DEPARTMENT OF COMMERCE
WASHINGTON

ELIMINATION OF WASTE SERIES

RECOMMENDED BUILDING CODE REQUIREMENTS FOR WORKING STRESSES IN BUILDING MATERIALS

REPORT
OF
BUILDING CODE COMMITTEE

BUREAU OF STANDARDS

WASHINGTON
GOVERNMENT PRINTING OFFICE
1926

UNITED STATES DEPARTMENT OF COMMERCE
HERBERT HOOVER, SECRETARY

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REPORT OF BUILDING CODE COMMITTEE JUNE 1, 1926

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Frank P. Cartwright, Technical Secretary.

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LETTER OF SUBMITTAL

WASHINGTON, D. C., *June 29, 1926.*

Hon. HERBERT HOOVER,

Secretary of Commerce, Washington, D. C.

DEAR SIR: I send you the inclosed report, entitled "Recommended Building Code Requirements for Working Stresses in Building Materials." It constitutes the sixth in the series thus far completed.

The stresses which are assumed in the design of buildings constitute a fundamental basis for establishing the structural safety of such buildings; hence the necessity for exercising care in striking a balance between employment of low stress assumptions affording high safety factors and the economy which might accrue from use of high stress values. Stresses must also be considered along with load assumptions.

The fact that these recommendations are to be generally distributed throughout the country, often in localities where there is little or no control over quality of materials employed, is justification for the moderately conservative attitude assumed by your committee on this subject.

A tentative draft of the report was distributed to technical experts and criticisms solicited. The suggestions received have been included in the report as now presented in as far as it seemed advisable to do so.

If the report meets with your approval, the committee recommends that it be printed for public distribution.

Yours very truly,

IRA H. WOOLSON,
Chairman, Building Code Committee,
Department of Commerce.

LETTER OF ACCEPTANCE

DEPARTMENT OF COMMERCE,
OFFICE OF THE SECRETARY,
Washington, July 13, 1926.

MR. IRA H. WOOLSON,
Chairman, Building Code Committee,
Department of Commerce, Washington, D. C.

DEAR MR. WOOLSON: I have received the Recommended Building Code Requirements for Working Stresses in Building Materials prepared by the Building Code Committee. A great amount of painstaking work has been done for which the committee deserves much credit.

The need for undertaking such a task is made evident by the variations in existing stress requirements in different localities. In many instances improvements in building materials and the advance of engineering knowledge appear to justify more economical practice. Your committee has given thoughtful attention to this. At the same time it has surrounded its recommendations with adequate safeguards.

Yours faithfully,

HERBERT HOOVER.

RECOMMENDED BUILDING CODE REQUIREMENTS FOR WORKING STRESSES IN BUILDING MATERIALS

This report is in three parts. The first describes the organization and purposes of the Building Code Committee; the second presents requirements recommended for adoption in building codes; and the third explains briefly the grounds upon which the recommendations of Part II are based, and discusses the conditions by which they are limited. Some of this material, such as stress tables, is quite suitable for inclusion in city building codes. Part III also contains various references to good building practice and other information helpful to building code committees.

Part I.—INTRODUCTION

The Building Code Committee of the Department of Commerce was organized early in 1921, to meet a generally expressed public demand for greater uniformity and economy in building code requirements. Its first work concerned small dwelling construction, and a report on that subject was published in January, 1923. Subsequent reports issued by the department have presented minimum requirements for plumbing installations, for the erection of masonry walls of all types, for the live loads to be assumed as a basis for building design, and a recommended form for code arrangement.[1]

Diversity of Present Stress Requirements.

It is well known that existing building codes differ as to the working stresses allowable for construction materials. These variations are considerable, as will be seen from tables presented in the Appendix, paragraph 4. Differences in the code requirements of different cities and in the manner of their statement, with resulting possibilities of misunderstanding, are reported to cause much inconvenience to architects and engineers responsible for building design. The opinion also was freely expressed, in connection with the earlier draft of this report, that uniform code requirements for working stress limits would greatly facilitate the work of contractors and would operate to reduce building costs.

Recent Stress Practice More Economical.

Considerable reduction in construction costs should result from adoption of code stress requirements more in keeping with recent

[1] See back cover page for list of department publications on building and building regulation.

improvements in materials, design, and construction. There is reported a general trend in the building industry toward more accurate grading of materials, more careful analysis of loads and stresses, and better control of construction operations, making possible, according to the consensus of opinion, the use of higher design stresses where such improvements obtain. Building codes have been slow to recognize this tendency, with the result that prevailing stress limits are considerably lower than are recommended by the major professional groups in the industry. This often involves an unnecessary additional building cost. It is only where high stresses are used by those not fully competent to conduct building operations that danger may result with more advanced stress provisions, and it is questionable in such cases if the most conservative requirements could be regarded as safe.

Stress limits for one material often are more conservative, from a safety viewpoint, than those for others. Such unjustifiable departures in code requirements at times result in cost differentials sufficient to handicap one material or construction type seriously in competition with others. While public safety is the primary object of building codes it is desirable that public economy be promoted also, so far as possible, and it is hoped that this report, by presenting recommendations based primarily on safety considerations, will help to prevent uneconomic situations due to disregard of such considerations.

The differences between the working stress requirements and the standards generally accepted are offset in some building codes by compensating departures in live load assumptions or other influential factors, with the result that construction is safe and under the circumstances reasonably economical. In other instances this is not the case. The Building Code Committee in previous reports has already treated some of those related factors, such as live load assumptions and the quality of masonry materials. The recommendations of this report are predicated on those of previous reports and should be used with caution unless the committee's findings on other related questions are also adopted.

Scope and Purpose of Report.

This report presents recommended working stress requirements for reinforced concrete, cast iron, steel, and timber, with an Appendix explaining the reasons for certain of the recommendations and giving in condensed form information bearing on some of the limits suggested. Wrought iron no longer is used to any extent for structural purposes and, therefore, has been omitted from consideration.

Procedure of Committee.

The report in an earlier form was submitted widely for critical review by those whose interest and experience qualified them to discuss its subject matter with authority. Over 250 reviews were received, representing the opinions of a much larger number of architects, engineers, contractors, building officials, steel manufacturers, and steel fabricators. The summary of professional opinion thus obtained was found to justify important revisions of the committee's tentative recommendations and was used extensively for reference in preparing the final report.

