such degree.

In his own practice the writer has the space between reinforcing bars, or rods, never less than twice the thickness or diameter. Underneath the steel the minimum thickness is twice the thickness or diameter of the steel, except when the steel is in the form of wire, when the least covering is half an inch. In slabs liable to be exposed to the action of fire, the minimum thickness is one and one-half inches and for beams two inches. For columns two inches, and for tank walls subjected to constant immersion, two inches.

The gripping action of concrete should not be too greatly checked, and for this reason round rods are favored by many designers. It requires no argument to show that the wet material is more apt to set well in contact with a round bar than with a square bar. Assuming very low unit stresses in both steel and concrete, the use of plain round bars is entirely defensible for 90 per cent of the structures erected in reinforced concrete. The other 10 per cent may be in situations such that the designer feels a deformed bar to be necessary. To discuss the deformed bars in the market in this connection would place the writer in an embarrassing position. He has used practically all, and, as before stated, to get a certain strength requires a certain percentage of reinforcement, regardless of shape. The deformation acts as anchorage, except with the Kahn bar, where it performs a double function, being anchorage as well as web reinforcement.

A round bar permits the material to flow around and grip it better than will a square bar, yet it does not offer the same surface for adhesion. A square bar twisted gives the adhesive surface, while it makes practically a round bar with corrugations through which the concrete may readily flow. For this reason a square bar twisted is economical, as every square inch of cross section is available for reinforcing purposes, while the twisting gives the mechanical bond. Some men claim the edges cause cracks to start when the concrete is shrinking. Others make this charge against all bars not having rounded edges. The writer believes most of this talk to be purely theoretical, and that any bar in the market is good, provided enough area is used to give the desired strength.

When designing a structure, go over the design carefully several times. If wind strains are to be feared, it is wise to use round bars and have the ends threaded, so that all connections will be made by screwed couplings. Connect columns, walls and girders by bolsters or brackets, reinforced to take care of twisting strains liable to be caused by wind or earthquake. See that all connections at corners and at beams, girders and walls are carefully computed and not merely guessed at.

It is such a common practice to make a continuation of reinforcement by simply lapping ends past a few inches that a warning must be sounded, although the matter was dealt with fully in Chapter I. Lapped connections greatly increase the stresses in the concrete at those points. Screwed connections are good. In columns the ends of the steel should rest on plates, as already mentioned, and the ends of the bars should be milled. A continuation should be made inside a sleeve, and if high winds are feared, the ends should be connected by screwing.

Bars and rods should be wired together at all crossing points to preserve the intervals. Black stove wire is generally used. This is known as No. 18, and should be black, as galvanized wire is not so good for adhesion as black wire. No. 16 wire is favored by many engineers as being stronger than No. 18, and a number of men require the use of No. 14, which is excessive and is more consuming of time to place and twist. For all-round use No. 16 is best, but requires the use of pliers, whereas No. 18 can be twisted by hand. The way pliers disappear on a job is an argument in favor of the lighter wire.

Never use painted or coated wire for reinforcement. Galvanized wire or steel rods should not be used except when provided with mechanical bond. The galvanized fabrics in the market, of course, have the mechanical bond created by the fastening of the crossed wires. A slight rusting is not harmful. The writer made some experiments several years ago by enclosing slightly rusted steel in wet concrete. After a year the blocks were broken and the steel was found without rust. The surrounding concrete was found to be slightly discolored. When the rust is in the form of scales, it should first be removed.

Many buildings are weakened by holes cut at random through slabs and walls. Common sense should be a guide here. It is well to remember that when a slab is made the strength is assumed to be that of the slab with the reinforcement complete and tied from support to support. If the steel is cut, it is equivalent to destroying one support, and the action of the slab on each side of the hole is a cantilever action, but as the reinforcement is in the bottom, the steel is of no assistance. This is an argument in favor of double reinforcement, in addition to the argument that may be presented from the fire protection standpoint.

Holes, therefore, should be located while the structure is being planned and the reinforcement arranged accordingly. The reinforcement should generally be in the form of two double reinforced beams each way, one on each side of the hole, or the slab can be designed as a cantilever beam on the four sides of the hole, the remainder of the slab resting on and being carried by the cantilevers.

As the fibre stresses in wood and in reinforced concrete differ slightly, the sizes of beams and thickness of floors in a slow-burning wood construction building and a reinforced concrete building are near a size. The reinforced concrete, however, will be more massive than the wood.

For this reason some men become panic-stricken when they see the building going up, and are apt to make some members smaller or lean toward the danger point in designing because "the thing looks too big." A reinforced concrete structure is unlovely from a structural standpoint. In the preceding chapter an example was given of how column sizes compared in different materials. A comparison in beams would be even more striking.

The eye accustomed to wooden beams as the biggest beams required to give certain strength does not like the appearance of reinforced concrete beams, and, as many of them show the grain of the wooden forms and the gray color is much like that of seasoned, weather-stained wood, a reinforced concrete building sometimes looks like a big, clumsy wooden structure. Perhaps a style of architecture peculiar to reinforced concrete may arise and have a beauty of its own so it will not offend.

Most of the data obtainable about cost of reinforced concrete work is misleading. It has generally been given out by men who simply were on the work at intervals in the capacity of supervising engineers and who did not know anything of the thousand and one exasperating delays incident to the work and who knew nothing of the dense stupidity of the laborers often employed. Sometimes the data has been given on the authority of some company controlling a splendid organization and all of the employes were skilled men working to standards and on only one class of work.

Look where one will it is almost impossible to secure actual cost data of reinforced concrete work. Most of the work is done by men who look aghast at the figures when a job is completed and are ashamed to publish the costs. They seem so much higher than current published data that they think it a reflection upon themselves and wish to try another job before giving out figures.

Their work is comparatively small and no jobs last long enough to enable them to perfect an organization. Neither do jobs follow quickly enough to enable them to retain men who have been broken in as foremen. This class of work, not done by regularly organized companies engaged in training skilled foremen, carpenters and laborers on one job for employment on another, is the real criterion for gauging the cost of work.

Much is written about the low cost of reinforced concrete work because of the fact that unskilled labor is largely employed. A visit to a good job will show that the unskilled laborers are in the minority. For every laborer who pushes a wheelbarrow and shovels concrete materials there will be employed a carpenter or skilled man.

While unskilled labor can be, and is largely, employed, there is a vast difference between the unskilled man who is born and reared in America, who can understand English, and the unskilled foreigner who has to be addressed in the sign language. Unskilled laborers are divided into two classes, the intelligent and the unintelligent. The intelligent ones are the kind to employ in reinforced concrete work, for the various operations are rapidly assuming the importance of trades. After a few months' work the active intelligent laborer has a right to be classed among skilled workers. Intelligent unskilled laborers are divided into rapid and slow, drunkards and nondrinking men, men who care and men who take no interest in anything except the whistle and the pay check. The success of the contractor can be gauged by his luck in picking up good men.

Carpenters have been mentioned. The men who apply for jobs as carpenters on this class of work are divided into wood butchers, bluffers, saw and hammer men, half trained apprentices, sidewalk and fence builders, cabinet makers, inside finishers, millwrights, bridge and trestle builders and good plain ordinary old-fashioned carpenters. The best carpenters are men who have really learned their trade and who have worked some years in small towns. Such men generally take to the work as ducks to water. All the others above enumerated have to be taught. Some never learn. Form and scaffold work for reinforced concrete jobs call for a distinct class of woodworkers and if they can be obtained, time and money are saved.

The average job of reinforced concrete is done by men who commence the work without a skilled worker to assist. They must advertise for men and try to select from the applicants such men as they think may develop. At first they take all who come and begin to weed and select after a start has been made. For weeks they conduct a kindergarten and just as a good working force is developed may be discharged and a hustler comes who takes the trained crew and makes excellent progress.

It must be remembered that skilled men in concrete work are hard to find, although the streets are full of men making great claims of proficiency. The really skilled men are in the permanent employ of the men who trained them or are in well-organized unions and averse to leaving the larger cities. It is well to be very careful in taking men who claim to have had experience. So many contractors (and engineers and architects as well) are criminally careless in placing reinforcement and looking carefully after details that a large number of experienced men are abroad who can not be trusted to do anything well on such a job except to "hustle 'er tru'."

An ordinary reinforced concrete job has carpenters and their helpers, making, setting and removing forms; men placing, altering and removing scaffolds and changing running plank; a steel yard where steel is assorted according to sizes and lengths and men engaged in bending and cutting steel; men placing steel in position before the forms are placed; men cleaning concrete and steel ahead of the forms and men putting in concrete. The unskilled, low browed unintelligent laborer has really no place except between the handles of a wheelbarrow and it takes many of a superior kind to keep the work so fixed that a few of the stupid ones can be kept moving. What is wanted on such work are handy men possessing considerable intelligence who will do anything, even to pushing a wheelbarrow if laborers run short.

There is nothing in low cost labor. Really cheap labor is well paid intelligent labor. When a man is efficient he must be paid correspondingly well for there is a constant demand for such men. The man at the head can not be always running a school. It is argued that there is nothing great in placing steel or placing concrete. Neither is there anything great in making a table, but the table made by the amateur does not look like the table of the craftsman. Reinforced concrete work requires intelligent men, but generally several hundred men go through a job before fifteen good ones are secured and twice that number before one decent foreman can be obtained.

All concrete work is not alike, hence the unreliability of most of the published cost data as a basis for estimates. Some structures are made completely of concrete with thin walls. Some have thick walls. In some the reinforcement is vertical and in some it is horizontal. In some all material is wheeled in barrows to the mixer and in barrows from the mixer to the forms. In some places automatic elevators, super-hoppers and grab buckets and a multitude of labor-saving devices are employed.

Some buildings have merely columns and girders and beams of reinforced concrete with reinforced concrete floors and roofs and the exterior walls of brick with interior walls of tile. There are good systems in use for flooring, involving a combination of reinforced concrete beams and tile. Some systems use frames of I beams instead of concrete beams. Some buildings have been erected having all the structural framework of reinforced concrete cast in shops near the work and erected like steel after the concrete has hardened thoroughly.

Remember that in erecting a reinforced concrete structure with solid walls two complete wooden structures are erected and taken down. Nearly all the lumber is afterward useless unless the forms are made in interchangeable panels and a new job is at hand so they can be used immediately. Scaffolds for ordinary buildings do not require to be so strong as scaffolds for concrete work, so the costs for scaffolding and running plank are high.

The greatest improvement in the future must be in the line of reduced cost for forms and simplicity of forming. Many builders are ashamed to tell what their forms cost. Each time they bid higher on work they blame the increased prices to the increased labor and material prices.

The cost of concrete can readily be figured. The cost of getting materials to the mixer and of mixing them can easily be figured. The cost of getting concrete from the mixer to the walls depends upon the length of run and convenience of arrangement of the running plank. In a well arranged, well managed job, $1.00 per yard should take the stone, sand and cement from the stock pile and shed through the mixer and into the wall.

The forms can not be figured on a yardage basis, except in the case of a company engaged in certain kinds of work of a standard nature. After several jobs a per yard cost for scaffold and forms can be obtained. It will, however, not be a safe guide for other work. To estimate the form work, the style of forms to be used must first be decided upon. Then figure it on a per thousand foot basis. With cheap carpenters it will cost from $8.00 to $20.00 per thousand to make forms. With good, higher paid carpenters it will cost from $5.00 to $7.00 per thousand to make forms. It will cost about $6.00 per thousand to place them and about the same to take them down and clean them for use again.

There is a big waste of lumber in form work and to reduce this waste only carpenters or the best of the workmen should take down old forms. If the wheelbarrow man is put at the work he will ruin them. The cost of cleaning up and the general expense on a job will be nearly $2.00 per cubic yard of concrete. To estimate work exactly, careful detailed drawings should be made of the forms. They should be designed as carefully as the rest of the building.

As some approximate rules are always wanted, the writer can offer the following as fitting very closely actual conditions on a job having experienced, well-trained foremen, and among the laborers a good proportion of men who have worked on similar jobs. This rule applies to ordinary building construction, for the cost per yard for sewers, culverts, retaining walls, etc., is much less than for buildings.

Get exact costs of sand, stone and cement delivered on the job and reduce the costs to cubic yards of concrete. To this add $5.00 per cu. yd. for steel. This will be one-half = 3/6, the cost of the concrete per cubic yard in place. The labor on concrete and steel will be 1/6 and the material and labor on forms will be 2/6. For average buildings containing about 2,000 cu. yds. of concrete this will be about true. Add 2 per cent to cost for each 100 cu. yds. less than 2,000. Complicated work will increase the cost greatly. To this must be added cost, or rent, of plant, and the profit of the contractor.

If the materials are obtained at exceptionally low prices, of course the above rule does not hold. It holds for average pay in the locality in which the work is done, for, left without a union organizer to run up pay, laborers in different sections are generally paid what they earn. It is always best to pay ruling rates of wages everywhere and not raise the pay, except to good men.

Sometimes men are frightened when estimating on reinforced concrete work because the costs run so high. While the cost of the building seems to be all right, the cost per yard of concrete seems fearfully high. A common sense rule is to consider that a reinforced concrete building does not cost less than a brick building equally as well constructed. The cost of a brick building being, say, 15 cents per cubic foot measured from the basement floor to the average height of the roof, with the length and breadth taken as over all dimensions, make first an estimate of the reinforced concrete building in that way. Then go through the plans carefully, taking out the exact yardage. Dividing the total estimated cubic foot cost by the yards will give a cost per yard close enough for a check on the careful detailed computations that will later be made by the careful estimator.

The writer would like to see more men making money at this class of work and knows of many who have lost money through wrong estimates, or through rushing into work with ideas fixed. Left to himself, the experienced man will generally get out a good plan and a close estimate of cost. There is so much irresponsible matter floating around in papers about reinforced concrete that few designers are given a free hand in the design and none of them are allowed to make a correct estimate. The employer will always declare the estimate "Too high!" and give instances of work he casually read about, or some forgotten individual told him about. As a result few buildings in this material are erected at the estimated cost. When the owner has his work done on a percentage basis it falls on him, but if some contractor new at the business takes the contract on poor information he is ruined.

Good work costs money, and work of a certain grade differs little in cost, no matter what the material. If reinforced concrete is absolutely fireproof, earthquake-proof, everlasting and never needs repairs, it is reasonable to suppose that for such perfection a price must be paid.

CHAPTER VI.

Forms.

The ideally conducted job is where the men in charge of the steel hanging get out of the way of the form placers so the forms may be filled. The steel should be placed, forms erected, concrete poured, additional steel placed, forms re-erected and concrete poured again, all without a hitch and without the mixer having to stop. As a matter of fact it is almost an impossibility to get the average sized job so well organized. On very large jobs and on concrete work where there is no reinforcement and where the walls are comparatively thick it must be done and is done right along.

The chief reason for delay is in the setting or erection of forms and the intelligence and experience of the men charged with the work of form erection and changing. On comparatively thick walls the time it takes to do the form work is seldom sharply drawn to one's attention for it takes so long to fill such walls that the form workers keep ahead of the concrete gang without trouble. It takes as long, however, to erect forms for a thin wall as for a heavy wall and sometimes considerably longer, for thin walls are generally cut up with pilasters or panels. The pouring of the concrete, however, is almost as rapidly done in a thin wall as in a thick one and as there is so much less concrete to pour the concrete gang soon overtakes the form gang.

Because of this the cost of forms per cubic yard of thin walls is much higher than for thick walls. It requires more lumber and while it takes little more labor the labor cost is higher in proportion, the thinner the wall. When very thin walls are used the cost is very high until the men become experienced and work like machinery. A great many inexperienced men think the cost of lumber the largest item but when lumber costs less than twenty-five dollars per thousand and inexperienced men alone can be had it will be found least expensive to sheet right along up and pay little attention to trying to use the material several times. The proper way to estimate forms is by square foot of surface and not per cubic yard of concrete.

Dressed and matched lumber is good for use once or twice

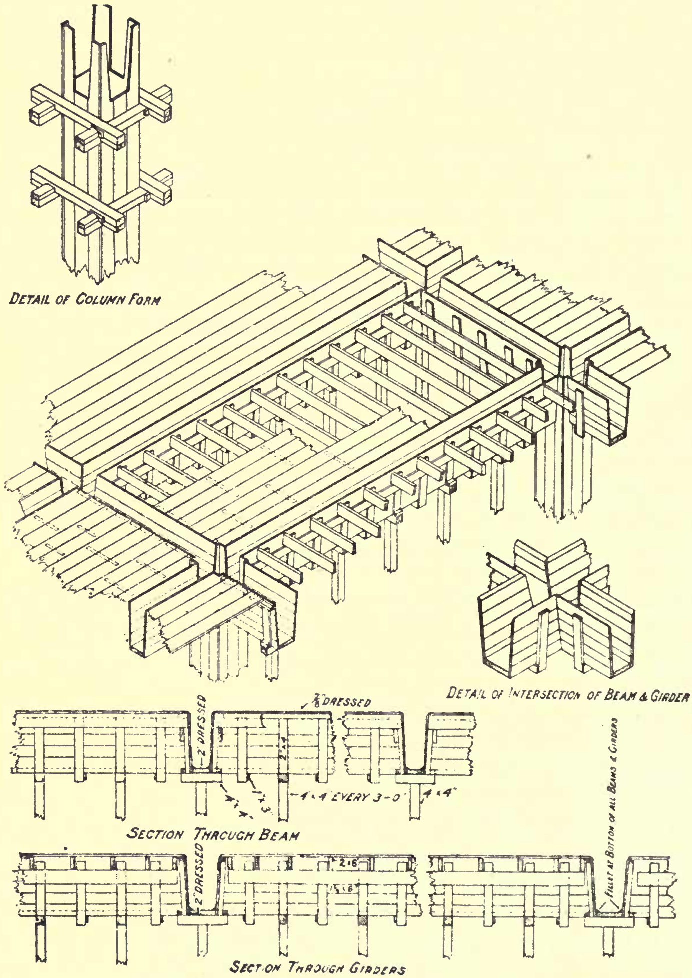


FIG. 10—FORM PLANS (GENERAL FIREPROOFING COMPANY).

and is not such a good material for re-use as plain edged lumber. Therefore it is better for thick walls than for thin ones, as the cost per yard for forms, when figured that way, is not high. While the pressure is as great in thin as in thick walls, considering concrete as a semi-fluid, thinner lumber can be used to advantage in thin walls, for it is easily handled and can be tied through the wall readily when necessary.

When experienced men can be had and lumber goes over the twenty-five dollar mark per thousand then it will pay to use some form of panelling or try the "board by board" method. Whether panels, or single boards, or the studding and sheathing method is considered best, it pays to use the boards and pieces in as near standard lengths as possible for they might be worth something after using. If this item is not looked after, all the lumber will be fit only for the scrap heap.

The writer believes the best carpenters to use are experienced men who have thoroughly learned their trade and the boss carpenter should be an elderly man accustomed to handling men. A few first-class men can do much more work than cheaper laborers. It is customary to hear men say that common laborers are good enough for carpenters on a concrete job. The writer begs to differ with them and say that if for nothing else than the habit a trained man has of staying with a job, the experienced, well-trained carpenter is best. The ordinary unskilled laborer is nomadic and as soon as he thinks he has learned something new he leaves the job to sell his services elsewhere. The writer prefers good carpenters as gang bosses and under each good carpenter put three to five handy unskilled laborers as apprentices and to do the roughing work. This is especially true if any calculation is made on using the material more than once.

The unskilled or incompetent man will use many more nails than the trained man. This means expense for nails, more time putting up the forms, more time taking them down and the likelihood of ruining a great deal of the lumber in trying to take it down and re-use it.

Fig. 11 represents the regulation form built up of boards nailed to upright studding and braced. The planks may be of any thickness according to the fancy of the carpenter or of the contractor. The studs may be of any convenient size and the braces and posts against which they rest will be arranged according to the ideas of the men in charge. For this style of forming each man's fancy is every man's guide. Common custom, however, seems to be coming around to one inch boards against two by four studs set two feet apart. The braces, however, may vary in size from two by four inches to six by six inches with little attention paid to the manner of securing the ends.

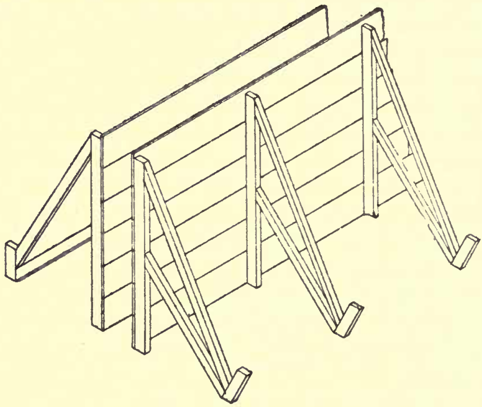


Fig. 11—Simple Braced Form.

In the illustration the braces are bevelled where they rest against the studs and are supposed to be fastened by "toenailing." This is very poor as the full strength of the brace cannot be secured. In spite of everything the nails will move a little and sometimes the end of the brace will slide up the stud and pull the nails entirely out.

The upper end of the braces should be cut square and butt against a block nailed to the stud. Instead of the push helping to draw the nail it will be at right angles and the nail cannot give without shearing. The length of the block and the number of nails to use depends upon the push. These things are seldom figured out.

The trust the ordinary wood butcher places in nails is pathetic, especially after a bad spill caused by a form giving way. Plenty of nails will not always do the work which should be done by plenty of brain. In a most excellent booklet distributed by a well known cement company is an illustration of forms braced by having the braces nailed to the sides of the studs instead of butting against them. Instead of the lower end of the braces resting against posts or on sills they rest simnly upon the ground. The writer has seen many braces thus placed and he has ceased to wonder at it being done but does wonder at the mental processes wherefrom such ridiculous results proceed. Common sense seems to be at a discount.

It is common to hear of braces giving way and yet in every case investigated by the author he has found it to be due to improper securing of the braces, except when a rain may have softened the ground into which the supports were driven. If the ground is exceedingly firm it is sometimes sufficient to drive a post as shown in Fig. 11. Whether or not the ground is firm the best and safest way to secure the lower ends of braces is shown in Fig. 12. Here a line of posts is driven and a two by six or heavier plank laid against them. Back of this line is driven a second line of posts with a brace from the top of the first line resting against their feet. The braces from the forms rest against the plank and it is wise to have two hard wood wedges at the end to take up whatever slight movement may develop.

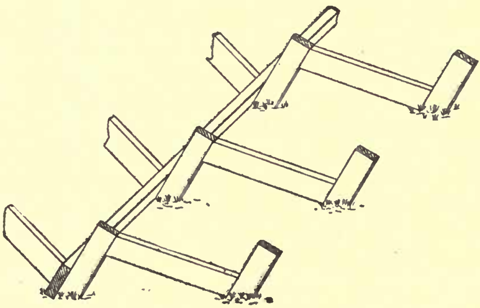


FIG. 12—METHOD OF SECURING FORMS.

While on this subject the writer wishes to say he does not consider bracing to be good practice when forms are used on both sides of walls and can be connected by wires or bolts readily. Bracing is only excusable when all the securing can only be done properly from the outside and from one side. Braced forms give way more readily than bolted forms, take more time to put up, use up more lumber and all round are least satisfactory of all methods used. Bolts are best and wires next with braces in the third place as regards merit.

When braced forms give way it is generally through ignorance of proper methods of securing the braces. The times, however, when the foundations give way are so numerous that one cannot be absolutely certain a properly braced form will not burst. When bolted or wired forms give way it is only when the bolts or wire are ignorantly placed.

Wire is very expensive for it requires a great deal of material and a tremendous amount of time. When the forms are taken down the wire projects and the ends must be cut off. They leave rust spots, disfiguring to the wall. A green man will run wire through both forms and twist the ends to form a loop. On one side he will put a soft wood wedge under the wire thinking that when the form gives he can drive it down and tighten the wires. Vain hope. If he can drive the wedge at all, which is something

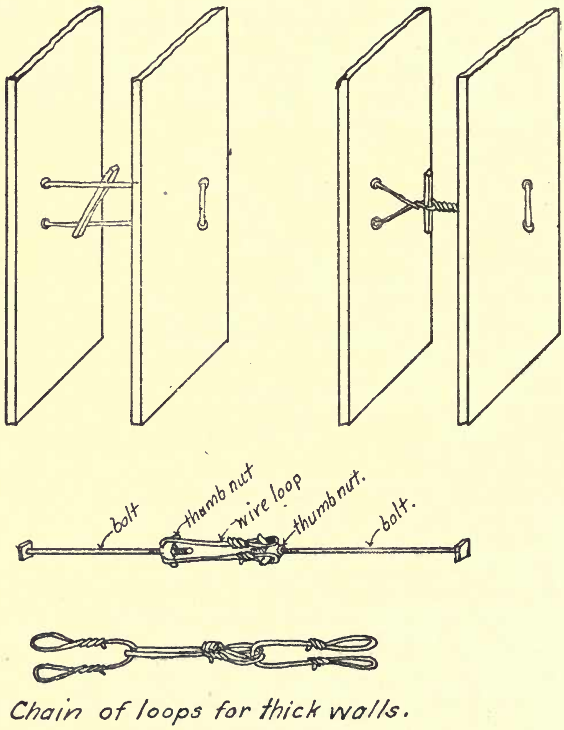


FIG. 13—METHOD OF WIRING FORMS.

to consider, he will either shear it off so it does not tighten the wire, or he will only succeed in driving it so the twisting will come undone and his wedge makes things worse. To use wire properly it should be passed through both forms and twisted inside the forms by means of a bolt or stick used as shown in Fig. 13. To keep the forms the right distance apart a piece of wood should be placed beside the wire and left there until the concrete reaches that height, when it is fished out. It happens many times that the fishing is forgotten and the ends of the stick may be seen when the form is removed. Or it may happen that in fishing for it the stick is knocked down into the concrete, there to remain, a weak spot although concealed from view. This happens so often that some men use pieces of concrete instead of wood for spacers and make no attempt to remove them.

Bolts passing through the wall have none of the objections inherent with wire. A well greased bolt can be removed easily from the wall after the concrete has set. Trouble always ensues when the greasing is not properly performed. Sometimes, however, the bolts pass through paper tubes or are wrapped in several thicknesses of greased paper in addition to being greased. Some men use hollow tubes of concrete through which pass the bolts, the tubes serving also as spacers. Wooden spacers used with bolts can be removed as soon as the bolts are screwed up. With wire the tendency, because of the twisting, is to draw up and for this reason it is best to leave the spacer in place until the pressure of the concrete relieves it. Wire will give a little and this give cannot be taken up. A bolt, however, is secured by a nut on the end bearing against a washer. When the nut has been screwed down far enough the spacer can be removed. If the form does get a little slack it will move out under the pressure of the concrete until a bearing is obtained against the washer. If the forms give while the concrete is being poured the nuts can be tightened.

While wires leave rust spots, bolts leave holes. These holes are generally filled with a neat cement paste pushed in with a small stick. If the wall is to be watertight the paste has mixed with it some waterproofing material. The grease with which the bolts were treated, or the paper tube left in the wall will not allow a perfect seal so it is almost impossible to secure a water tight wall when bolts alone are used.

While holes on the surface may be filled and thus sightliness be preserved, the hole through the body of the wall is an objection. Several arrangements have been made to overcome this by using short bolts connected to wire loops in the body of the wall. The bolts being greased are withdrawn when the forms are removed, leaving the wire loops and thus sealing the wall. While many such devices are in use the writer for years has used ordinary thumb nuts connected by wire loops as shown in Fig. 13. A machine for making the loops is illustrated in the chapter following. When one loop is not long enough several may be connected in chain form as shown. The small surface holes left by the bolts may be plugged with neat waterproofed cement, although the greasing is an objection, for it interferes with the bond.

Unless the faces of the forms are smooth and clean the surfaces will be bad. How to overcome roughness of surface on concrete, due to forms, will be taken up in the next chapter. Tables to determine proper sizes and spacing for braces, wires and bolts will also be given there.

The form illustrated in Fig. 11 is usually made by erecting studding and nailing the boards to the inside when the walls are

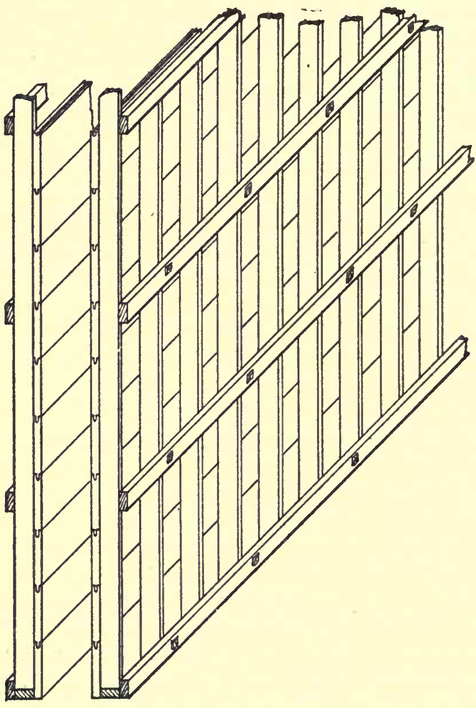


FIG. 14—FORMS OF MATCHED LUMBER.

thick. If the walls are thin the boards are nailed to the studding while lying down and then the whole side is raised, precisely like sheathing the side of a wooden house when erected close to another house. A departure is made by erecting the studding and putting in one board at a time. The lower board is set against the studding and filled with concrete when a board is placed on top and the work continues in this manner until a height of eight or ten feet is reached. No nailing is required except for an occasional four or sixpenny nail to merely keep the boards from falling down.

By starting with one board at a time a foot of concrete per day can be assured on a very large building and it should be possible to keep the mixer going all the time. When a height of about eight feet is reached a plate is placed on the top of the studs and another set started. At the bottom they are wired through the wall and when the concrete has gone up a foot or two on the second set the lower set of studding can be removed and the boards, not being nailed, fall out.

Fig. 14 shows a method where the boards are one and one-quarter to one and one-half inches thick, dressed and matched. Because of the tongue and groove no nails whatever are required and joints may be broken anywhere. A two by six sill is laid to start and this is set with an instrument and secured so the line can be maintained. Studs are set two feet apart and carefully plumbed so the wall will be truly vertical. Such forms are readily kept to line and if they get out of line can be quickly pulled back. It is important that lines be kept stretched while concrete is poured so that bulging may be remedied before the concrete sets.

The boards are set in one at a time and when a height of about eight feet is reached a new set is started. The longitudinal stringers shown with bolt heads are two to four feet apart and the upright studs are lightly fastened to them with small nails merely to preserve the intervals. The bolts run through the walls independently of the upright studs and through holes in the boards. This method is rapid and satisfactory for walls not very thin.

With the systems of forming just described it is usual to have braces against the lower set and have wire ties or bolts in the higher sets. An objection to braces not already mentioned is that they interfere with getting close to the walls to pour unless the pouring is done from a considerable height.

Fig. 15 shows a panel method developed by Mr. Ransome and very popular for thin wall work. The uprights are made of two pieces of two by four, separated at each end by blocks, thus forming slotted braces. To start a wall, spacers the thickness of the wall at the bottom are set on the ground level and an upright board set at each end. On the outside of the boards are placed sets of the upright slotted frames bolted through at top and bottom and with a spacer at the top equal in length to the thickness of the wall plus the thickness of the two boards.

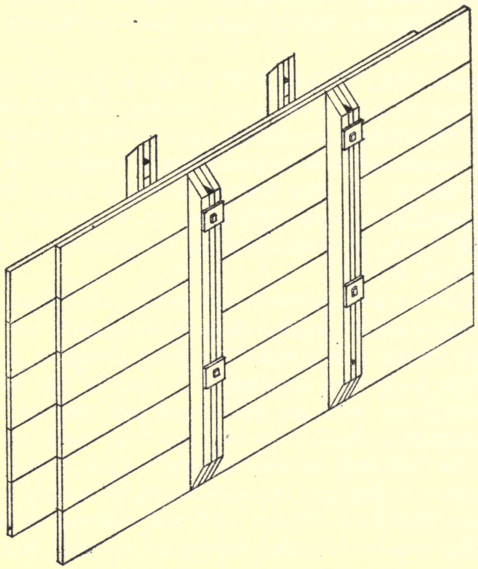


FIG. 15—PANEL METHOD.