The committee's recommendations for concrete are based largely on those of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete;[2] those for timber on work of the United States Forest Products Laboratory; and those for steel on work of the American Institute of Steel Construction and of the American Society of Civil Engineers. These organizations are believed to have been instrumental more than any others in ascertaining and codifying good practice in their respective fields. Their findings already represent a very wide consensus of professional opinion, and in experienced hands should assure safe, economical construction. Such variations as may be found in these recommendations from those advocated by the authorities named have resulted from careful consideration, and are believed to be justified by the present status of building regulation.

Building Inspection.

The committee's recommendations represent in nearly all cases relaxations from the prevailing limitations imposed by building codes. To this extent they should effect economies in building construction. In order to be structurally safe, on the other hand, buildings utilizing these increased stresses must be planned and constructed by those technically trained and experienced; and both materials and workmanship must be of uniform, dependable grades. Both the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete and the American Institute of Steel Construction have codified the requirements for materials and workmanship on which their stress recommendations are based, and while publications of the latter have given somewhat less attention than those of the former to the factor of design, it is understood that material bearing on this phase of the subject is in preparation. The American Society of Civil Engineers' Special Committee on Stresses in Structural Steel, after long investigation, also has predicated its recommendation of higher steel stresses on thoroughly competent design and construction. The lumber industry also has done much to safeguard the use of its products. Under the leadership of

[2] Representing the American Society of Civil Engineers; American Society for Testing Materials; American Railway Engineering Association; American Concrete Institute; and Portland Cement Association.

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the Forest Products Laboratory and the National Lumber Manufacturers' Association carefully defined lumber grades have been developed, and much information on the proper use of wood in construction has been published. The discussion of timber grades, factors influencing strength, and directions for design presented in this report are believed to afford at least a partial basis, in competent hands, for use of the timber stresses recommended.

Discussions of this report in tentative form disclosed a prevailing opinion that safety is not so much influenced by stress limits as by the competency of those employing them. Relatively few building codes at present provide any machinery for controlling the class of men responsible for building design and construction. City inspection departments do not and should not assume the full responsibility for safe construction. There undoubtedly is a strong trend toward the employment of more competent talent to design and supervise the construction of buildings. The registration of architects and engineers in several States provides at least partial foundation for control, and is utilized to that end in some cities. Nevertheless there is, in general, no method, except the hurried and necessarily casual check by building officials, of insuring that building design or construction will not be attempted by those utterly inexperienced or incompetent.

No very promising procedure has been proposed to remedy this situation, and it is probable that safety in building must continue for a time to depend on the personal standards of those in the industry, supplemented by the vigilance of public building officials. It, therefore, behooves cities adopting the higher stresses herein recommended to provide the necessary personnel and facilities for an effective check on the conduct of building operations. The expenditures involved are minor in comparison with the economies possible under such a régime. It can readily be shown, for example, that the saving in cost on one or two large buildings would pay the annual cost of providing adequate building inspection for a medium-sized city.

Acknowledgment.

The committee desires to acknowledge its indebtedness to the great number of architects, engineers, builders, building officials, and others who assisted in the preparation and review of the tentative draft of this report, and whose advice has been helpful in making the final draft representative as nearly as possible of the average opinion in the building industry.

Part II.—RECOMMENDED CODE REQUIREMENTS

Section 1. General Requirements.

All members shall be so framed, anchored, tied, and braced together as to develop the maximum strength and rigidity consistent with the purposes for which they may be used or to which they are likely to be subjected, and the stresses hereinafter recommended are based on the assumption that the details and connections used are fully as strong as the members connected. (See Appendix, par. 3.)

Workmanship in fabrication, preparation, and installation of material shall conform throughout to good engineering practice. (See Appendix, par. 3.)

Section 2. Working Stresses in Reinforced Concrete.

1. The direct compressive working stress on plain concrete used in reinforced concrete construction shall not exceed 0.25 of the assumed ultimate compressive strength of the concrete. (See Appendix, par. 5.)

2. The stress in the extreme fiber of the compression face of a reinforced concrete beam shall not exceed 0.40 of the assumed ultimate compressive strength of the concrete. (See Appendix, par. 6–5.)

3. The working stress in shear on plain concrete in reinforced concrete construction shall not exceed 0.02 of the ultimate compressive strength of the concrete. (See Appendix, par. 6–7.)

4. The working stress in bond between concrete and steel shall not exceed 0.04 of the ultimate compressive strength of the concrete in the case of plain bars and 0.05 in the case of deformed bars.

5. The temperature of freshly placed concrete shall be maintained at not less than 50° F. for at least five days after placing. (See Appendix, par. 6–5.)

6. Concrete in reinforced concrete construction, when mixed in the following proportions, stated by volume, shall be assumed to develop a laboratory cylinder compressive strength at 28 days as follows: (See Appendix, par. 6–3.)

TABLE 1.—*Assumed strength of concrete mixtures*

PLASTIC MASS CONCRETE (SLUMP 1 TO 3 INCHES)

Approximate mix: Volume of Portland cement to sum of separate volumes of fine and coarse aggregate	Water-cement ratio,[1] U. S. gallons per 94-pound sack of cement	Assumed ultimate strength at 28 days
		Lbs./in.²
1–6	7¼	2,000
1–5	6½	2,500
1–4	5¾	3,000

[1] Water or moisture contained in the aggregate must be included in computing the water-cement ratio.

TABLE 1.—*Assumed strength of concrete mixtures*—Continued

MODERATELY WET CONCRETE (SLUMP 6 TO 8 INCHES)

Approximate mix: Volume of Portland cement to sum of separate volumes of fine and coarse aggregate	Water-cement ratio, U. S. gallons per 94-pound sack of cement	Assumed ultimate strength at 28 days
		Lbs./in.^{2}
1-6	8	1,600
1-5	7¼	2,000
1-4¼	6½	2,500
1-3½	5¾	3,000

VERY WET CONCRETE (SLUMP 10 INCHES OR MORE) [2]

1-5	8	1,600
1-4	7¼	2,000
1-3¼	6½	2,500
1-2½	5¾	3,000

[2] Concrete of this consistency, while frequently used, should be avoided, as it will be porous and of low resistance to weathering.