The lower bolt is slid down the slot until it rests on top of the boards and remains there. The form is built up one board at a time until the upper bolt is reached, when it is slid down to the top of the nearest board below it. The building up is continued until the top of the slotted frames is reached, when the lower bolt is withdrawn, the slotted frames moved up to a height as great as may be obtained when the upper bolt reaches the bottom of the slot. The lower bolt is now passed through the upper part of the slot, with a new spacer there to preserve the interval, and work is recommenced. When the slotted frame is moved up the boards held by it are released and can be used again after cleaning.

Sometimes the slotted frames are moved up oftener than here indicated. Sometimes, also, instead of putting in one board at a time, panels are made of three to five narrow boards, fastened together by cleats on the back. The slotted frames are seldom more than five feet long. Panel forms of this kind have many points of weakness about them on account of the joints, unless care is taken to have the slotted frames lapping over the ends or to have two frames close together and near the ends of their respective panels. The amount of lumber required is small and thin boards may be used. To prevent undue deflection a great many frames must be used with thin boards and this means considerable labor. Each panel being, to a certain extent, independent of the others makes it extremely difficult to keep true lines with ordinary care. Men become expert in using them after awhile and as they require less material and skilled workmanship than any other style of forms their use is increasing. With such forms, however, it is found as with others that every endeavor to save material results in an increase of labor. The contractor has, therefore, to strike a balance so that by saving lumber he can use such a system of forms that the lowest paid labor can be employed to advantage.

Fig. 16 shows another panel form greatly used. Sometimes a bolt is put through the bottom with a spacer to preserve the interval and a cleat nailed across the top. The panels are made about three feet wide and can be of any length. When filled to the top the concrete is allowed to set. The form is then raised so the bolt rests on top of the old concrete and another cleat is nailed across the top. The bottom of the form is on each side of the concrete already poured which thus acts as a spacer.

It often happens with concrete work that work cannot proceed with such regularity that the forms can be moved one at a time, for some concrete sets slowly. Two sets of forms will then be used, which permits of one set being left in place while concrete is poured in the set above. It is most satisfactory to have three sets. The bolts have large washers about two by five inches on the ends. As men seem to be very slow and experience much trouble in putting the bolts through the bottom of the forms it is convenient to put them through the top with a spacing cleat nailed across above them. This cleat is of course knocked off when the panel is filled. When the panel above is placed the washer is turned so the edge of the frame is caught and thus one bolt serves two forms. The forms are generally made of one-inch boards with frames of two by four stuff. The panels are generally made with the upright two by fours two feet apart.

The last mentioned panel forms are more expensive than the slotted frame forms, but walls made with them can be kept to line as well as walls made with studding set up. All the skilled labor is employed in making the panels and ordinary men can set up and remove them. The writer, however, has had a job lately where the setting of these forms seemed beyond the ability of ordinary labor and the men who were on the pay roll as carpenters killed so much time on the form work that it was heartbreaking. So, after all, the costliness of any particular kind depends very much upon the class of labor employed and the spirit it is possible to infuse in the men.

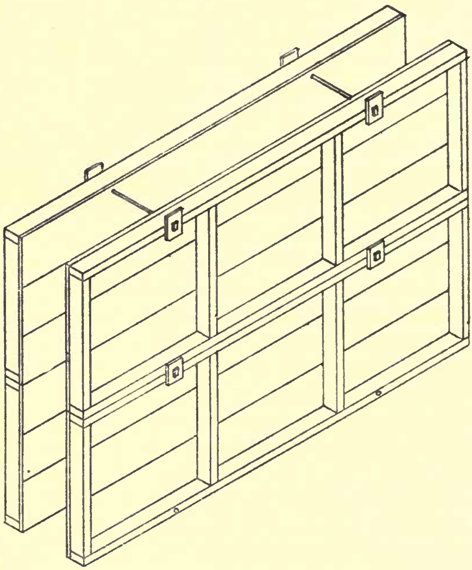


Fig. 16—Another Panel Form.

In illustrations on pages 87 and 89 are shown several patented methods of placing forms by means of steel clips and fastenings that permit of the use of one board at a time and no nails or uprights are required.

The writer has a decided preference for the panelled forms shown in Fig. 16 for use in ordinary thin walls of reinforced concrete. For heavier walls and especially for walls not reinforced the system shown in Fig. 14 is his favorite. This expression, however, is to be modified by the statement already made referring to cost of lumber and class of labor.

For long walls, not much broken by projections or recesses and where appearance is not placed high above most other considerations the forms shown in Fig. 11 can be used to advantage with very cheap labor and poor carpenters and the forms shown in Fig. 15 can be used with cheaper labor and no carpenters. The slotted frames can be made at a planing mill or carpenter shop.

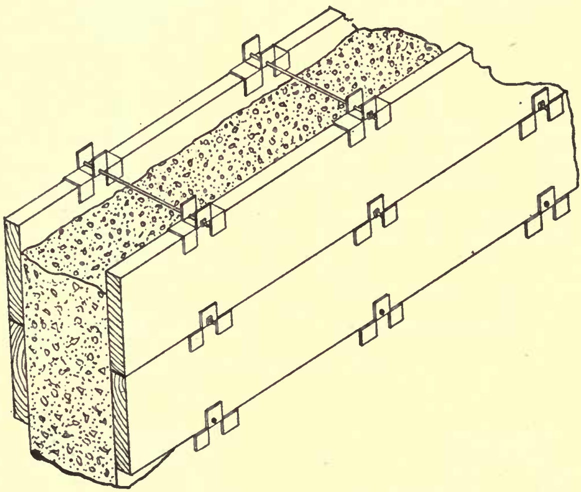


FIG. 17—SULLIVAN'S PLANK HOLDER.

For thick walls the cost of forms does not cut much figure and can be kept low, per cubic yard of concrete, so that any of the kinds mentioned can be used. The studding and boarded forms made in place will generally be found cheapest and most advantageous in every way.

When walls are cut up at close intervals by pilasters and counterforts or buttresses, and especially when such walls are battered, the forms shown in Fig. 16 will be the cheapest and most satisfactory to use, for they may be of standard lengths and the projections can be formed by special pieces. Fig. 18 shows special pieces for corners, permitting the use of standard length forms.

No office draughtsman is competent to design any reinforced

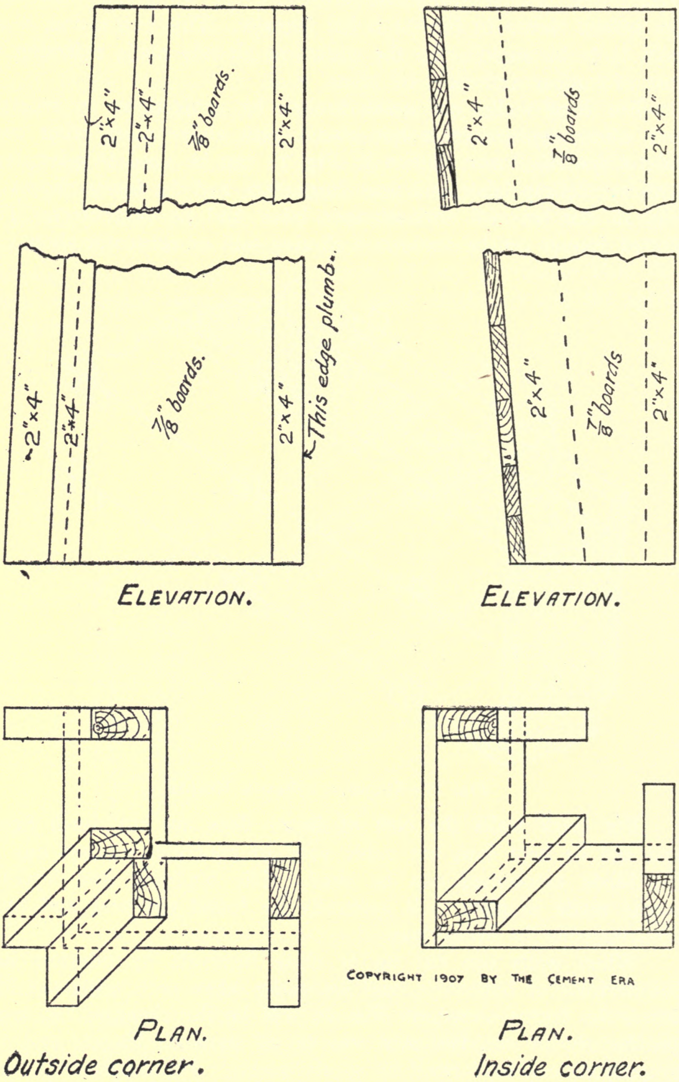


FIG. 18—CORNER FORMS FOR BATTERED WALLS.

concrete structure out of the ordinary unless he has had actual construction experience. Forms should be as carefully designed as the rest of the details of actual construction. They should be intended to strike a happy medium between economy in material and economy in labor. Forms designed with a view to only one or the other will not do. It is well to design forms with a view to re-use if possible several times.

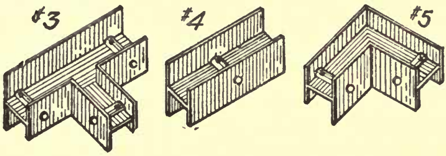


FIG. 19—FARREL'S PLANK HOLDER.

For retaining walls all the foregoing applies, for the actual construction work is the same for all walls. For sewers and tanks, panels of curved shape made on the principle of the panels shown in Fig. 16 must be used. For arches, adaptations of the forms in Fig. 11 and Fig. 16 are used, for all the material must remain in place until centers are struck.

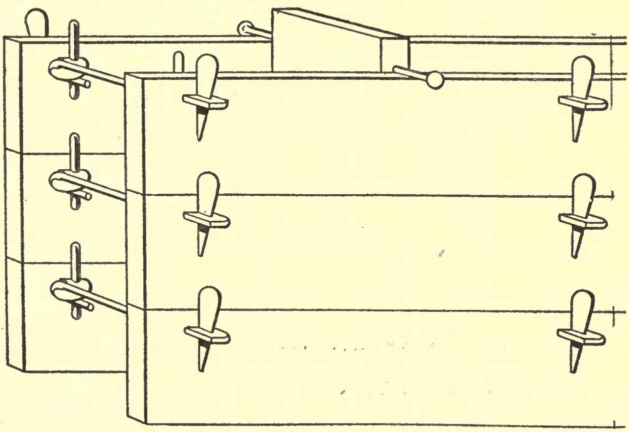


FIG. 20—DIETRICH'S PLANK HOLDER.

Draughtsmen occasionally send out designs for pier and column footings showing a pyramidal form. Avoid forms of such shape for they will invariably float and it is next to impossible to fasten

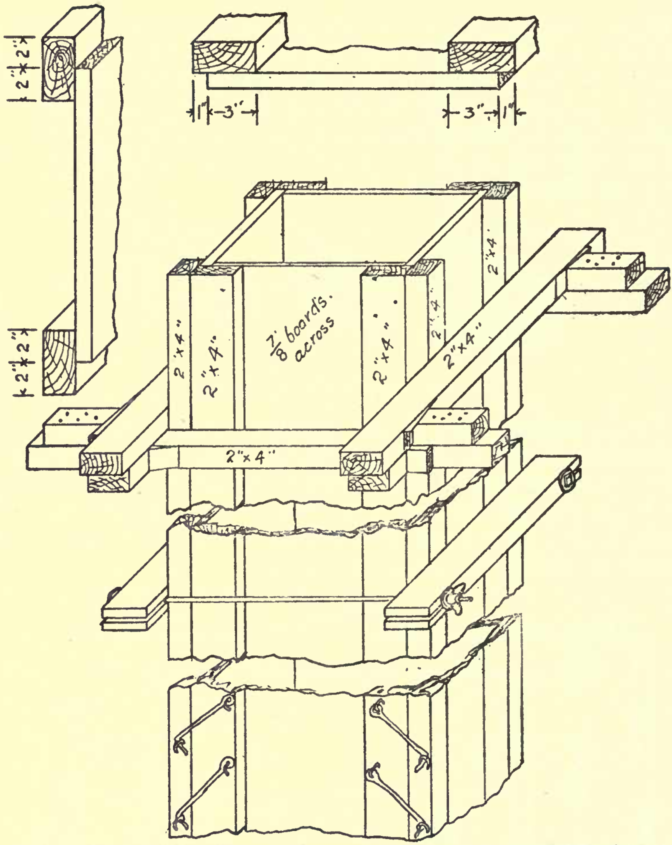


FIG. 21—THREE STANDARD METHODS FOR SECURING COLUMN FORMS.

them down. Owing to the difficulty of tamping next the face they will be rough in spite of all work done to assure a nice face. The writer uses for footings, frames six to eight inches high. One frame is put down and filled. While the concrete is still somewhat soft the next frame is put in place and filled, so proceeding until the height of the footing is attained. The frames are then removed and if the sloping shape is insisted upon the steps are filled with stiff mortar trowelled to a nice finish. Otherwise the steps remain.

For columns it is customary to make a form for each side an1 place around them frames of two by fours at intervals decided upon by the pressure to be resisted. Bolts are run through the ends of the pieces, or they have blocks and wedges. Fig. 21 illustrates column forms in general use.

Beams and girders are best poured in forms that are braced on the side like trusses. Then posts can support these forms and framing under the floor panels will rest on the lower chords of the beam and girder forms. It will not be necessary to carry supports to the floor underneath to carry the floor or roof forms above. This means a tremendous saving in lumber for bracing. It also means a saving in labor for removing the bracing. The writer knows contractors of great experience, who look closely into savings, who give the lumber and braces used under floors to any one who will remove them, if the material has to be passed out of narrow or small openings. When this material is cheaper than the labor to remove it some thought taken in saving the amount means a great deal.

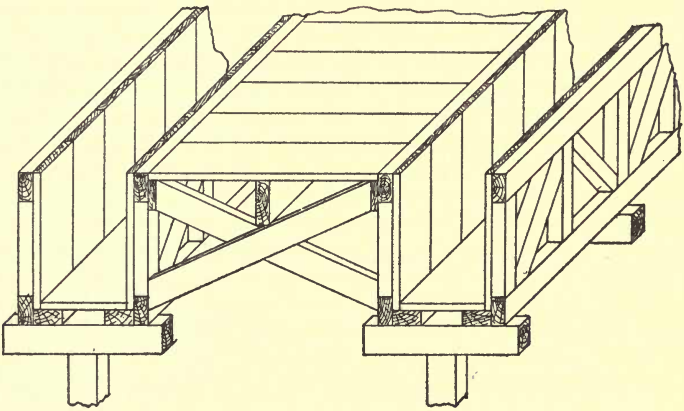


Fig. 22—Beam and Floor Forms, Illustrating Method of Trussing and Bracing to Save Posts Under Floors.

Simplicity in design should always be aimed at to save cost of forms. It is surprising how high the cost can be when the work is at all complicated. When walls are so thick that spouts can be arranged to deliver the concrete inside the reinforcement without disturbing it then high forms can be used. Sometimes as much as twenty feet can be poured and vertical joints instead of horizontal be obtained. In such walls braces may be preferable to wires or bolts.

When walls, however, are thin and the falling concrete cannot miss the steel, it is risky, to say the least, to pour more than three or four feet at the most, for the falling concrete may disarrange the steel. If high forms are used the pouring should not stop until the form is filled, even if a night crew is put on. If this is not attended to a great deal of concrete will adhere to the steel and to the sides of the forms and get dry. When pouring is resumed this dry stuff will be knocked off and falling into the fresh concrete will make a bad wall. Some of it will still cling to the steel and prevent a good adhesion of the newer concrete to the reinforcement.

In erecting forms, building lines must be carefully preserved by means of strings stretched between points previously accurately set by surveyor's instruments. Do not put any confidence in the ability of the average carpenter to carry walls vertically with a level. The plumbing of all the lines should be done by a few careful men and the rank and file in the form gang should be required to keep the lines fixed. Before pouring concrete all forms should be instrumentally tested for position and horizontal and vertical alignment. Warped forms should be rejected. During the pouring of the concrete there should be men at hand whose duty it shall be to notice when forms give way and rectify the trouble at once.

The finish of the forms depends upon the degree of finish called for in the specifications. If matched boards are not used it is a good plan to chamfer one edge of the boards so that the joint will close tightly when the wood is wet. If cracks open more than an eighth of an inch the forms should be rebuilt.

After all it is a question of price of lumber and of cost and efficiency of labor. When lumber is high in price small panels and shallow pourings will be the rule. When lumber is cheap deeper pourings will be had and higher panels. The man who ties himself to one standard is weak. Interchangeability is good and an endeavor should be made to standardize the work as much as possible. Each structure must be a special study.

The construction of a reinforced concrete structure is vastly different from that of a plain concrete structure. Reinforced concrete calls for a richer concrete than plain concrete work, as strength must be obtained. In many situations, however, especially in tank walls, mass and weight often accomplish as much as strength obtained by reinforcement. Where weight will do, a very cheap concrete can be used in heavy walls at a less cost than with thin reinforced walls.

CHAPTER VII.

The Conduct of Work.

The cement used for reinforced concrete work must be a Portland cement. It should be tested in order to insure uniformity. It should be used directly from the original packages as received from the factory. Batches should be of a size that do not call for parts of bags or barrels as no exact measurement can then be made. If the specifications do not state the bulk of cement to be used the packed barrel or packed bag must be used as the unit and the size of the unit must be determined by the engineer.

Common custom now makes four bags equal to one barrel of cement, and in bag batches one bag is considered equal to one cubic foot. This is about a mean between a packed foot and a moderately loosened foot of cement, and in the absence of a strict definition in the specifications is about all that can be insisted on. In this connection the writer received from Mr. Robert B. Hansell, C. E., of Baltimore, the following table. The sizes of cement barrels showed such variations that many tables appeared before the bags came into common use, based on barrels containing 3.5, 3.8 and 4.0 cubic feet. This table shows the number of cubic feet of each aggregate based on one cubic foot per bag.

TABLE X.

Cu. ft per barrel =	3.5			3.8			4.0		
Proportions	Bag	Sand cu. ft.	Stone cu. ft.	Bag	Sand cu. ft.	Stone cu. ft.	Bag	Sand ft.	Stone cu. ft.
1 : 3 : 5	1	10.5	17.5	1	11.4	19.0	1	12	20
1 : 2½ : 5	1	8.7	17.5	1	9.5	19.0	1	10	20
1 : 2 : 4	1	7.0	14.0	1	7.6	15.2	1	8	16

Two mixtures at present seem to be in general use for reinforced concrete work. For work requiring concrete of considerable density when the percentage of steel is high a 1:2:4 mixture is favored. Taking stone and sand as usually received, estimates for purchasing material can be made on the basis of six bags of cement, 0.4 cubic yards of sand and 0.8 cubic yards of stone for each cubic yard of concrete. For the greater amount of work done a 1:3:5 mixture is used. This will call for 4.5 bags of cement, half a yard of sand and 0.8 cubic yard of stone per cubic yard of concrete. Rules for proportioning concrete materials abound, but after all none are exact, for an exact determination of quantities depends upon the percentage of voids in the stone and sand. A rule credited to Mr. Fuller, used by many contractors, is to add the proportions together and divide 11 by the sum. For example the sum of 1:2:4 is 7. Eleven divided by 7 gives 1.58, the number of barrels of cement per cubic yard. The barrel of cement in this rule contains 3.8 cubic feet. Multiplying the two numbers together gives 1.58x3.8=6 bags of cement. For sand, 6x2=12 cubic feet, or 0.45 cubic yard, and for stone 6x4=24 cubic feet, or 0.9 cubic yard. This rule, it is understood, was obtained by taking the number of barrels of cement known to have been used on very large works, together with the number of cubic yards of sand and stone, and ascertaining the number of cubic yards of concrete made. It is necessarily very general, but allows liberally for waste and is safe enough for ordinary jobs. For close competition the voids should be ascertained and careful calculations made.

All concrete for reinforced work should be mixed by machine. This rule is imperative. If hand mixing is required in an emergency the inspector should pay the closest possible attention to it. It should be done carefully and with due deliberatoin. All hurry that smacks of haste should be avoided. The ordinary methods good enough for mass work cannot be tolerated. The sand and cement should first be mixed until the color is uniform and no streaks show. A minimum number of turns should be six after spreading the cement first carefully over the sand. Then the water should be added gently through a rose nozzle. A minimum number of turns during this operation is four and one set of turnings should not be considered sufficient to mix the sand and cement and at the same time apply the water. When the mixing is done the paste should be somewhat thick. The stone must be previously wetted and when the paste is ready thrown into it and the whole mass quickly turned until all the stone is covered with cement paste. Water will be added until the concrete is of the right degree of plasticity. For this final mixing with stone a minimum number of turns will be four.

No matter what the directions of the makers no batch of concrete should be given less than twenty turns in the mixer. If a mixer of the continuous delivery type is used the length of the trough should permit a number of turns equivalent to twenty turns per batch or should be so arranged that the mass can be held in place until it receives such turning. In continuous feed machines the cement is apt to clog so the cement feed should be closely looked after. Several tests should be made each day to see that the cement delivery is constant.

The writer wishes to state as a result of his own experience that it pays to equip a mixer with charging apparatus to reduce the number of men wheeling stone and sand. With a batch mixer it is well to have a super hopper containing one batch in reserve. In addition there should be an elevating hopper which can be loaded on the ground level by men and do away with inclines. While a batch is being mixed there is one in reserve in the super hopper and another being placed in the charging hopper. It will reduce the number of men required by two-thirds in charging alone. Charging hoppers generally have a steel diaphragm, on one side of which the sand is placed and on the other the stone. The measuring is thus to a certain extent automatic. When the hopper is discharged into the super hopper a fair mix is obtained. The cement can be added while the charge is in the super hopper and when the materials run into the mixing drum another fair mix is obtained. This tends to lessen the time required for actual mixing.

When materials are delivered by wheelbarrows the incline should be easy. When starting the work place the mixer as low as possible and send the material down through a chute to the bottom of the work. As the walls rise the chute can be shortened until the work reaches the height of the mixer. Then the mixer can be raised until the incline is as steep as men can work on. After that a hoisting elevator of some sort should be used to take the concrete to the elevation where wanted. A concrete hoist is a good investment if the concrete has to be elevated more than ten feet.

Measuring stone and sand by wheelbarrows is simply guessing and should not be tolerated. The illustration shows a form of box used by the writer. The length and widths given fit the average concrete wheelbarrow. The height of course depends upon the size of load wanted. The boxes are bottomless and are placed in the wheelbarrow and filled flush to the top. It is easy to lift them by the handles and the material drops out. By using such boxes exactness and uniformity can be attained in proportioning the ingredients. The ordinary wheelbarrow is said to hold three cubic feet. This means three cubic feet, water measure, when the wheelbarrow is held in such a position that the water exactly touches all the edges. If left to themselves the men will not load much more than two feet and seldom load with as much as two feet. When driven hard they will carry much more and the lack of uniformity is bad.

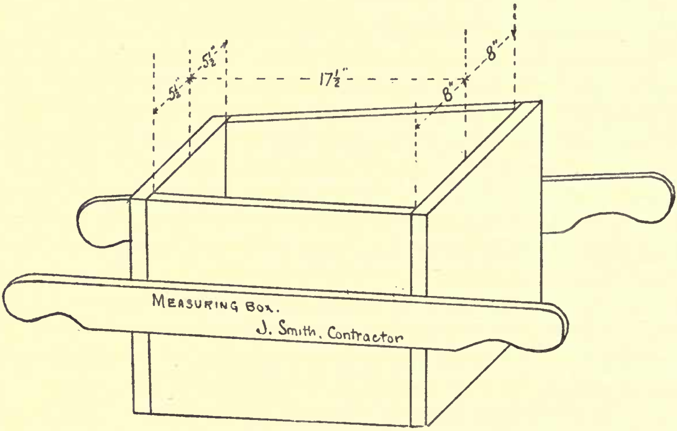


FIG. 23—MEASURING BOX.

Concrete must be wet enough to flow readily around and cover the steel, but must not be thin and watery. The ideal mixture is one that will not slip too readily from the wheelbarrow or cart and is best described by the word "pasty." It should be assisted out by a shovel.

Thin concrete is discharged more rapidly from the mixer and is consequently favored. The water, however, during the setting and seasoning process leaves the concrete porous if too thin when poured. Speed in operating a mixer depends largely upon the amount of water pressure available and it pays to insure a good pressure even if an elevated tank must be put up when the pressure in the city mains is low. Measuring the water is best done by an experienced man with good judgment. It simply requires training and such a man is better than the best automatic device ever invented, except of course for very large mixers. After a rain the sand and stone contain so much water that the amount used must vary with each batch. In dry summer weather more water must be used than in wet, cool weather.

The importance of using plenty of water in dry weather should be impressed upon workmen. This water should not all be added in the mixer. In hot weather the stone and sand pile should be wet, precisely as brick masons wet brick, and for the same reason. The stone may be of a kind that will absorb water rapidly and interfere with the proper setting of the cement next to it.

The concrete must be worked to the face of the forms to insure a good surface and must be well worked into all interstices to insure a good job. The methods used must be those calculated to best secure the desired results. Heavy tamping is not necessary with pasty concrete and is apt to displace the forms. What is wanted is something that will insure every piece of stone being thoroughly imbedded in the mass and also see that all the steel is surrounded and that none is displaced. All air must be released so far as that is possible. Churning with rods and pipes is good. A piece of one by three wood is as good as anything when the lower end is sharpened and fixed like a butter paddle.

The concrete must be worked back from the forms and many tools are used by different men—wooden paddles as above, potato forks and manure forks with curve taken out. There are several special concrete spades on the market also. The writer uses wood and also slicers and spades made with handles seven feet long of three-quarter-inch pipe, having on the end a piece of sheet steel one-eighth of an inch thick, ten inches long and with a width at the lower end of four, six and eight inches. That is, the three widths are used on the job. The narrower ones are very handy for cleaning forms. Churning with rods is so good that a number of men should be kept at it.

In pouring columns or filling forms exceeding three feet in height the directions given in the chapter on columns should be carefully followed.

To assist in obtaining a dense, impervious concrete for floors and roofs a wooden float about ten or twelve inches wide and not less than four feet long should be used as a tamper, two men continually tamping with same to consolidate the mass and help draw the cement to the surface. There has lately been placed on the market a tamper for floor surfaces consisting of flat bars arranged like a grating. These bars cut down into the concrete and do the work the stirring rods do in wall forms. The usual top coat and finish can be applied within the customary time.

Particular care must be exercised to see that steel is not displaced when pouring concrete. Occasional tapping or pounding on the forms as they are filled helps the flow of the concrete around the steel and prevents to some extent the lodging of stones in such positions that a rod or bar may be permanently displaced.

Unless prevented by accident, beams and girders should be poured with the floors, regardless of design. If designed as T sections this rule is imperative and no accident can be an excuse for not doing it. Beams and girders of T section are designed for economy. The economy is usually all on paper. The design of T beams has been discussed in Chapter I. There are times often when it is an absolute impossibility to pour the beams and slab at one operation and complete the work. To stop properly would require that a stop be placed in the center of the beam, running its entire length and going clear to the bottom. This makes in effect two narrow beams. Practically this is an impossibility owing to the presence of the reinforcement, so the work is stopped somewhere on the beam with a small ledge left on which to rest the beginning of slab work the next day. With the class of labor usually to be had one cannot always stop where theoretically he should, so there is always more or less of a question about the job. There are days when it makes no difference but the days occur so frequently when there is a difference, that one is safest in so designing that if considered advisable all the beams and girders may first be poured and the floor later.

The steel must be correctly placed and all the steel must go in. This is imperative. The work is designed with the steel calculated to occupy a certain definite position. To the extent that a bar or rod deviates from its proper position to that extent the work is defective. Wherever rods or bars cross they should be wired together securely. No. 18 wire is the best. The writer uses it cut into lengths of about eight inches and generally has the men use two of such pieces at each intersection. This has already been mentioned. Several parties have written to the writer saying it was not necessary to be so finicky and that the wiring was too expensive! The only reply possible to such critics is that the structure must be erected as designed and it is not possible to so erect it unless the steel is actually occupying the position intended. The writer has seen contractors placing steel in a hurry ahead of a concreting gang. He has seen it displaced and very irregular and confesses he does not like it. He has seen floors with steel all in, according to the superintendent, that called for twenty per cent more bars after they were regularly spaced and wired. Wiring is expensive, else experienced men would not use ready fabricated materials at the high prices charged for them. These fabrics sell readily because they do not cost much to place and when placed it is a great comfort to know that every strand is where the designer wanted it.

Lastly it must be taken into consideration that the height from which concrete can be poured depends to a great extent upon the security with which the steel is fastened together. So it must be wired to get it to the place it belongs and to keep it there during construction.

It is not difficult to place steel in walls and keep it the right distance from the face. With flat slabs however it is different. Some provision must be made to insure a flow of concrete under the bars. Some contractors have men ahead of the wheelbarrow gang hastily prying up the mat to allow the mortar to flow under. Others use small pieces of steel under the bars to hold them up, which are consequently imbedded and show on the underside. Others use small blocks of concrete broken from thin slabs made for the purpose. Some use regular concrete supports made for the rods used and others use a recently patented chair of thin steel which acts also as a clip and saves the expense of wiring. The writer has used them all and also uses a method which is about as satisfactory as any. He places lines of two by fours about four or five feet apart over the floor and suspends the reinforcing mat from them by wires. The supports for the timbers serve also as supports for running plank. Thin mortar is first poured to a depth of about an inch when the wires are cut and the steel drops into the mortar. The timbers and their supports are removed and the concrete rushed in on running plank laid on the steel.

Opinions and practice differ on the subject of the thin mortar coat placed on the floor, into which the steel is allowed to drop. It costs less and is quicker work when concrete is poured first along one side of the floor and the concrete after that is poured on the concrete so that the thin mortar flows away from it and under the steel. This makes it unnecessary to first mix batches of mortar, with the attendant annoyance and delay and going over the floor twice. The only objection to the method of pouring on the advancing edge and letting the mortar flow ahead under the steel is that the steel may not always drop as far as it should, so that some may be held nearer the neutral axis than is right. With an experienced crew and careful boss who does not rattle the men by hurrying them, any method is good, and this method of dispensing with the first coat of thin mortar is excellent. The floor forms should be soaked with water before pouring concrete.

Steel placed in beams and girders should be suspended in some way until mortar can be placed under and around it. In fact no harm is done if the beam is filled half way to the neutral axis in order to set and retain the steel in place. Care must be taken, however, to insure a good joint when the pouring is completed. If the beam is one that will show, such a proceeding is not possible for the line between the two pourings will be well defined.

Care must be taken that forms are secure, either by wiring, bolting or bracing.

The writer has computed some tables for his own use considering that concrete will exert a pressure equal to a fluid weighing eighty pounds per cubic foot.

The first table shows the pressure per square foot at different depths and the pressure on a strip one foot wide for the heights indicated. The second table gives the spacing vertically for wires and bolts and a comparison is afforded between wires of different gauges and bolts of different diameters. This gives the user an opportunity to take whatever the market can give. The horizontal distances are twenty-four inches, the vertical spacing alone varying. For example if a form six feet high is poured it will require twenty-five ties of No. 18 wire with intervals as shown, or ten wires of No. 14, or only three of No. 9. The wires to be doubled, as shown in drawing, and twisted.

TABLE XI.

PRESSURE OF FRESH "SOUPY" CONCRETE.