In no case shall concrete for any assumed strength be placed with a water-cement ratio exceeding that shown. Where the aggregates are such that the mixes shown do not produce proper workability with the given water-cement ratios, the mixes shall be changed, but not the water-cement ratios.

The graded sizes of the combined aggregate shall be such that when separated on a No. 4 standard sieve the weight passing shall not be less than one-half or more than two-thirds of the total. (See Appendix, par. 6–5.)

7. If and when it is shown by evidence of tests made by competent authorities satisfactory to the building official that concrete of a higher strength than that specified in the preceding paragraph will be used and that competent field control of mixing and placing is assured; or that concrete of less strength will be employed; the stresses shall be proportionately modified, provided that in no case shall a strength of more than 3,000 pounds per square inch be assumed. (See Appendix, par. 6–5.)

8. The tensile stress in steel reinforcement shall not exceed 16,000 pounds per square inch, provided that this stress may be increased to 18,000 pounds per square inch when assurance is furnished the building official that the material conforms to the American Society for Testing Materials Standard Specifications for Billet Steel Concrete Reinforcement Bars, serial designation A 15–14, or for Rail-Steel Concrete Reinforcement Bars, serial designation A 16–14. The tensile stress in cold-drawn steel wire meeting the tentative specifications A 82–21T of the American Society for Testing Materials

shall not exceed 18,000 pounds per square inch. Concrete shall not be assumed to resist direct tensional stress. (See Appendix, par. 6–8.)

9. In the absence of rules adopted by the building official, and except as otherwise specifically provided in this code, the assumed formulas and specifications of the latest report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete shall be assumed in calculating the strength of slabs, beams, columns, and other elements of reinforced concrete structures, and designs shall conform thereto. (See Appendix, par. 2.)

Sec. 3. Cast Iron.

1. Compressive stresses in hollow cast-iron columns shall not exceed values determined by the formula

$$\frac{P}{A} \text{ equals } 9{,}000 - \frac{40L}{r}$$

in which

$\frac{P}{A}$ equals compression in pounds per square inch,

L equals length of the column in inches, and

r equals minimum radius of gyration of the column.

2. The maximum allowable ratio of L to r shall not exceed 90, except that when allowable working stresses computed by the above formula are reduced one-third the ratio of L to r may be increased, but shall not exceed 120. (See Appendix, par. 8–2.)

3. Cast-iron columns shall not be used in any case where the load is so eccentric as to cause tension in the cast iron; nor shall they be used for parts of the structural frame of buildings which are required to resist stress due to wind. Tensile stresses in the extreme fiber of cast-iron lintels or elsewhere, except in columns, shall not exceed 3,000 pounds per square inch.

4. The material and workmanship of cast-iron columns shall be equal in all respects to that described in the American Society for Testing Materials Standard Specifications for Cast-Iron Pipe and Special Castings, serial designation A 44–04. (See Appendix, par. 8–6.) All columns resting on or supporting other columns shall have their ends machine faced to a plane surface perpendicular to the axis.

Sec. 4. Working Stresses in Structural Steel Shapes.

1. For steel acceptable to the building official, but of which the origin and physical characteristics are not definitely determined, the maximum working stresses shall not exceed those given in column (*a*) of the following table. (See Appendix, par. 9.)

2. When assurance satisfactory to the building official is furnished that the steel to be used conforms to the A. S. T. M. Standard

Specification for Structural Steel for Buildings, serial designation A 9–24, the maximum working stresses shall not exceed those given in column (*b*) of the following table.

TABLE 2.—*Maximum working stresses in structural steel*

	(*a*) Acceptable steel	(*b*) Standard steel
	Lbs./in.²	*Lbs./in.²*
Direct axial tension on net section	16,000	18,000
Direct axial compression, maximum for short columns	12,500	14,000
Compression in columns	[1] $16{,}000-60\frac{L}{r}$	[1] $18{,}000-70\frac{L}{r}$
Fiber stress in flexure, in tension, or in compression when the unsupported length (L) is not more than 15 times the breadth (b)	16,000	18,000
Compressive fiber stress in flexure for values $\frac{L}{b}$ between 15 and 40	$19{,}600-210\frac{L}{b}$	$22{,}000-270\frac{L}{b}$
Fiber stress in pins	24,000	27,000
Bearing on plane faced or rolled surfaces	24,000	27,000
Shear in gross section of webs of girders and rolled shapes in which (d) the unsupported depth between flanges or the distance between stiffeners, if less, divided by (t) (the thickness of web) does not exceed 43	10,700	12,000
Shear when $\frac{d}{t}$ exceeds 43	$13{,}300-62\frac{d}{t}$	$15{,}000-70\frac{d}{t}$
Shear in power-driven rivets or in pins	12,000	13,500
Shear in hand-driven rivets or in rough bolts	9,000	10,000
Bearing upon power-driven rivets or in pins subjected to single shear on one side of the bearing in question	24,000	24,000
Bearing upon power-driven rivets or on pins when the bearing metal lies between two planes of shear of opposite character immediately adjacent	30,000	30,000
Bearing upon hand-driven rivets or on rough bolts subjected to single shear on one side of the bearing in question	16,000	16,000
Bearing upon hand-driven rivets or on rough bolts when the bearing metal lies between two planes of shear of opposite character immediately adjacent	20,000	20,000

[1] Compression stresses in columns, computed by the formulas for column design, may not exceed in any case the maximum for direct axial compression short columns. L equals length of column; r equals least radius of gyration.

3. Columns shall be limited in slenderness to a value of $\frac{L}{r}$ equals 160. Compression flanges of beams and girders shall not exceed in length between lateral supports 40 times their width. By the term "lateral supports" is meant points where definite resistance to lateral deflection is provided of sufficient strength to prevent buckling.

Combined stress due to flexure and axial stress shall not exceed that allowed for flexure. The axial stress alone, if compression, shall not exceed that allowed in columns.

4. For stresses either direct or flexural produced by wind loads, or by a combination of wind loads and dead and live loads, the working stresses allowed in paragraphs (1) and (2) may be increased by 25 per cent, provided the resulting sections are not less than those required for the dead and live loads alone.