Depth in feet.	Pressure on Vertical Strip 1 Foot Wide.	Pressure on 1 Sq. Foot.	Depth in feet.	Pressure on Vertical Strip 1 Foot Wide.	Pressure on 1 Sq. Foot.	Depth in feet.	Pressure on Vertical Strip 1 Foot Wide.	Pressure on 1 Sq. Foot.
1	40	80	9	3,240	720	17	11.560	1,360
2	160	160	10	4,000	800	18	12,960	1,440
3	360	240	11	4,840	880	19	14,440	1,520
4	640	320	12	5,760	960	20	16,000	1,600
5	1,000	400	13	6,760	1,040	21	17,640	1,680
6	1,440	480	14	7,840	1,120	22	19,360	1,760
7	1,960	560	15	9,000	1,200	23	21,160	1,840
8	2,560	640	16	14,240	1,280	24	23,044	1,920

TABLE XII.

WIRE AND BOLT TABLE FOR CONCRETE.

Showing vertical intervals, measuring from the top, to secure forms. Horizontal spacing, two feet. Wires doubled.

SIZES OF BOLTS.				GAUGE OF WIRE.						
⅝"	½"	⅜"	¼"	8	9	10	12	14	16	18
ft.in.	ft.in.	ft.in.	ft.in.	ft.in.	ft.in.	ft.in.	ft.in.	ft.in.	ft.in.	ft.in.
8 6	6 6	5 0	3 6	3 6	3 6	3 0	2 0	2 0	1 0	1 0
12 6	9 6	7 6	5 0	5 6	5 0	4 0	3 0	2 8	2 0	1 7
15 0	12 0	9 0	6 0	6 6	6 0	5 0	4 0	3 4	2 6	2 0
17 6	14 0	10 6	7 0	7 6	7 0	6 0	5 0	4 0	3 0	2 4
19 6	16 0	11 9	8 0	8 6	8 0	7 0	5 6	4 4	3 4	2 7
21 6	17 6	13 0	8 6	9 6	9 0	8 0	6 0	4 8	3 8	2 10
23 3	18 8	14 0	9 0	10 3	9 6	8 6	6 6	5 0	4 0	3 0
24 6	19 10	14 9	9 6	11 0	10 0	9 0	7 0	5 4	4 3	3 3
....	21 0	15 6	10 0	11 9	10 6	9 6	7 6	5 8	4 6	3 6
....	22 0	16 3	10 6	12 6	11 0	10 0	8 0	6 0	4 9	3 9
....	23 0	17 0	11 0	13 0	11 6	10 6	8 4	6 4	5 0	3 11
....	24 0	17 9	11 6	13 6	12 0	11 0	8 8	6 8	5 3	4 0
....	25 0	18 6	12 0	14 0	12 6	11 6	9 0	7 0	5 6	4 2
....	....	19 3	12 6	14 6	13 0	12 0	9 4	7 3	5 8	4 4
....	....	20 0	13 0	15 0	13 6	12 6	9 8	7 6	5 10	4 6
....	....	20 8	13 6	15 6	14 0	13 0	10 0	7 9	6 0	4 8
....	....	21 4	14 0	16 0	14 6	13 4	10 4	8 0	6 2	4 10
....	....	22 0	14 5	16 6	15 0	13 8	10 8	8 3	6 4	4 11
....	....	22 6	14 10	17 0	15 6	14 0	11 0	8 6	6 6	5 0
....	....	23 0	15 3	17 4	16 0	14 4	11 4	8 8	6 8	5 2
....	....	23 6	15 8	17 8	16 4	14 8	11 8	8 10	6 10	5 4
....	....	24 0	16 0	18 0	16 8	15 0	12 0	9 0	7 0	5 6
....	....	24 6	16 4	18 4	17 0	15 4	12 3	9 3	7 4	5 8
....	....	25 0	16 8	18 8	17 4	15 8	12 6	9 6	7 7	5 10
....	....	....	17 0	19 0	17 8	16 0	12 9	9 8	7 9	5 11
....	....	....	17 4	19 4	18 0	....	13 0	9 10	7 11	6 0
....	....	....	17 8	19 8	....	....	....	10 0	8 0	....
Wt. per Foot of Rods, lbs.				Feet per Pound.						
1.043	.668	.376	.167	14.29	17.05	20.27	33.69	58.58	95.98	166.20

The third table gives the total load in pounds for columns, struts or braces having lengths varying by single feet. If the brace is placed at an angle of forty-five degrees the length will be 1.41 times the height from the ground to the top of the brace. For convenience in mentally estimating the length it is called 1.5 times the height. From the first table the pressure at any point can be ascertained and thus the load to be carried by the brace. Looking in the third table for braces of the required length the load can be found on the same line and at the top of the column in which this load is found will be found the size of brace required. This table is also useful for putting supporting posts and studding under floor forms. The loads are for ordinary white

TABLE XIII.

SAFE LOADS FOR POSTS AND BRACES (Common Pine).

Lengths in Feet.	Sizes in Inches.								
	2x4	2x6	2x8	3x4	4x4	4x6	6x6	6x8	8x8
	Load in pounds.								
1	....	....	....	....	....	....	....	....	....
2	4,800	7.200	9,600	....	....	....	....	....	....
3	4,136	5,904	8,272	7,200	....	....	....	....	....
4	3,648	5,472	7,296	6,048	9,600	14,400	....	....	....
5	3,360	5,040	6,720	5,760	8,160	12,240	....	....	....
6	3,072	4,608	6,144	5,472	7,872	11,808	21,000	28,800	....
7	2,784	4,176	5,568	5,184	7,584	11,376	18,576	24,768	....
8	2,496	3,744	4,992	4,896	7,296	10,944	18,144	24,192	....
9	2,208	3,312	4,416	4,608	7,008	10,512	17,712	23,616	33,400
10	1,920	2,880	3,840	4,320	6,720	10,080	17,280	23,040	33,280
11	1,632	2,448	3,264	4,032	6,432	9,648	16,848	22,464	32,768
12	1,344	2,016	2,688	3,744	6,144	9,216	16,416	21,888	32,256
13	1,056	1,584	2,112	3,456	5,856	8,784	15,984	21,312	31,744
14	768	1,152	1,536	3,168	5,568	8,352	15,552	20,736	31,232
15	480	720	960	2,880	5,280	7,920	15,120	20,160	30,720
16	192	288	384	2,592	4,992	7,488	14,688	19,584	30,208
17	....	....	....	2,304	4,704	7,056	14,256	19,008	29,696
18	....	....	....	2,016	4,416	6,624	13,824	18,432	29,184
19	....	....	....	1,728	4,128	6,192	13,392	17,856	28,672
20	....	....	....	1,440	3,840	5,760	12,960	17,280	28,160
21	....	....	....	1,152	3,552	5,328	12,528	16,704	27,648
22	....	....	....	864	3,264	4,896	12,096	16,128	27,136
23	....	....	....	576	2,976	4,464	11,664	15,552	26,624
24	....	....	....	288	2,688	4,032	11,232	14,976	26,112
25	....	....	....	....	2,400	3,600	10,800	14,400	25,600
26	....	....	....	....	2,112	3,168	10,368	13,824	25,088
27	....	....	....	....	1,824	2,736	9,936	13,248	24,576
28	....	....	....	....	1,536	2,304	9,504	12,672	24,064

TABLE XIV.

THICKNESS OF HORIZONTAL BOARDS FOR FORMS. VERTICAL STUDDING, 2x4 INCHES.

Thickness of Boards. inches.	Studding Intervals.		
	12"	18"	24"
	Height of Forms—Feet.		
1	8½	3	2½
1¼	14	6½	4
1½	21	10½	5½
1¾	28	12½	9
2	39	18½	12

It is assumed all boards are ⅛ in. less thickness than here given.

Studding is assumed to be sufficiently braced or tied.

As heights of form increase use thicker boards at bottom or set studding closer.

pine or spruce. For yellow pine (southern) the loads may be thirty per cent greater and for white oak twenty per cent.

The following table shows the spans that may be used for boards of different thickness. This is assuming that studding is used vertically and the boards are nailed or held in some way against them horizontally, the uprights being secured by wires, bolts or braces.

The following tables containing the strength of beams one inch thick and giving sizes of nails and spikes and weights of nut and bolt heads are from the Carnegie Pocket Book and will be useful in ordering material for forms.

TABLE XV.

SPIKES AND NAILS.

Standard Steel Wire Nails						Steel Wire Spikes			Common Iron Nails		
Sizes	Length Inches	Common		Finishing		Length Inches	Diam. Inches	No. per Pound	Sizes	Length Inches	No. per Pound
		Diam. Ins.	No. per lb.	Diam Ins.	No. per lb.						
2d	1	.0524	1060	.0453	1558	3	.1620	41	2d	1	800
3d	1¼	.0588	640	.0508	918	3½	.1819	30	3d	1¼	400
4d	1½	,0720	380	.0508	761	4	.2043	23	4d	1½	300
5d	1¾	.0764	275	.0571	500	4½	.2294	17	5d	1¾	200
6d	2	.0808	210	.0641	350	5	.2576	13	6d	2	150
7d	2¼	.0858	160	.0641	315	5½	.2893	11	7d	2¼	120
8d	2½	.0935	115	.0720	214	6	.2893	10	8d	2½	85
9d	2¾	.0963	93	.0720	195	6½	.2249	7½	9d	2¾	75
10d	3	.1082	77	.0808	137	7	.2249	7	10d	3	60
12d	3¼	.1144	60	.0808	127	8	.3648	5	12d	3¼	50
16d	3½	.1285	48	.0907	90	9	.3648	4½	16d	3½	40
20d	4	.1620	31	.1019	62	...	.....	....	20d	4	20
30d	4½	.1819	22	.....	....	...	.....	....	30d	4½	16
40d	5	.2043	17	.....	....	...	.....	....	40d	5	14
50d	5½	.2294	13	.....	....	...	.....	....	50d	5½	11
60d	6	.2576	11	.....	....	...	.....	....	60d	6	8

WROUGHT SPIKES.

Number to a Keg of 150 lbs.

Length Inches	¼ In. No.	5/16 In. No.	⅜ In. No.	Length Inches	¼ In. No.	5/16 In. No.	⅜ In. No.	7/16 In. No.	½ In. No.
3	2250	....	....	7	1161	662	482	445	306
3½	1890	1208	....	8	....	635	455	384	256
4	1650	1135	....	9	....	573	424	300	240
4½	1464	1064	....	10	....	...	391	270	222
5	1380	930	742	11	....	...	...	249	203
6	1292	868	570	12	....	...	...	236	180

—Carnegie Pocket Book.

TABLE XVI.

SAFE LOADS UNIFORMLY DISTRIBUTED FOR RECTANGULAR SPRUCE OR WHITE PINE BEAMS ONE INCH THICK.

The following table has been calculated for extreme fiber stresses of 750 lbs. per square inch corresponding to the following values for Moduli of Rupture recommended by Prof. Lanza, viz.:

Spruce and white pine....................3,000 lbs.
Oak ....4,000 lbs.
Yellow pine ....5,000 lbs.

For oak increase values in table by ⅓. For yellow pine increase values in table by ⅔.

The safe load for any other values per square inch is found by increasing or decreasing the loads given in the table in the same proportion as the increased or decreased fiber stress.

Span in Feet.	DEPTH OF BEAM—INCHES.										
	6	7	8	9	10	11	12	13	14	15	16
5	.600	820	1,070	1,350	1,670	2,020	2,400	2,820	3,270	3,750	4,270
6	.500	680	890	1,120	1,390	1,680	2,000	2,350	2,730	3,120	3,560
7	.430	580	760	960	1,190	1,440	1,710	2,010	2,330	2,680	3,050
8	.380	510	670	840	1,040	1,260	1,500	1,760	2,040	2,340	2,670
9	.330	460	590	750	930	1,120	1,330	1,560	1,810	2,080	2,370
10	.300	410	530	670	830	1,010	1,200	1,410	1,630	1,880	2,130
11	.270	370	490	610	760	920	1,090	1,280	1,490	1,710	1,940
12	.250	340	440	560	690	840	1,000	1,180	1,360	1,560	1,780
13	.230	310	410	520	640	780	930	1,080	1,260	1,440	1,640
14	.210	290	380	480	590	720	860	1,010	1,170	1,340	1,530
15	.200	270	360	450	560	670	800	940	1,090	1,250	1,420
16	.190	260	330	420	520	630	750	880	1,020	1,180	1,330
17	.180	240	310	400	490	590	710	830	960	1,100	1,260
18	.170	230	290	370	460	560	670	780	910	1,040	1,190
19	.160	210	280	360	440	530	630	740	860	990	1,130
20	.150	200	270	340	420	510	600	710	820	940	1,070
21	.140	190	260	320	390	480	570	670	780	890	1,020
22	.140	190	240	310	380	460	540	640	740	850	970
23	.130	180	230	290	360	440	520	610	710	810	920
24	.130	170	220	280	350	420	500	590	680	780	890
25	.120	160	210	270	330	410	480	560	660	750	860
26	.110	160	210	260	320	390	460	540	630	720	820
27	.110	150	200	250	310	370	440	520	610	690	790
28	.110	140	190	240	300	360	430	500	580	670	760
29	.110	140	180	230	290	350	410	490	560	640	740

To obtain the safe load for any thickness: Multiply values for 1 inch by thickness of beam.

To obtain the required thickness for any load: Divide by safe load for 1 inch.

—Carnegie Pocket Book.

TABLE XVII.

WEIGHT, IN POUNDS, OF 100 BOLTS WITH SQUARE HEADS AND NUTS.

Length Under Head to Point.	Diameter of Bolts—Inches.								
	¼	5/16	⅜	7/16	½	⅝	¾	⅞	1
1½	4.0	7.0	10.5	15.2	22.5	39.5	63.0	....	...
1¾	4.4	7.5	11.3	16.3	23.8	41.6	66.0	....	...
2	4.8	8.0	12.0	17.4	25.2	43.8	69.0	109.0	163
2¼	5.2	8.5	12.8	18.5	26.5	45.8	72.0	113.3	169
2½	5.5	9.0	13.5	19.6	27.8	48.0	75.0	117.5	174
2¾	5.8	9.5	14.3	20.7	29.1	50.1	78.0	121.8	180
3	6.3	10.0	15.0	21.8	30.5	52.3	81.0	126.0	185
3½	7.0	11.0	16.5	24.0	33.1	56.5	87.0	134.3	196
4	7.8	12.0	18.0	26.2	35.8	60.8	93.1	142.5	207
4½	8.5	13.0	19.5	28.4	38.4	65.0	99.1	151.0	218
5	9.3	14.0	21.0	30.6	41.1	69.3	105.2	159.6	229
5½	10.0	15.0	22.5	32.8	43.7	73.5	111.3	168.0	240
6	10.8	16.0	24.0	35.0	46.4	77.8	117.3	176.6	251
6½	....	...	25.5	37.2	49.0	82.0	123.4	185.0	262
7	....	...	27.0	39.4	51.7	86.3	129.4	193.7	273
7½	....	...	28.5	41.6	54.3	90.5	135.0	202.0	284
8	....	...	30.0	43.8	59.6	94.8	141.5	210.7	295
9	....	...	...	46.0	64.9	103.3	153.6	227.8	317
10	....	...	...	48.2	70.2	111.8	165.7	244.8	339
11	....	...	...	50.4	75.5	120.3	177.8	261.9	360
12	....	...	...	52.6	80.8	128.8	189.9	278.9	382
13	....	...	...	...	86.1	137.3	202.0	296.0	404
14	....	...	...	...	91.4	145.8	214.1	313.0	426
15	....	...	...	...	96.7	154.3	226.2	330.1	448
16	....	...	...	...	102.0	162.8	238.3	347.1	470
17	....	...	...	...	107.3	171.0	250.4	364.2	492
18	....	...	...	...	112.6	179.5	262.6	381.2	514
19	....	...	...	...	117.9	188.0	274.7	398.3	536
20	....	...	...	...	123.2	206.5	286.8	415.3	558
Per inch Additional.	1.4	2.1	3.1	4.2	5.5	8.5	12.3	16.7	21.8

WEIGHT OF NUTS AND BOLT-HEADS IN POUNDS.

For Calculating the Weight of Longer Bolts.

Diam of Bolt in Inches.		¼	⅜	½	⅝	¾	⅞
Weight of Hexagon Nut and Head	...	.017	.057	.128	.267	.43	.73
Weight of Square Nut and Head	...	.021	.069	.164	.320	.55	.88
Diam of Bolt in Inches.	1	1¼	1½	1¾	2	2½	3
Weight of Hexagon Nut and Head	1.10	2.14	3.78	5.6	8.75	17.0	28.8
Weight of Square Nut and Head	1.31	2.56	4.42	7.0	10.5	21.0	36.4

—Carnegie Pocket Book

At one end, or at both ends of all walls, there should be a hole in the form, the bottom of the hole being level with the top of the concrete already poured. This hole is to enable the space between forms to be thoroughly cleaned. It is impossible to raise the waste matter so the best way is to provide a hole through which to drive it. In the bottom of all column forms there should be a hole. In the bottom of all beam and girder forms there should be a generous sized hole. These holes are absolutely necessary to secure a good job and no great ingenuity is required to devise efficient methods for closing them. It does require watchfulness however to insure their being closed before pouring begins. For this reason a definite location for all such holes should be determined beforehand and adhered to. For example, in columns the holes may be on the north side and for walls in the middle of the space between cross walls and for beams at the north end and at the west end.

Forms must be so attended to that concrete cannot adhere to them and thus make a rough looking job. Wooden forms oiled should be thoroughly soaked in crude oil and each time they are used they should be again coated. The objections to oil are the expense, the weight of oil-soaked forms in handling, the liability of making the wall dirty, and the fact that if the walls require plastering or any subsequent treatment the oil skin must first be removed at great expense.

Soap is excellent. It should be dissolved in water to the consistency of very thin paste and is applied with brushes. The objections are that it is expensive and is seldom applied properly. The use of soap helps make a dense surface and does not interfere materially with subsequent treatment, provided the surface of the concrete is washed well with clean water before applying the treatment or plaster.

The cheapest and most satisfactory all round method for securing a good surface is to thoroughly soak the forms with water so they will not absorb water from the concrete. A sprinkling of the inside with a hose is not sufficient. See that the hose is applied to both sides thoroughly until the boards can take up no more water. When dry spots appear, wet them.

The surface obtained with oiled, soaped or wetted forms is practically identical. The great objection to water is that it is not always possible to thoroughly wet the forms and many times it is absolutely impossible to keep them wet without adding too much water to the concrete already poured. Hot summer days, for example, are poor days in which to use wetted boards unless there is a plentiful supply of water and it can be applied from the outside.

The following rules deserve to be printed in large type and pasted in the hats of all foremen or in places where all the men can read them:

See that all the steel goes in. Check closely.

See that all the steel is wired in position.

See that forms are well scraped and cleaned and properly "doped" before concrete is poured.

See that forms are in line and are plumb.

See that the proper space between forms is obtained.

See that all bracing or wiring is done.

See that all holes are closed and all cracks stopped up with soft clay well tamped into the spaces.

All joints should stop with a key. Never allow sloping joints. Vertical joints should be vertical and consist of a board placed across the form having on the side towards the concrete a key equal in width to one-half the thickness of the wall and projecting into the concrete a depth equal to at least one and one-half times the thickness of the key. It should be so shaped that it will admit of comparatively easy removal. This will make a groove in the end of the wall which will assist materially in bonding the new work to the old. A similar groove should be made in the top of all concrete when pouring is stopped and in this groove should be poured a water-tight compound of some sort if the walls are to be water tight, for it is at the joints most leaks occur.

In joining new work to old (meaning to concrete poured so long that it has alreday set), see that the old surface is scrubbed vigorously with a wire brush. The excess of water rising to the surface as concrete sets contains a great deal of partially set cement. It forms a muddy deposit on top and slowly sets and forms a scale hard to detach yet which prevents a perfect bond. It must be gotten rid of at any cost. Brushing with wire brushes is one method. A solution of ten per cent chemically pure hydrochloric acid in ninety parts of water has been used for years for plain concrete work. Its use in reinforced work is questionable on account of the acid having a tendency to go down the very small space between the steel and concrete, ultimately destroying the adhesion. When acid is used it must be washed off with clear water.

Compressed air has been used to clean shavings and sawdust out of forms with great success. The writer used on a piece of work he had charge of the past winter (1907-8) live steam at a pressure of considerably over one hundred pounds. He was so well pleased with results that hereafter the use of steam will be required in his specifications.

Nothing is so absolutely bad for joints as sawdust and nothing is so hard to get rid of. Shavings and blocks of wood are picked up with rag pickers' sticks, which are pieces of wood about one inch square having a sharpened nail driven into one end. Loose gravel, etc., can often be brushed out. Sawdust however remains. Even a strong stream of water fails to get rid of it. Live steam at a high pressure will however clean off the surface of the concrete to the bone. It removes all the half set and dead cement and all the sawdust. The writer also used this steam to clean his forms. It was directed against the forms until the concrete adhering to them softened, when it was scraped off readily.

Knock all loose scale from the steel before cleaning the space for another pouring. Clear the steel entirely of all concrete that may have become attached during the previous pouring. This is imperative. When this cleaning is done remove all the dirt and dust before erecting the next set of forms. After the next set is erected clean out the space with the rag pickers' rods and then with steam.

Before pouring see that the surface of the old concrete is thoroughly wet. Then give it a coat of neat cement applied preferably with a brush. After the neat cement wash has been applied, deposit an inch of cement mortar pretty wet, composed of one part of cement and two parts of sand. This should not be poured but should be thrown in with shovels, half a shovelful at a time. Then begin to deposit the concrete. If the joint is vertical and the mortar cannot be readily applied, use rods or spades or pieces of wood to work the mortar in the concrete up to the face of the joint and into the key. It is imperative that old concrete be well soaked with water before new concrete is joined to it.

Experiments lately made in France showed that practically perfect joints were made by first thoroughly wetting the old surface and then painting it with neat cement paste and thoroughly tamping a layer of concrete on top before proceeding to fill the forms. This tamping was good hard pounding, and this is a good thing to know, for joints are hard to make. Up to the present time perfectly satisfactory joints between old and new work were deemed to be a practical impossibility. The reinforcement may be in the way of getting the hard pounding in thin walls, but still some of the pounding may be secured.

When possible to precure the men, work should be carried on in two eight hour shifts for the concreting gang and three eight hour shifts for the form gang so that not more than eight hours will intervene between pourings. Sometimes the carpenters are worked on the two eight hour shifts having daylight and the laborers are worked on three shifts. The contractor should not be permitted to evade this method until an actual trial demonstrates the impossibility of doing it and getting a satisfactory job.

Work left over night must have the surface roughened before the concrete is entirely set. When forms are removed the walls must be at once attended to. If appearance is an object all projections must be removed by chiselling or bush hammering. All pitting must be brushed out with steel brushes until the surface is rough. It must then be soaked with clean water, painted with a neat cement wash and plastered with a one to two or one to three mortar floated with a wooden float to the level of the surrounding concrete. The finish should be a sand floated surface for this patching. Do not point up with a mortar richer than the mortar used in the matrix of the concrete, for cracks will appear around the edges of the patch, besides which the color will be different.

If the appearance of the work requires a coat of plaster, clean the surface with steam, afterward using wire brushes and then the steam again. Wet it with water, paint it with neat cement and immediately follow with two coats of one to three mortar, the lower coat scratched and the top coat wood floated to a sand surface.

For work done in cold weather the writer uses steam and hot water. He has steam pipes perforated with small holes running all through the sand pile and the steam continually escaping. It is well to have this done under the stone pile as well, using only the stone that has no ice on it and the sand that is not lumpy, for the lumps may be frozen sand. He has a steam coil in a water barrel to get the water almost to boiling heat and the steam pipe ends in the mixer drum, keeping it so hot that it is warm to the hand. A little salt added to the water also is used occasionally.

Sometimes long cylinders of sheet steel are laid on their side and over them are placed coarse screens. A fire of scrap form lumber is kept going in the cylinders and a stove pipe in the farther end insures a draught. The sand and stone are shovelled over the screens and lumps thus broken up, the materials falling down to the hot cylinder and keeping warm until used.

Hot concrete sets up quickly. If it obtains its initial set inside of thirty minutes add a little cold water while mixing, to reduce the temperature. Heat the steel and inside of the forms before pouring concrete. This is done best with steam, followed by hot water. It is a mistaken notion that hot water freezes more quickly than cold water.

Some European experiments seemed to show that hot concrete is much weaker than concrete mixed with cold water. Little has been said about it in America, but the general custom is to use hot water, and some men even heat the stone, believing that it will hold the heat a long time. The writer does not approve of this heating of stone and sand, but merely warms them enough to take the frost and ice out of the voids. The water he does not allow to reach the boiling point, and if the stones are not hot they will prevent the mixture from becoming warm enough to injure the cement.

Calcium chloride is the best material known to prevent concrete from freezing before it sets. About one pound per bag of cement seems to be sufficient and all tests made seem to show that the strength of the concrete is increased.

If possible it is a good idea to have a salamander in the room under the floor that is being poured to prevent the under side of the forms from getting too cold. Yet there is danger of the concrete baking in the forms, so it is well to have the under side continuously sprinkled with cold water until they do not feel warm to the hand.

Salamanders should be kept burning during the night in concreted rooms or tanks and in freezing weather all surfaces of concrete poured during the day should be covered with sawdust or straw. Old manure is heating but fresh manure is injurious to fresh concrete. Boards placed one inch above the surface of floors and covered with sacks or straw make a good covering.

CHAPTER VIII.

Tools.

When ordering steel for a piece of work it is usual to have the mills cut all the pieces to lengths, especially when there is no extra charge for such cutting.

It is almost impossible to keep exactly to the schedule, as men will make mistakes, especially when working with pieces varying by only a few inches, so it is better to cut some of the steel on the job. A good rule to follow is to have no pieces cut to lengths when less than twenty-five of any certain length are required. When less than that number may be wanted then order long pieces in multiples of the required lengths, exactly as a carpenter orders lumber.

Every job should have a shear for cutting steel into lengths required. Such shears cost from fifteen to one hundred and fifty dollars. If properly used a fifteen dollar shear is about all that is required. The writer has used on his work the Badger and the Edwards but knows that there are others as good, though those mentioned gave perfect satisfection. With them he has cut high carbon twisted steel three-quarters of an inch square. As a rule high carbon steel is easier to cut than medium steel. In using such shears the steel should be pushed well into the jaws and bear well against both, before the power is applied. If the jaws have to move a little before coming in contact with the bar there is danger of the shear being broken when the men throw their weights on to the handle. All the bolts on the shear should be tightly screwed up also. When the jaws are tight the best work is done.

An extremely useful tool on a reinforced concrete job is a combination shear arranged for cutting round, square and flat bars and also for punching holes in flats half an inch thick. Such a combination shear and punch costs between forty and fifty dollars.

The two following cuts illustrate the method the writer has adopted for keeping track of his steel.

STEEL ORDER FOR

No. of pces. ordered an Lengths

Sizes	6'	9'	11'	19'	22'	23'	2
1"		4				6	10
7/8"				8			
3/4"	210					50	
5/8"							
1/2"	200	150		109	198	54	
3/8"						500	
1/4"	400					1000	

STEEL RECORD

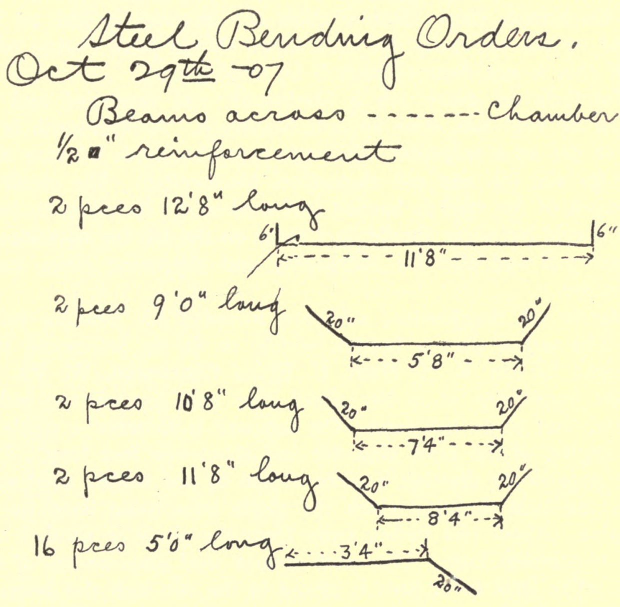
No. of 5/8" Bars. Lengths in feet

Dates	7 feet		13 feet		21 feet		30 feet	
	Rec'd	Used	Rec'd	Used	Rec'd	Used	Rec'd	Used
May 9	190		42		110		78	
June 1		11						
Aug 9		3		26		50		
Sept 14		100						
Nov. 1		76		16		60		78
Dec 3	10	10						

When he sends his order to the dealer he rules on a page in a book, as many columns as there are lengths ordered, as shown. On a vertical line he places the sizes. Then from his order sheet he copies the number of pieces in each column in lead pencil, As each car is received the number is checked off and when all the steel has come he writes the number of pieces in each column in red ink. About five per cent surplus should be ordered in half inch bars. Extra pieces are often wanted.

He also has a small blank book ruled for keeping track of each size and the lengths. This is the Steel Record. When the steel is received the proper entries are made in the column headed "Received" and as the steel is used the number of pieces is entered in the "Used" column. It is thus an easy matter at all times to know exactly how many pieces of a certain size and length there should be on hand.

The following cut shows how orders for steel are given to the steel boss:



One man has charge of the steel yard. To him all orders are given for steel to be delivered in the building. He has charge of the steel bending gang also. In the book where the steel record is kept all orders for bending or delivery of steel are entered. The book is turned upside down and entries made in what was the back. The writer generally uses a quadrille ruled book so front and back are merely relative terms. The accompanying illustration is a copy of one page in this book. The steel boss has a small leather bound book that he can carry in his pocket. When an order is given it is first entered fully in the office book and entered in the "Used" column of the Steel Record. Then the bending order is copied into the steel boss's book in full, date also. He bends the steel and piles it in a convenient place until wanted.

When there is plenty of room the most convenient way to take care of the steel is to put it in piles on the ground with posts alongside telling the size and length. If there is not much room then a steel rack must be made. These racks are simple and do not require description. Any intelligent carpenter should be able to make one after inspecting the store room of a hardware dealer or of a blacksmith shop. But whether the material is laid on the ground or put on racks one man alone should have charge of it and if this is not done there will be trouble.

Formerly on every job a blacksmith was employed and all bending of steel was done by first heating it. A blacksmith is as hard to get as a carpenter, although many hundreds of men may apply for the job and call themselves blacksmiths. The result is that much of the steel is burned and weakened. It is best to bend all of it cold but there should be a forge on the job for occasionally heating steel and also for sharpening and tempering picks, gads, chisels, etc.

Nearly all the benders sold are intended for bending metal hot. The writer uses a bender, which, so far as he knows, was developed by Mr. R. S. Hunt, C. E., of Charleston, W. Va. It consists essentially of a bar about one and one-quarter inches diameter and three feet long. On one end is fastened a pintle at right angles, of the same diameter and about one foot long. This pintle goes down through a piece of ten by ten timber used as a bench, the hole through which the pintle goes being lined by an iron pipe. About two inches away from the pintle is another one about two inches long. This rests upon a steel plate on top of the bench. The steel to be bent is placed in the two-inch slot between the two projections and men turn the bender round. The writer has bent high carbon steel cold, one inch square, and medium steel one and one-quarter inches square, cold, with such a bender. Sometimes the smaller projection has an anti-friction roller on it.

Sometimes it is necessary to bend a small piece of steel with a shoulder almost square and this is done by placing a steel pin in a hole in the plate a short space away from the large pintle.