Section 5. Working Stresses in Wood Members.

1. All wooden structural members shall be of sufficient size to carry the load safely without exceeding the allowable working stress of the material specified in Table 3 below. The strength of such members shall be determined from actual dimensions of the pieces and not from nominal dimensions.

2. Stress due to dead and live loads acting singly or in combination, without wind load, shall not exceed the allowable stress specified in Table 3, below. For stresses produced by wind loads or by a combination of wind loads and dead and live loads the working stresses allowed below may be increased by 50 per cent, provided the resulting sections are not less than those required for dead and live loads alone. (See Appendix, par. 14.)

3. Stress in compression perpendicular to the grain may be increased by 50 per cent above that specified in Table 4 below in the case of joists supported on a ribbon board and spiked to the studding rather than resting upon or in masonry. (See Appendix, par. 10.)

4. The restrictions and limitations of the two preceding paragraphs apply to all timber structures in which the lumber is in a dry location and not exposed to the weather. Timbers exposed to the weather shall be designed on a basis of working stresses 25 per cent[1] lower than those recommended in Table 3.

TABLE 3

(A table of allowable working stresses similar to that given in the Appendix, par. 10, Table 7, is recommended to be inserted at this point. It is not expected, however, that each code will include all the species listed in Table 7. It should include the species locally used and careful consideration should be given to all of the species listed in Table 7 since under present lumbering and transportation conditions many species are available which grow in remote sections of the country. Frequently such species are of secondary importance from the standpoint of lumber produced, but the very fact that they are little used is often reflected in a lower price. Their inclusion in a code would not infrequently lead to cheaper construction without loss in quality.)

5. Working stresses in compression parallel to grain for columns shall not exceed those prescribed in Table 4 for the respective species and ratio of unsupported length to least dimension. The ratio of unsupported length of columns to least dimension shall not exceed 50.

[1] When the amount of timber construction exposed to weather justifies a scale of stresses, the reductions advocated by the Forest Products Laboratory in U. S. Department of Agriculture Circular No. 295 should be followed. These reductions range from 12½ to 33⅓ per cent of the stresses recommended in paragraph 10 of the Appendix, depending upon the strength property under consideration.

TABLE 4.—*Safe stresses for square and rectangular wooden columns*

Species	Grade	Ratio of length to least dimension: 11 or less	14	17	20	23	26	29	32	35	40	50
		Lbs./in.^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}
Aspen Basswood Cedar, western red Cottonwood Fir, balsam	Select Common	700 560	668 544	630 524	566 492	466 440	365	293	241	201	154	99
Fir, commercial white Hemlock, eastern Poplar, yellow	Select Common	700 560	678 549	653 536	611 515	544 480	446 430	358	294	246	188	121
Chestnut Maple, red and silver Pine, white, sugar Pine, western white Pine, western yellow	Select Common	750 600	718 583	680 564	617 532	518 481	405	326	268	224	171	110
Douglas fir (Rocky Mountain) Elm, slippery and white Gum, red, black, cotton Pine, Norway Spruce, red, white, Sitka	Select Common	800 640	774 627	742 610	688 582	604 540	486 476	391	321	268	206	132
Hemlock, western	Select Common	900 720	872 706	839 688	783 660	695 615	567 549	456	375	313	240	153
Ash, white Oak, red and white	Select Common	1,000 800	967 783	927 762	860 728	755 674	608 595	489	401	336	257	164
Douglas fir (Coast type), dense Pine, southern yellow, dense	Select Common	1,285 1,060	1,222 1,025	1,147 985	1,023 913	831 803	649	521	428	358	274	175
Beech Birch, yellow and sweet Douglas fir (Coast type) Pine, southern yellow Maple, sugar and black	Select Common	1,175 880	1,127 861	1,070 837	975 796	826 734	649 640	521	428	358	274	175

NOTE.—The values in this table are obtained from the values given in Table 7, using the formulas for intermediate and long columns given in Part III. The grouping of the species, in order to save space and make the table simple, has resulted in somewhat lower stresses for some of the little-used species than can be safely applied. This table may be shortened by omitting some of the species, or more detail may be added if desired. There are available at the Forest Products Laboratory, Madison, Wis., tables which give the stresses based on the factors of safety approved by the committee for each species and each ratio of $\frac{L}{d}$ from 11 to 50.

Source: U. S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, Wis., Jan. 16, 1926.

NOTE:—*Masonry stresses:* Requirements for quality of masonry materials, workmanship, and the allowable stresses in masonry structures have already been approved by the Building Code Committee. They appear in the committee's report on Recommended Minimum Requirements for Masonry Wall Construction.

Part III.—APPENDIX

Par. 1. Purpose.

The Appendix consists of explanatory matter referring to Part II, and is a vital part of this report. The Building Code Committee believes that every building code should be printed with an appendix containing sufficient explanation of the code requirements to make them easily understandable, and to assist building officials in their enforcement. Such information on good practice as can not be obtained elsewhere in concise form also may be presented. The Appendix to this report is intended to clarify certain questions which may be raised concerning the recommended requirements and to assist in their adaptation for local use.

Par. 2. Sources of Information.

The committee has endeavored in all cases to base its recommendations on the latest and most authoritative information. Where references are made in Part II to the requirements or standards of national professional societies these references apply to the latest published form of such requirements and standards and it is expected that those writing or revising building codes will refer in similar manner to the latest material then available.

Part II requirements for timber are based on investigations by the U. S. Forest Products Laboratory, Forest Service, Department of Agriculture, supplemented and checked by various national and local associations concerned with the utilization of timber. The Forest Products Laboratory was established in 1910. Since then it has become the best equipped and most dependable source of information on the use of wood for building and other purposes. It conducts annually, in accordance with a carefully considered program, a great number of experiments with timber, to develop the full structural possibilities of this material.

In preparing recommended stress requirements for steel the Building Code Committee has studied, among other sources of information, the report of a committee of architects and engineers retained by the American Institute of Steel Construction to investigate and recommend steel construction requirements for building codes, and the report, submitted at the January, 1925, meeting of the American Society of Civil Engineers by a Special Committee on Stresses in Structural Steel, appointed in August, 1922. Careful consideration also has been given to the specifications for railway and highway bridges of the American Railway Engineering Association and the American Society of Civil Engineers as well as to other standards in common use. The comparison with specification for bridges is believed entirely proper considering that the stresses therein specified are for equivalent static forces, full allowance having been made for impact. No two specifications of those authorities are in precise agreement and the committee can not accept in its entirety any one. Most of the working stresses recommended are to be found in one or more existing specifications and where new suggestions are offered they are for the purpose of simplification.