The hot bender is a small bender that can be attached to an anvil (which should be on the job), for bending rods that have been heated or for bending very small rods and bars cold. It is remarkably efficient. It consists first of a round piece of iron about

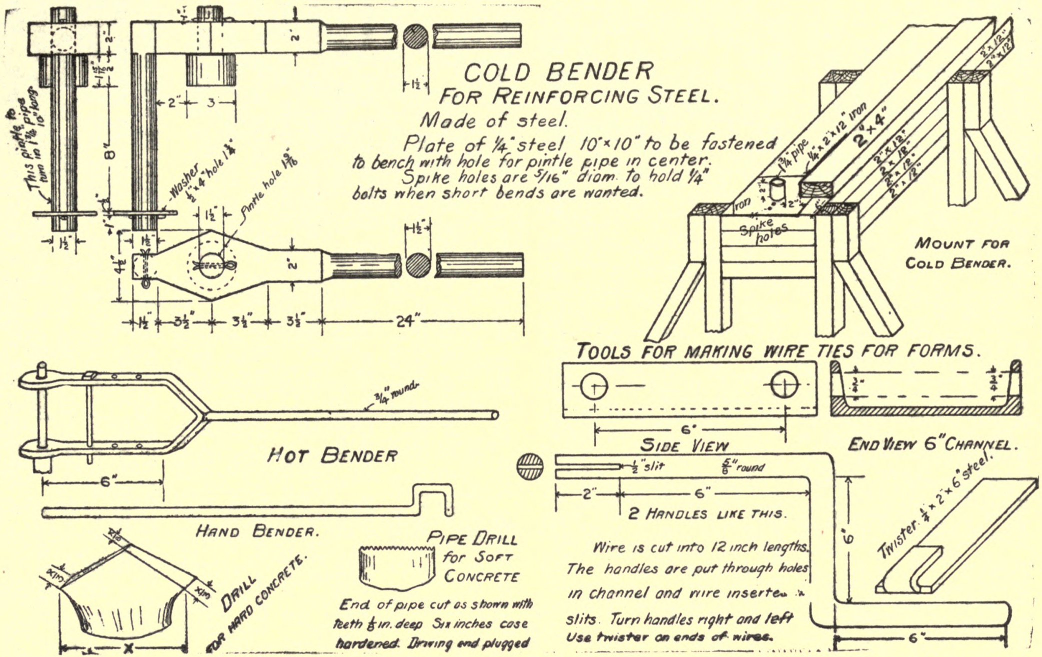


FIG. 24—USEFUL TOOLS FOR CONCRETE WORK.

five inches long with a square extension of about the same length to fit into the hole on the anvil. A piece of three-quarter inch round steel about four feet long is split at one end and the pieces formed into a yoke about eight inches long. The ends have holes to fit over the upright round piece sticking up from the anvil. Along the sides of the yoke are holes to hold pins which shorten the space according to the size of the rod to be bent. When the heated rod is passed through the yoke between the upright standard and the pin run through the sides of the yoke the blacksmith turns the bender so the bar is bent to any required degree. There should be a cold bender and a hot bender on every job.

Hand benders are readily made by bending some pieces of the reinforcing steel on the job. Three-quarter inch bars are best, although sometimes something heavier is needed. The bar is first bent at right angles. Then the short end is bent again parallel with the main piece and about an inch and a half away from it. Then at a distance of an inch it is bent again toward the main piece thus forming a letter U at right angles to the bar. Small pipes are used satisfactorily as hand benders in many cases but there is nothing really as satisfactory as a twenty-one inch monkey wrench.

Occasionally rods and bars have to be cut in situations where it is impossible to use the shear, so for such work cold chisels must be used and a plentiful supply should be on hand. They are made of tool steel properly hardened by a competent metal worker. The edge is held on the steel and a hand hammer used for striking. Cold cutters have longer edges than cold chisels and are fastened to a handle like a hammer. By means of this handle the cold cutter is held in place by one man while another does the striking with a sledge hammer. Cold chisels and cold cutters are useful and occasionally necessary but they are very expensive tools to use. While they should be on the job it is seldom any use will be had for them if the shears are properly handled. As the edges go fast a number must be handy when needed.

Bolt cutters are scissors-like tools having jaws of well tempered steel and powerful levers in the jaws. They come in several sizes and as they are intended for cutting iron should not be used for cutting steel bars although some are large enough to cut a half inch bolt readily. The best size to use on a reinforced concrete job is the No. 0, intended for cutting a quarter inch bolt. Many uses will be found for bolt cutters as considerable wire is used. There should be two or three on an ordinary job.

A pair of tinners' snips are very handy and also a few pieces of sheet iron for making a smooth job of patching a hole in forms.

The handiest tool for cutting, tying and bending the small wire (No. 16 or No. 18) used for fastening steel is a ten-inch blacksmith's cutting nipper. It is better than a flat plier for bending the wire and the end jaw is good for cutting close to the surface when wires project. These nippers can be used for cutting wires as heavy as No. 10, but to do this requires a twist of the wrist few ignorant unskilled laborers can acquire, and the edges of the cutting nippers are broken, rendering the tool useless for anything but bending.

Flat pliers do well for bending but there should be a cutting edge. Some of the many fence tools on the market are good. They combine pliers, wire cutters, staple pullers and hammers. They are really very efficient but for the fact that the cutting edges are on the side, whereas an end cutter is most convenient. The edges of all cutters break quickly when used by the average laborer as they will twist the wrong way in spite of all instruction. The furnishing of pliers and cutting nippers is expensive. It is really a good plan to have a few men do all wiring and make them furnish their own pliers, etc.

Every job should have at least two crowbars. The writer generally has a number of pieces of the reinforcing steel sharpened on the end with chisel-like edges. Such bars can be used as pinch or crowbars and as slicing or cutting bars in breaking out keys or chipping concrete.

Some small bars with claws on the end for removing forms and drawing nails are useful. They can all be used for reinforcement toward the end of the job. A number of chisels for cutting concrete and a number of steel gads are also needed.

A couple of 4 or 6 pound sledge hammers on a job will pay for themselves many times over. A couple of post mauls come in handy and a post auger is a very convenient and sometimes necessary tool.

Forms give way suddenly and occasionally there are heavy tools or pieces of machinery to raise, so there should be about four bottle or house-raising jacks on the job. It is well to have also a good rope about two hundred feet long, one inch in diameter, with a set of triple blocks. There should be several half or three-eighths ropes and some hooks, which can be made from pieces of reinforcing steel, for raising forms and cleaning up. A timber saw (cross cut) and one or two twenty-six inch hand saws, a couple of hammers, two hand axes and a couple of pole axes should be included in the outfit. The carpenters furnish their own tools of course, but these tools are for the use of men on odd jobs at which a carpenter would not be put. Two or three plumb bobs, a square, good level and several hundred feet of chalk line (not forgetting balls of chalk) and some marking crayons and pencils must not be forgotten. For marking steel for cutting and bending, soapstone crayon used by machinists is best.

When power is obtainable, a circular saw for cutting and ripping lumber for forms is a good investment. A great deal of sawing must be done on every job and a circular saw will soon pay for itself. The writer knows some contractors who have planers and buy all their lumber in the rough, thus saving one dollar and a half per thousand in first cost and also being sure of getting the stuff when in a hurry. Surfaced lumber sometimes demands a day or so extra time to furnish.

Several nail pullers are a necessity. There should also be a good grindstone on the job. A boring machine is needed if there will be many holes to bore, as for wires and bolts through forms. Boring such holes with brace and bit is slow, expensive work and the holes are not always exact.

On a reinforced concrete job, big bills for work at machine shops run up with alarming frequency. For this reason it is well to have the forge and anvil mentioned and in addition there should be a bench vice, pipe cutter, stocks and dies for threading pipes and taps and dies for bolts and rods. These articles are cheap and oftentimes one piece of work will pay for the entire outfit.

Some concrete mixers admit of loading directly into wheelbarrows and with such mixers no loading box is required. When the work has reached a height where a concrete hoist is put in operation there should be a box into which to dump the concrete and from which to load the wheelbarrows or carts. Boxes and chutes of steel for this purpose are made by a number of firms and it pays to buy them when over five hundred yards must be handled. However, they may be made on the job, the box of wood and the gate of sheet steel. While the home made ones work all right, when they work, they are likely to break and as every man of experience knows, breaks generally occur when the work can be prosecuted advantageously and the repairs are just completed when the weather changes. The hope of saving a few dollars should not influence the contractor in the making of tools unless he can make them as well as those he can buy.

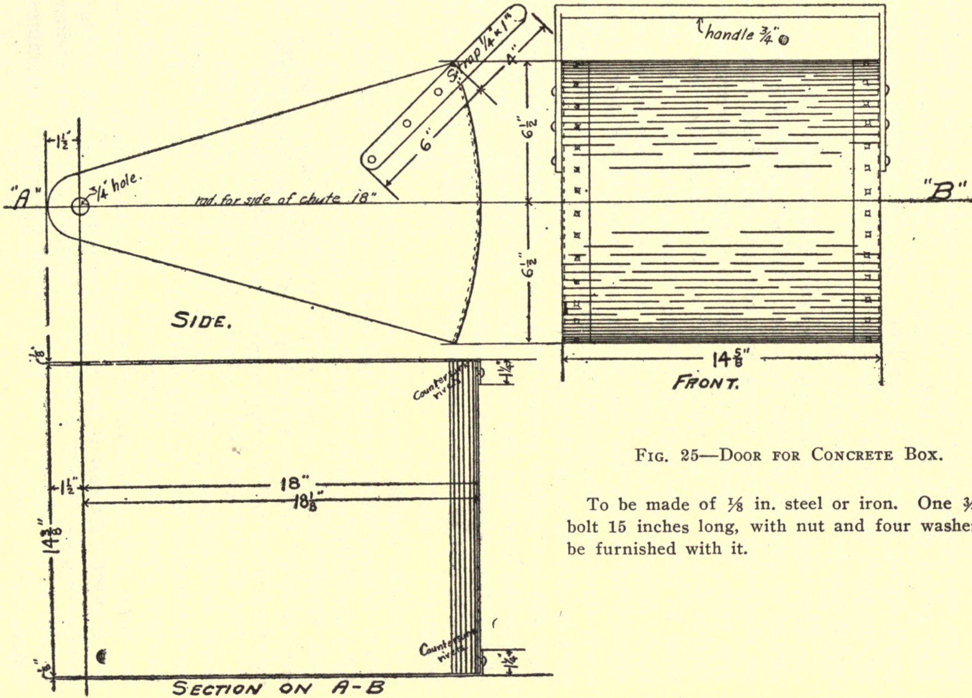


Fig. 25—Door for Concrete Box.

To be made of ⅛ in. steel or iron. One ¾ in. bolt 15 inches long, with nut and four washers to be furnished with it.

The writer has several times used wooden boxes for hoisting concrete with a horsepower hoist. It is very efficient and lessens the cost of handling concrete amazingly. Yet one or two careless drops may make such a box much more expensive than a steel box, high priced though the latter may be. The horsepower hoist in small towns, however, is excellent. The cost is about one-fourth or one-sixth the cost of a gasoline or steam hoist of equal capacity and it may be required only a few days.

The ideal power for all purposes is electricity. One motor cannot be used on all jobs, however. When a contractor possesses an alternating current motor and can get only direct current, or vice versa, he expresses violent opinions regarding electricity as a motive power. If alternating current is supplied then his motor must be suitable as regards voltage, phase and cycle. The differences found in power plants may require a contractor to own a number of motors for one mixer and hoist if his work covers much territory. Assuming, however, that his motor can be used he has only to keep the apparatus well protected from rain and dust and not neglect proper lubrication. To throw the switch is simple and any man can do it after one showing. The power is always there when wanted. The absence of dirt and grease and the saving in cost of skilled attendance count on the right side of the ledger.

Gasoline engines come next in convenience to electricity. The power is there when wanted and it is no trouble to start and stop. Like electricity, no skilled attendant is necessary. Simply an intelligent careful man is needed. The trouble, however, lies in the engines sold. Contractors seldom obtain good gasoline engines for they look alike and the lowest priced are seldom the best.

The best of gasoline engines give trouble occasionally. The secret in operating gasoline engines lies in attending properly to the lubrication and in keeping them absolutely clean. That is, provided a good engine is purchased. A gasoline engine sold at a low price and not guaranteed by a responsible firm is a delusion and a snare. As cost is not the proper criterion, a man buying a gasoline engine should employ a competent mechanical engineer to buy it for him or else pay a good price for a guaranteed engine.

With electricity or with internal combustion motors the power when on is paid for. When not used the expense ceases. Steam engines, however, require that steam be kept up ready for use when wanted. This means expense for fuel and also the wages of a high priced attendant. The engineer must be licensed in most states and is generally a union man. When an engineer is employed he does not want to do any other work, even on a small job with a small mixer. If the mixer is a fairly good sized one he wants a helper to act as fireman. Therefore steam engines are not very satisfactory on jobs where the concreting is not continuous. Conditions do not always permit of coal or other fuel being stored or handled on the job.

The foregoing remarks on motive powers apply especially to summer work. Cleaning between forms, especially between reinforcing steel set in the forms, is almost impossible with brooms, and washing with water is not always feasible because the required pressure can not be had. Compressed air is most excellent and when electricity or gasoline or oil may be used for power there should be an air compressor capable of furnishing plenty of air at not less than eighty pounds pressure, to blow out dust, sawdust, etc., and clean the forms. As there is a thin film formed on top of all concrete which prevents a

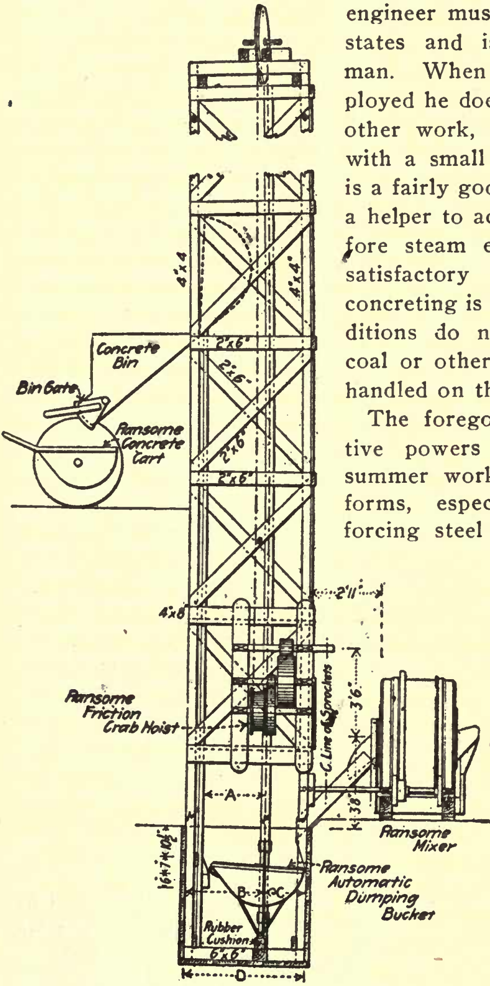


Fig. 26—Ransome Concrete Hoist.

good bond, the compressed air apparatus should be provided with a sand blast to roughen the surface.

The writer has found compressed air excellent. He has found steam better. In winter steam is the ideal power for it is capable of so many uses. The boiler should be large and situated at a convenient point, which is seldom one close to the mixer. One pipe can lead to the engine on the mixer and a smaller pipe can be led in a coil through a barrel of mixing water and end in the drum, which will be kept full of live steam. Another line can be led to coils under the piles of sand and stone and a line can be led to the building, to which a steam hose may be attached for blowing out the forms and cutting the skin off the top of the concrete. The pressure should be not less than one hundred pounds. When the number of uses to which steam can be put are considered it may be termed the ideal thing to use in cold weather while electricity and gasoline or oil are better in warm weather.

The loss by condensation of steam is great in long pipes, no matter how well covered. They should be covered and if the condensation is considerable some simple form of separator should be used close to the engine, or cylinders will be damaged. There are a number of excellent coverings in the market, but generally expensive. The writer reduced condensation once about fifty per cent by enclosing a long pipe in a box six inches square with a cover that would shed water. Around the pipe was wrapped two thicknesses of asbestos paper and the box was filled with plasterers hair, so the pipe had no less than an inch of this hair surrounding it. Another good covering is composed of flour paste and sawdust. This is placed in a box surrounding the pipe, the heat from which bakes the paste, forming the air spaces so necessary for insulation.

Wheelbarrows of course are the usual conveyances for the delivery of concrete. They may continue to be so for delivery of material to the mixer, but many forms of cars and buckets are often superior for delivery of concrete.

The size of wheelbarrows has already been mentioned. The most convenient and least wasteful wheelbarrows made for concrete work have the bed raised two or three inches in front. This increases the capacity and prevents slopping. Such wheelbarrows are a regular article of manufacture and can be purchased almost everywhere. The No. 2 size is probably best as it is light and lasts, in proportion to price, as well as any made. If the local store does not handle them the ordinary pressed bowl barrow mounted on a wooden frame can be readily altered by cutting down the

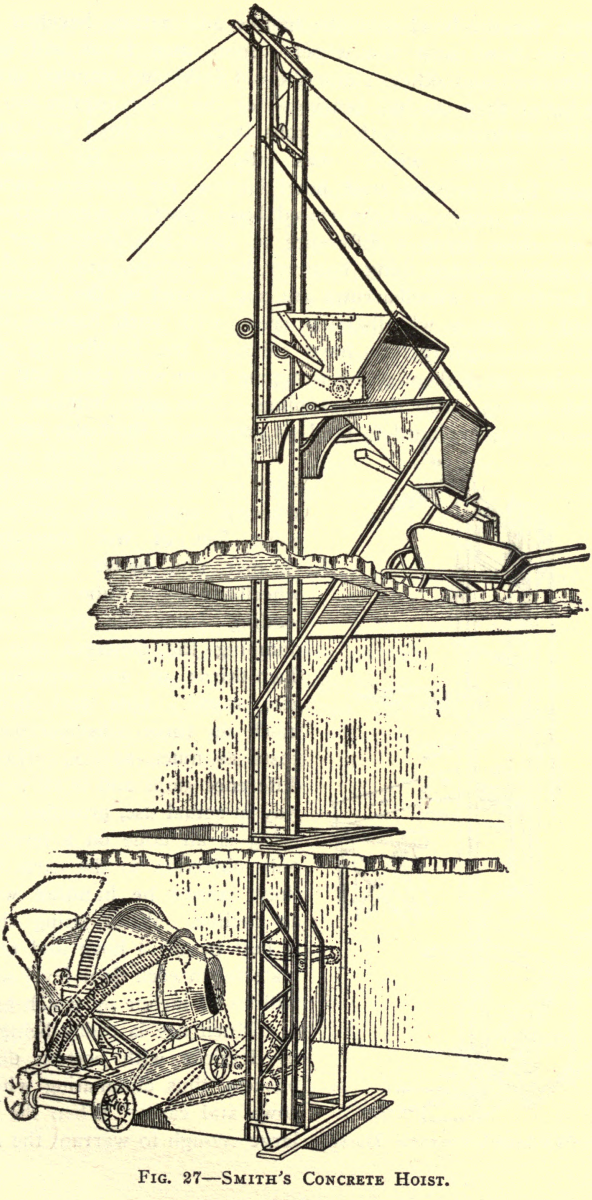


Fig. 27—Smith's Concrete Hoist.

supports for the bowl near the handle and putting bevelled pieces under the bowl near the wheel. Some men favor ball bearing wheelbarrows and others claim the old fashioned stapeled squeaker to be better than the ball bearings, for the latter require too much oil. The writer used both on jobs lately completed and was not able to decide which was the better. He uses the ordinary light pressed steel bowl barrow for carrying sand and stone to the mixer and the raised bowl concrete wheelbarrow for concrete, there being a difference of about five dollars per dozen in the price. Of one thing, however, he is certain, and it is that gas pipe handles on wheelbarrows are not favored by the laborers. It is hard to get a wheelbarrow crew, with such handles on the work. Some contractors have enlarged the handles by putting rubber hose over them or by wrapping them with cloth and sewing painted canvas over the wrappings. The pipe handles without some covering are hard to hold on account of their size and shape.

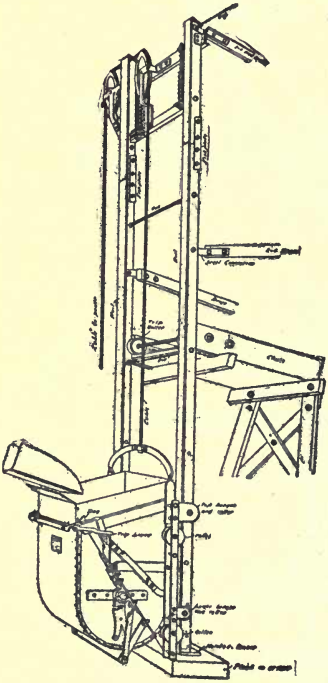


Fig. 28—Wallace-Lindsmith Hoist.

When running plank and scaffolds can be arranged properly two-wheeled carts, each holding six cubic feet of wet concrete, are cheaper and better to use than wheelbarrows. Time is saved in loading from the mixer and in dumping the concrete into place. Two-thirds of the wheelers are also saved. One cart holds as much as three wheelbarrows, but costs as much as six. There the difference stops and is all the other way in actual use, provided the carrying is all done on a level. The only objection to the carts is that they cannot be handled well on an incline. When using them it is best to use a hoist and take the material to each floor level so there will be no incline up which to push.

Much use is today being made of derricks with bottom dumping buckets, thus eliminating wheelbarrows and carts. When the job is large enough to warrant the cost of installation some such method should always be used. It can be used to carry stone and sand and cement from stock piles to the mixer and to carry the mixed concrete from the mixer to the forms. The writer wishes, however, to go on record as being opposed to the use of bottom dumping buckets for reinforced concrete walls less than eighteen inches thick.

Another method is to have cable ways from the mixer, and one system contemplates the use of cable ways arranged on the cash carrier system over all the walls, so the concrete can be delivered at any point. It also involves a similar arrangement for stone, sand and cement to the mixer.

Another system provides belt conveyors running alongside the walls, with adjustable scrapers to throw the concrete off to one side and into the forms. There is also a system consisting of semi-circular troughs having the bottom made in detachable sections a couple or three feet long. The trough contains screw conveyors supplied with universal joints. The bottom is one section shorter than the frame and can be shifted to leave a hole through which concrete will drop when carried to the point by the conveyor.

Before starting a piece of work the man in charge should obtain all the literature he can on the subject of conveying machinery for contractor's use. Some men have too much machinery and some do not have enough. It pays to be well equipped, but it does not do to go to too great an expense for one job. If a man, however, does not wish to spend enough money to prosecute the work properly, he should sublet it rather than try to do it himself.

INDEX

Adhesion 10, 66
formula 24
length of steel to secure... 32
Air, compressed................108
Amount of water............... 96
Angle steel in columns......... 50
Areas and weights of steel..... 27
Arm, moment, defined.......... 15
Axes 118
Axis, neutral, defined.......... 10
formula 13
tables 14-15

Bag and barrel of cement....... 93
Bars, areas and weights........ 27
bending of 26
deformed 18, 27, 67
lap 26
length for adhesion........ 10
patented 17
spacing in beams........... 10
Base of walls.................. 55
Beams, continuous............. 34
design of...............14, 42
double reinforced.......... 28
failures of..............21, 24
forms for.................. 91
formulas 9
loads on................36, 43
proportions of........9, 10, 15
spacing of bars in......... 10
of T section............... 30
table of wooden............104
Benders for steel..............114
Bending bars.................. 26
steel 113
Blacksmith 114
Board by board forms.......... 77
Boards, thickness for forms....102
Bolt cutters....................116
holes in walls.............. 81
Bolts in forms................. 79
table of..............101, 105
Bond formula.................. 24
Bottle jacks....................117
Boxes for concrete.............118
for measuring............. 95
Braced forms.................. 78
Braces and posts, loads on......102
Breadth, proportion............ 9
of beam.................. 15
Bridges 43
Buckling of steel.............. 30
Built-up columns.............. 45

Cableways 124
Calcium chloride.............. 110
Cantilever beams.............. 33
Carbon, high, steel............ 13
Carpenters 70, 77
Carts 124
Cast-iron, strength of.......... 10
Cement measuring............. 93
Chalk line.....................118
Cheap labor.................... 71
Chicago ordinance...........14, 43
Chimney design................ 61
Cinder concrete, strength of.... 20

Circular tanks................. 59
Cleaning forms.................106
Coated steel................... 68
Cold chisels....................116
Cold cutters....................116
Cold weather...................109
Column, various materials...... 44
beam acting as............. 9
divisors 47
forms 91
formulas 48
footings 51, 63
stresses 45, 46
Compressed air............108, 122
Compression, concentrated...... 13
failure 24
formula 22
illustrated 11
steel 30
Compressive strength of concrete 11
Concrete, below steel........... 9
columns 44
joints 107
protected columns.......... 45
protection 10
quantities 94
stress 22
walls 52
weight of.................. 21
Concentrated loads...........36, 43
Condensation of steam.........122
Connections 32
screwed 68
Considere's tests............... 64
Continuous beams and slabs.... 34
Conveying machinery...........124
Cast-iron columns 44
Cost data...................... 69
estimates 73
of forms.................. 75
of lumber................. 77
of placing steel............ 28
of steel.................... 18
of walls................... 58
Counterforts 57
Cracks in beams............... 64
Crayon 118
Crowbars 117
Cutters, for steel..........111, 116
Cutting steel...................111

Data on costs.................. 70
Dead load..................... 43
on columns................ 45
Deflection, defined............. 35
of beams................21, 65
Deformation, unit.............. 12
Deformed bars..............18, 67
when to use............... 27
Dense concrete................. 97
Design of beams............... 14
Designing forms................ 89
Depth of beam, definition....... 9
proportions 9
Depth of beam.................. 15
Diagonal tension failure........ 24
defined 36

Distributed loads............36, 43
Distribution of steel........... 34
Divisors, for column design..... 47
Double reinforced beams........ 28
Drawing of wire............... 17

Earth pressure................ 53
weight of................ 56
Elastic limit................... 17
Elasticity, Modulus of........13, 20
Electricity 120
Elevators 95
Estimating costs............... 73
Expanded fabric............... 27

Fabrics for reinforcement....... 27
Failures of beams.............. 21
of **walls** 52
Fence tools...................117
Fibre stress.................. 17
Finishing work.................109
Finish of forms............... 92
Fire protection............... 10
Floor design.................. 42
loads 43
slabs 33
space, value of............ 44
Footings for columns........... 51
forms for................ 90
for walls, etc............ 63
Forces in beams............... 12
Forges 118
Forms, boards for..............102
cost of.................. 75
designs for............... 89
per cubic yard............ 75
Framed forms................. 77

Galvanized steel............... 68
Gasoline engines...............120
Girder forms.................. 91
Gravel, pressure of............ 53
Greased bolts................. 81
Grip of concrete.............10, 66

Hammers 117
Hand mixing.................. 94
Heating material...............109
Heaving of walls.............. 56
High carbon steel...........13, 18
Highway bridges............... 43
Hoists 95, 120
Holders for planks............. 89
Holes, bolt, in walls............ 81
in forms..................106
in slabs.................. 68
Hot concrete..................110

Impervious concrete............ 97
Internal stresses............... 15
Iron, cast, strength of.......... 10
columns 44

Jacks, house raising............117
Joints in concrete............. 107
screwed 68

K, moment factor.............. 9
Labor, ideas on................ 70
Lap of bars.............26, 68, 82

Length and thickness of columns 45
Limit, elastic, defined........... 17
Line, chalk....................118
Lining forms.................. 92
Live loads.................... 43
on columns............... 45
Loads, allowed................ 43
on beams..............36, 43
on columns............... 45
dead and alive............ 43
on posts and braces........102
on supports............... 34
Longitudinally reinforced columns 44
Low cost labor................ 71

Machine mixing................ 94
Manure 110
Material for forms............. 77
Measuring, cement............. 93
aggregates 95
Medium steel................13, 17
Mixing concrete............... 94
Modulus of elasticity.....13, 20, 65
Moment, defined.............. 35
arm 15
factor 9
formula 16
table 16
of resistance.............. 9
formula 22
Mortar around steel............ 99

Nailing forms................. 78
Nail pullers..............117, 118
Nails and spikes.............. 103
Neutral axis.................. 10
formula 13
table 14, 15
Night work....................109

Oiled forms...................106
Ordering steel.................111
Ordinances, Chicago............ 14
St. Louis.................. 43
fixing stresses............. 31
governing loads............ 43

Painted steel.................. 68
Panel forms................... 83
Parabolic formulas............. 21
Parallel forces................ 40
Patented bars................. 17
Percentage of steel.......13, 14, 18
in columns................ 47
Pinch bars....................117
Pipe cutters...................118
Placing steel................28, 98
Placing steel.................. 98
Plank holders................. 87
Plastering 109
Pliers 117
Plumbing forms................ 92
Posts and braces, loads on.....102
Pouring columns............... 51
concrete 92, 97, 98
Pressure, circular tanks......... 59
of concrete...............100
different materials......... 53
against walls............... 53

Proportions for concrete........ 94
of beams.................9, 10
of walls.................... 56
Protection against fire.......... 10
of steel...................9, 10
Protected concrete columns..... 45

Quantities of aggregates........ 94

Racks for steel.................114
Ratio of modulus of elasticity.. 21
stress 29
Reaction defined............... 35
Record of steel................111
Reinforcement, double.......... 28
over supports.............. 32
Resisting moment.............. 9
Restrained walls............... 58
Retaining walls................ 52
Rock concrete, strength......... 20
Rods, areas and weights........ 27
deformed 67
Roof slabs..................... 33
Round rods..................... 67
Rules for foremen.............107
Rust 108

St. Louis Ordinance........... 43
Safety, factor of...........19, 65
Salamanders 110
Sand pressures................. 53
Saw dust......................110
Saws 118
Scale on steel.................108
Screwed connections.........68, 32
Shear 24, 25, 36, 43
Shears for steel...............111
Shrinkage, in columns......... 51
of concrete................ 66
Slabs, continuous.............. 34
designing 33, 42
Sidewalks 43
Snips, tinners.................117
Soap 106
Spacers between bars.....32, 10, 67
in walls................... 81
Span, proportionate........... 9
Spikes and nails...............103
Spirally wound columns........ 45
Square bars.................... 67
Steam 108, 122
Steel columns................. 44
in compression............. 30
cost of.................... 18
cost of placing............ 28
distribution in slabs........ 34
galvanized or coated........ 68
high carbon................ 13
medium 18
ordering. 111
percentage of.........14, 18,47
place in columns........... 50
protection 9, 10
record 111
formula for stress.......16, 22
too much................... 21
in walls................... 56
area and weights........... 27
use of high carbon......... 18

Stirrups, use and rules........ 25
Straight line formulas.......... 21
Straw 110
Strength added to columns by
steel 49
of concrete..............11, 20
ultimate, of steel.......... 18
Stress, fibre................... 17
Stresses fixed by ordinance...... 31
illustrated 11
internal 15
ratio of.................. 29
shearing 25
steel and concrete.......... 22
Studded forms................. 87
Supports, reinforcement over... 32
loads on.................. 34
Surcharged walls............... 53

T beams....................30, 98
Tamping concrete.............. 97
Tanks, circular................ 59
Temperature 42
Tensile strength of concrete..... 11
Tension failure................ 24
formula 22
illustrated 11
Testing beams................. 64
Thawing materials............. 109
Thickness of boards for forms..102
Thickness and length of columns 45
Thickness of slabs............. 33
Tinners' snips.................117
Top reinforcement............. 32
Twisted bars................18, 67
Turneaure's tests.............. 64

Ultimate strength, vs. elastic
limit 18
Unit deformation.............. 12

Variation in concrete.......... 65
Vertically reinforced columns... 44
Vertical shear................. 24

Wall footings.................. 63
formulas 55
pressures 53
proportions 56
restrained 58
retaining 52
Water, amount of.............. 96
pressure of............... 53
Weight of earth............... 56
concrete 21
and areas of steel.......... 27
Welded fabrics................. 27
Wetted forms................. 106
Wheelbarrows 95, 122
Width of beam................. 15
Wire drawing.................. 17
table 101
woeking 117
Wiring forms.................. 79
around columns............ 44
Wooden beam table............104
Working concrete.............. 97
Wound Columns................ 45
Woven fabrics................. 27