The recommended stresses for reinforced concrete are based mainly on the 1924 report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. This report is the outcome of a long series of investigations under the auspices of interested national organizations. The first joint committee, representing the American Society of Civil Engineers, the American Society for Testing Materials, the American Railway Engineering Association, and the Portland Cement Association, was organized in 1904, to prepare a code of practice for concrete and reinforced concrete. The American Concrete Institute became a member in 1915. Progress reports were made in 1909 and 1912 and a final report in 1916.

The present joint committee, representing the same organizations, was formed in February, 1920, and presented a report under date of August 14, 1924.[1] Its membership, and the standing of the agencies concerned not only insured access to every valuable source of information on the subject, but also made it possible to institute further investigations of questions not well developed, and to test the practicability of its recommendations in the field before their adoption.

Par. 3. Influence of Building Inspection.

Throughout the report the quality of materials to be used has been considered in conjunction with the working stresses allowed for such materials. There are, however, many other factors influencing the safety of structures, which are partly controllable by code requirements, but are in great part dependent on the knowledge and integrity of those in charge of construction.

Skill in design and construction is much more important with the new types of skeleton-framed buildings than in those with masonry bearing walls. Masonry buildings have only a moderate area of openings and loads are readily and uniformly distributed down and through the walls. In steel and reinforced concrete buildings, on the other hand, modern practice permits excessively

[1] Copies of the report may be obtained from the American Society for Testing Materials, 1315 Spruce Street, Philadelphia, Pa., or the American Concrete Institute, 1807 East Grand Boulevard, Detroit, Mich.

large wall openings with resulting slender supporting members, the economical design of which introduces complicated problems. The new structures do not depend on gravity alone for stability and rigidity, but also upon interaction of beams, girders, and columns, and the attachment of these last to footings which must be specially adapted to support excessive foundation loads. Success in designing such buildings demands extensive and intimate knowledge of materials, of the mathematics by which the sizes of columns and beams are computed, and the effective use of reinforcement in concrete. Years of training and preliminary experience under competent direction are essential before the responsibility for design and construction of the modern large building can safely be undertaken. A general increase in working stresses, which narrow the margins of protection for buildings poorly designed or built, makes competency even more necessary.

Popular opinion has been slow to recognize that the new types of building construction are work for specialists. Building officials, recognizing the need for greater care, have extended the scope of their efforts and have done much to improve building construction and promote safety. With present enforcement methods, however, and with the facilities usually provided for municipal inspection, the possibilities of such inspection are limited. It is questionable, furthermore, whether the extension of inspection service would materially improve the situation, since it would only tend to transfer responsibility from those executing the work to the supervising authorities.

Discussions of the tentative report reflect very strongly the opinion that building officials should be given authority to distinguish in some manner whether applicants for permits are generally competent to plan and conduct building operations, and to refuse permits to those judged incompetent. Such an arrangement might be, and probably would be, subject to abuse in some cases, but the same objection could be raised to practically all forms of licensing and regulation of professional and trade activities. There is legal precedent in full measure for such an arrangement, and it is understood in fact that measures partially effective in fixing responsibility are already in force in some cities.

A full discussion of the subject is without the scope of this report. It is strongly recommended, however, that in connection with increase of stresses measures be taken by which owners will be compelled to employ competent architectural or engineering service throughout each important building operation.

Such measures, compelling careful design and construction, would bear dividends not only in increased safety but in economy. The issue between 16,000 and 18,000 pounds per square inch stresses for steel, with its possible marked economy in the use of material, is not so much a question of what the material will safely bear, as whether the designer is able to predict accurately the stresses which will actually occur and build the structure to resist them. Except for cases where ability to do this can be at least presumptively established, it seems unwise to suggest any increases over the customary stresses. Competent supervision should also control workmanship and quality of materials. This is particularly necessary with reinforced concrete construction, since the major material of the combination is made in the field from a number of constituents variable in quality and quantity, and often by workmen who are more concerned with rapid production and easy handling than with accuracy of proportions and care in mixing and placing.

It is recommended that building officials, in supervising the activities of privately employed inspectors, utilize the standards for materials and workmanship outlined in the 1924 Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, and the American Society for Testing Materials Tentative Rules for Inspection of Concrete and Reinforced Concrete Work, serial designation C 44–22 T. The American Concrete Institute also has prepared a report on building regulations for concrete and reinforced concrete construction which should assist considerably in establishing standards of workmanship consistent with the stresses recommended in Part II.

Directions for good workmanship on steel frame buildings may be found in the code of standard practice of the American Institute of Steel Construction, and much valuable information on the proper use of lumber in the publications of the National Lumber Manufacturers' Association.

It also should be remembered that the Building Code Committee, in its Report on Minimum Live Loads Allowable for Use in Design of Buildings, has considerably reduced the prevailing live load assumptions with a view to making them approximately the maximum reasonably probable in the projected use of a building and confining the factor of safety to the choice of stresses. If this policy is coupled with an increase in design stresses great care is necessary in every department of the work to secure safety.

Par. 4. Variations in Building Code Requirements.

Tabulations summarizing the requirements of several existing building codes for working stresses in timber and reinforced concrete have been prepared by the National Lumber Manufacturers' Association and the Portland Cement Association, respectively. These are presented here to illustrate the variations in present practice and the need of greater uniformity in code requirements.

TABLE 5.—*Summary of analysis of wood stress requirements of 117 codes, National Lumber Manufacturers' Association*

BENDING

Species of timber	Extreme fiber stress			Horizontal shear stress		
	Codes giving stresses	Codes with no stresses	Range of requirements	Codes giving stresses	Codes with no stresses	Range of requirements
			Lbs./in.²			*Lbs./in.²*
Douglas fir:						
Dense	61	56	1,800– 800	60	57	750–80
Sound	71	96	1,600–1,000	18	99	350–85
Hemlock:						
Eastern	68	49	1,300– 600	62	55	250–40
Western	30	87	1,500– 600	32	85	160–40
Oak	96	21	1,800– 600	96	21	240–80
Norway pine	46	71	1,250– 700	43	74	150–40
Southern yellow pine:						
Dense	100	17	1,800– 500	101	16	400–70
Sound	59	58	1,500– 900	58	59	350–70
Spruce	85	32	1,350– 250	86	31	500–50
Tamarack	9	106	1,200– 900	9	108	170–95
White pine	81	36	1,500– 250	77	40	500–40

COMPRESSION

Species of timber	Parallel to grain "short columns"			Perpendicular to grain		
	Codes giving stresses	Codes with no stresses	Range of requirements	Codes giving stresses	Codes with no stresses	Range of requirements
			Lbs./in.²			*Lbs./in.²*
Douglas fir:						
Dense	58	59	1,600–100	60	57	800–200
Sound	15	102	1,500–100	18	99	400–200
Hemlock:						
Eastern	62	55	1,200– 80	70	47	1,000–150
Western	31	86	1,500– 80	26	91	500–150
Oak	99	18	1,500–100	99	18	1,000–250
Norway pine	67	50	1,000–100	43	74	400–100
Southern yellow pine:						
Dense	98	19	1,800–820	102	15	1,500–250
Sound	57	60	1,200–750	62	55	800–170
Spruce	83	34	3,000–600	88	29	1,000–180
Tamarack	9	108	1,000–750	9	108	350–220
White pine	81	36	1,100– 80	80	37	1,000–150

NOTE.—Dense and sound in the foregoing includes values for No. 1 and No. 2 structural, longleaf, shortleaf, etc., respectively.

TABLE 6.—*Analysis of concrete stress requirements of 58 building codes from data furnished by the Portland Cement Association, September, 1925*

	Direct bearing	Direct compression on columns			Compression in extreme fiber		Shear			Bond	
		Without spiral reinforcement; on concrete alone	With spiral reinforcement				Diagonal tension on beams and slabs		Punching shear	Plain bars	Deformed bars
			On concrete alone	Additional credit given for spirals	Ordinary case	Underside of beams over support	Without web reinforcement	With full web reinforcement			
Number of codes limiting stress	51	47	44	{17, yes / 25, no}	52	16	50	33	21	48	41
Maximum stress allowed by any code:											
Given as percentage of ultimate	29.0	29.0	35.5	----------	37.5	37.5	2.5	6.0	6.0	4.0	6.67
Given in pounds per square inch	650	650	900	----------	800	850	75	150	150	90	150
Lowest stress given by any code as maximum:											
Given as percentage of ultimate	20.0	22.5	25.0	----------	32.5	37.5	2.0	5.0	5.0	3.33	5.0
Given in pounds per square inch	200	400	500	----------	500	650	30	65	100	50	50
Average of values given in percentages	22.25	24.00	32.87	----------	34.05	37.50	2.06	5.67	5.00	3.83	5.67
Average of values given in pounds per square inch	473.81	502.30	----------	----------	661.11	750.00	45.24	124.83	120.00	73.97	98.00
Joint Committee Requirements, 1924 report, given in percentage of ultimate	25.0	20.0	Variable.	----------	40.0	45.0	2.0 to 3.0	6.0 to 12.0	----------	4.0	5.0

Until recently code requirements for structural steel stresses have been uniformly based on a maximum of 16,000 pounds per square inch in tension and compression, though the stresses allowed for rivets varied considerably and a number of column formulas were in use. Efforts by those interested in more economical utilization of steel have changed this somewhat, and existing practice among the larger cities is divided between the older scale of stresses and a scale based on a maximum stress of 18,000 pounds per square inch.

Par. 5. Working Stresses in Plain Concrete.

It will be noted that the percentages of ultimate strength recommended in this report for plain concrete used in reinforced concrete building operations contemplate working stresses somewhat larger than are provided for in the Committee's Report on Masonry Wall Construction. This is in consideration of the better control over materials, proportioning, and workmanship usually obtaining in operations involving reinforced concrete. The working stress on the extreme fiber of the compressive face of a concrete beam is a theoretical stress. It has not the same significance as a direct axial stress. Failure of concrete beams in compression is rare.

In the Committee's Report on Recommended Minimum Requirements for Masonry Wall Construction, plain concrete was defined as concrete containing not more than two-tenths of 1 per cent of reinforcement.

Par. 6. Working Stresses in Reinforced Concrete.

1. The method of making and testing concrete test pieces is described in Standards C31–21 and C39–25 of the American Society for Testing Materials. The test specimens are cylinders 6 inches in diameter and 12 inches high, aged 28 days. The average strength of five test pieces is required.

2. Concrete materials vary so widely in the United States that it is inexpedient to fix stresses by exact values. Where coarse and fine aggregates are prevailingly uniform in quality, experimentation will determine quite closely the strengths obtainable with different mixes. In such cases the requirements of Part II, section 2, should be replaced by the strengths determined by tests on the locally used materials.

This procedure recently was followed by the District of Columbia Building Code Committee in revising certain portions of the District Building Code. Tests were made for the committee by the Bureau of Standards. Two consistencies of concrete were used; that is, (1) concrete defined by a cone slump of 6 inches, and (2) concrete defined by a cone slump of 9 inches. Samples of the local materials, which are quite uniform in nature, were mixed in several proportions in these consistencies and the compressive strength determined at the age of 28 days. The District of Columbia Committee then cited a table of assumed ultimate compressive strengths at 28 days for concrete of these two consistencies and indicated the mixtures which would be accepted as giving these strengths. To provide for field conditions the assumed values in the code were fixed at 60 per cent of the ultimate strengths shown by laboratory tests.

Such a procedure determines the quality and makes possible safe and efficient designs, the stresses for which will certainly vary between the washed and accurately prepared aggregates of the larger centers and the poor local materials in the less favored regions. Those expecting to invest large sums in important buildings should secure skilled designers and make preliminary tests, and should surround the construction with technical control in manufacture and placing of concrete.

3. The descriptive expressions "plastic mass concrete" and "reinforced concrete mixed moderately wet," upon which the stresses in Part II, section 2–6 are based, are best interpreted in terms of slump. The slump test determines the relative plasticity of fresh concrete by measuring its subsidence from the height of a truncated 12-inch cone after removal of the surrounding form. A standard procedure for making the slump test is described in the Tentative Test for Consistency of Portland-Cement Concrete for Pavements or Pavement Base, serial designation: D 138–25T, of the American Society for Testing Materials. In general, plastic mass concrete will have a slump of not more than 3 to 5 inches, and reinforced concrete mixed moderately wet not more than 6 to 8 inches.

4. Columns and beams, etc., in construction are larger and less carefully made than test pieces and will show lower strengths.

5. The average strength of three concrete test cylinders should not fall below a minimum of at least 1,600 pounds per square inch. The maximum to be expected from rich mixes carefully made will not usually exceed 3,500 and the committee, for building-code purposes, has limited the assumed maximum strength to 3,000 pounds per square inch. This implies a range of working stresses from 400 to 875 pounds per square inch for plain concrete, and 640 to 1,200 pounds per square inch in the extreme fiber of compression faces. These stresses suggested in the tentative report were criticized as somewhat too liberal. Careful investigation, however, discloses no cases in which reinforced concrete beams have failed by compression in the extreme fiber at the top of the beam.

High stresses for concrete are to be used only when tests demonstrate the increased strength of the material. Such strength results from clean and tested materials. Concrete should be mixed at least one minute in a batch mixer with devices for water control and time control. The concrete should be placed in uniform horizontal layers, sliced to avoid segregation, and protected, beginning after placing, and lasting for a time depending upon exposure (at least seven days when exposed to weather). The wetness of the mix necessary for various classes of construction will exert a preponderating influence on the strength.

It is well established that concrete of the best quality in respect to strength and durability is produced with the minimum amount of mixing water that is practicable. The table of assumed compressive strength in Part II, section 2–6, indicates the minimum ultimate strength in compression which may be expected at 28 days, with the amounts of water specified, and when cured and tested as specified by the joint committee's report. When mixing water is increased the cement content should also be increased to preserve a constant water-cement ratio appropriate for the desired strength.

Water or moisture contained in the aggregates must be included in determining the ratio of water to cement.

The Committee on Building Laws and Specifications of the American Concrete Institute has adopted the water-cement ratio as a means of controlling the strength of concrete. No specification of mixtures and aggregates is made.

The proportions in Part II, section 2–6, are specified in terms of unit volume of packed cement to the sum of the separate volumes of fine and coarse aggregate. The volume of the mixed aggregate is approximately 0.8 of the sum of the volumes of the separated aggregates. Thus, in the case of premixed aggregate, it must be remembered that a mix of 1:2:4—that is, 1:6 expressed as the sum of the volumes of separated aggregates—is approximately only a ratio of 1:4:8 expressed as the value of the premixed aggregate. The amount of coarse material should be limited to avoid harshness in placing or honeycombing in the structure. High strengths in laboratory tests are sometimes obtained by high proportions of cement to aggregate that does not contain enough fine material.

Variations occur in cement and aggregates. Fine aggregate is affected greatly by moisture; damp sand may increase in volume 30 per cent over dry and will produce a too wet mix or a nonuniform concrete. Fine soluble dust in the aggregate causes a porous concrete that shrinks; such concrete will absorb water and will be likely to deteriorate when exposed.

Fine material, such as silt or dust, even though inorganic, when in concrete mixed with an excess of water, will rise and form fill planes in a structure where subsequent disintegration will likely take place because of porous concrete, especially after the action of freezing and thawing. A recent survey of disintegrated concrete structures charges the causes of these defects to excess water in the mix. A recent rule on heavy construction is that the mix shall be of such consistency that workmen will not sink below their ankles in the freshly placed concrete.

The water-cement ratio specified should, of course, include the amount of water present in the aggregate. Sand and gravel coming directly from sand and gravel washing plants will contain an amount of water which, if not accounted for, would upset the required water-cement ratio. The varying amount of water in the sand also affects the volume occupied by a given weight of sand. A fine sand may bulk to increase of 40 per cent in the presence of 8 per cent of water; medium sand, 28 per cent; coarse sand, 24 per cent. The amount of water in the sand can be determined by a test in advance. Recent advanced practice measures the sand in an excess of water; that is, by inundation in order to obtain a more uniform concrete.

Concrete shrinks mostly during the first 10 hours. Therefore, protection against drying or freezing should begin as soon as possible after placing so that the concrete may attain strength to withstand shrinkage strains before these begin. The requirement in Part II, section 2–5, that the temperature of freshly placed concrete be maintained at not less than 50° F. for at least five days, is to prevent damage by frost and to allow the concrete to attain sufficient strength to withstand construction loads safely. Costly experience with failures of concrete structures has demonstrated that such a requirement is essential for safety.

Tests.—All tests of materials called for in Part II, or ordered by the authorities, should be made in accordance with the standard method of testing covering the particular material under consideration prescribed by the American Society for Testing Materials.

Tests should be made on all materials entering into concrete or reinforced concrete construction when in the opinion of the building official there is any doubt as to its suitability for the purpose.

The building official or his authorized representative should have the right to require tests of the concrete from time to time during the progress of the work to determine whether the materials and methods in use are such as to produce concrete of the necessary quality, or at any other time when in their opinion there is any doubt as to the quality of the concrete being produced. Specimens for such tests should be taken at the place where concrete is being deposited, and cured and tested in accordance with the American Society for Testing Materials Standards as required above.

All such tests should be made by competent persons approved by the building official, and copies of the results kept on file in the office of the building official for a period of at least two years after the construction work is completed.

If for any reason, in the opinion of the building official or his authorized representative, the testing of a completed structure is necessary, the member or portion of the structure should be given a superimposed load equal to one and one-half times the live load plus one-half the dead load. This load should be left in position for a period of 24 hours before removal. The structure should be considered to have failed under the test if within the 24 hours after the removal of the load the floor system fails to recover 75 per cent of the maximum deflection shown during the 24 hours while under load. In cases where failure is declared the building official should have the authority to order the defective construction removed.

If permission is sought to use systems of construction or reinforcement, the safety of which can not be demonstrated mathematically, the question should be settled by testing the completed structure in the manner described above.

6. Reinforced Columns.—When reinforcement is used to increase compressive strength of concrete, the allowable stress specified for plain concrete may be increased by a factor to be applied to columns whose length between supports does not exceed 10 times the least dimension. This increase is defined in the joint committee report, previously mentioned, and is provided for in Part II, section 2–9. (See also Appendix, par. 2.)

Part II specifies working stresses for materials. Methods of calculating the strength of materials in combination, such as slabs, beams, flat slabs, columns, etc., are to be adopted as defined in the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, representing the most acceptable practice.

Stresses on the concrete cores of reinforced columns should be considered in the light of test information. Spirals raise the ultimate crushing resistance and increase the toughness of columns but they have no substantial effect in increasing the elastic limit. That is to say the concrete in the core under vertical stress is not materially restrained from flowing sidewise. Therefore, the elastic limit of the column should be the basis of judgment, especially because there is a plastic or time flow of the concrete under stress, by which the concrete flows away from the load and throws added stress on the steel.

It will be noted that the assumed strength of concrete in compression is limited in Part II, section 2–7, to 3,000 pounds per square inch.

The proportional elastic limit of concrete—that is, the stress beyond which deformation shows a marked increase under load—will vary with the age and stiffness of the concrete. While the available scientific information is conflicting, reliable tests show a range of elastic limit from one-third to two-thirds the ultimate strength in the case of well made and aged concrete.

7. A plain concrete beam with no shear reinforcement fails (when it fails in shear) by formation of cracks at approximately 45° with

the axis of the beam, caused by tensional stress on an internal inclined plane. When this tensile stress exceeds 0.02 of the ultimate compressive strength, the factor of safety is too greatly reduced, and steel reinforcement is necessary. The rules of the joint committee provide for such reinforcement.

Practice has fixed the custom of allowing a stress of 40 pounds per square inch on concrete in shear and the remainder on the shear reinforcement.

Inclined rods tend to retard the formation of cracks. The shearing stress may then be increased. The greater the number of inclined rods the greater the toughness of the beam.

8. The joint committee report allowed 18,000 pounds per square inch on high-carbon steels. In this it follows the practice of a score of large cities, in 10 of which a tensile stress of 20,000 pounds per square inch is allowed in the reinforcement in the case of hard grade steel.

In indoor structures, the steel stresses may be raised to recognize the greater factor of safety against exceeding the elastic limit of high-carbon steel, provided, however, that allowable bond stresses are not exceeded. A favorable consideration is the fact that overloads which would produce a permanent open crack in structures reinforced with mild steel will not do so when reinforced with high-carbon steel, because the steel, still elastic, draws the crack together. Reinforcing bars rolled from either billet steel or steel rails are suitable reinforcement. Some question has been raised in past years as to the dependability of bars from the latter source and in 1922 extensive tests were made at the Bureau of Standards to determine the suitability of this material for general reinforcement purposes. The committee, after careful consideration of these and other recent tests and of experience with their use, is convinced that if such material meets the American Society for Testing Materials Standard Specifications for Rail Steel Concrete Reinforcement Bars, serial designation: A 16–14, it is suitable for concrete reinforcement. The joint committee report also supports this conclusion. No question of public safety is involved.

9. The Building Code Committee's recommendations for stresses differ in certain respects from a report on recommended code requirements for concrete and reinforced concrete construction, now under discussion by the American Concrete Institute.

1. The latter report allows only 16,000 pounds per square inch for reinforcing bars of structural grade, whereas the Building Code Committee has recommended that 18,000 be allowed for all grades of steel passing standard specifications.

2. The American Concrete Institute report would allow rail steel reinforcing bars larger than three-fourths inch to be used

only where bars are not to be bent. The Building Code Committee does not distinguish in size.

3. The American Concrete Institute Report, section D–1, fixes the amount of water in United States gallons per sack of cement against the expected ultimate strength in design as follows:

Ultimate strength:	U. S. gallons per sack of cement
1,500	8¼
2,000	7¼
2,500	6¼
3,000	5¾

The Building Code Committee has adopted the water-cement ratio as a simple and reliable method for controlling the strength of concrete.

The American Concrete Institute report is practically in agreement as to stresses and design formulas with the report of the joint committee. Except for the variations necessary because of the wide range of conditions to which building code requirements apply, the requirements of Part II reflect the recommended practice of both these organizations.

Par. 7. Design of Web Reinforcement.[2]

The equations for designing web reinforcement as given in the 1924 report of the joint committee are

$$v \text{ equals } 0.02 f'_c \text{ plus } \frac{f_v A_v}{bs \sin \alpha} \text{ for angles } \alpha \text{ of } 45° \text{ to } 90° \quad (31)$$

and

$$v \text{ equals } 0.02 f'_c \text{ plus } \frac{f_v A_v}{bs} (\sin \alpha \text{ plus } \cos \alpha) \text{ for angles } \alpha \text{ less than } 45° \quad (32)$$

where

v equals shearing stress,
$f'c$ equals compressive strength of concrete,
f_v equals tensile stress in web reinforcement,
A_v equals sectional area of bent-up bar or stirrup,
b equals thickness of web,
s equals distance from stirrup to stirrup or from support to first stirrup or bent-up bar. s is measured in the direction of the axis of the beam,
α equals angle of web member with longitudinal axis.

Although these equations seem unfamilar they are in fact quite similar to those in ordinary use. The similarity and the differences may be brought out by reducing them to a form which involves the ratio of the sectional area (or volume) of web reinforcement to the sectional area (or volume) of the concrete which is reinforced by it. The term $s \sin \alpha$ of equation (31) is the distance from stirrup to stir-

[2] Discussion supplied by Dr. W. A. Slater, of the Bureau of Standards.

rup measured at right angles to the direction of the stirrup. The term $\frac{A_v}{bs \sin \alpha}$ of equation (31) then becomes the ratio of web reinforcement. Call this ratio r. Equation (31) then becomes

$$v \text{ equals } 0.02f'_c \text{ plus } r\, f_v \qquad (31')$$

and equation 32 becomes

$$v \text{ equals } 0.02f'_c \text{ plus } r\, f_v\, (\sin^2 \alpha \text{ plus } \sin \alpha \cos \alpha) \qquad (32')$$

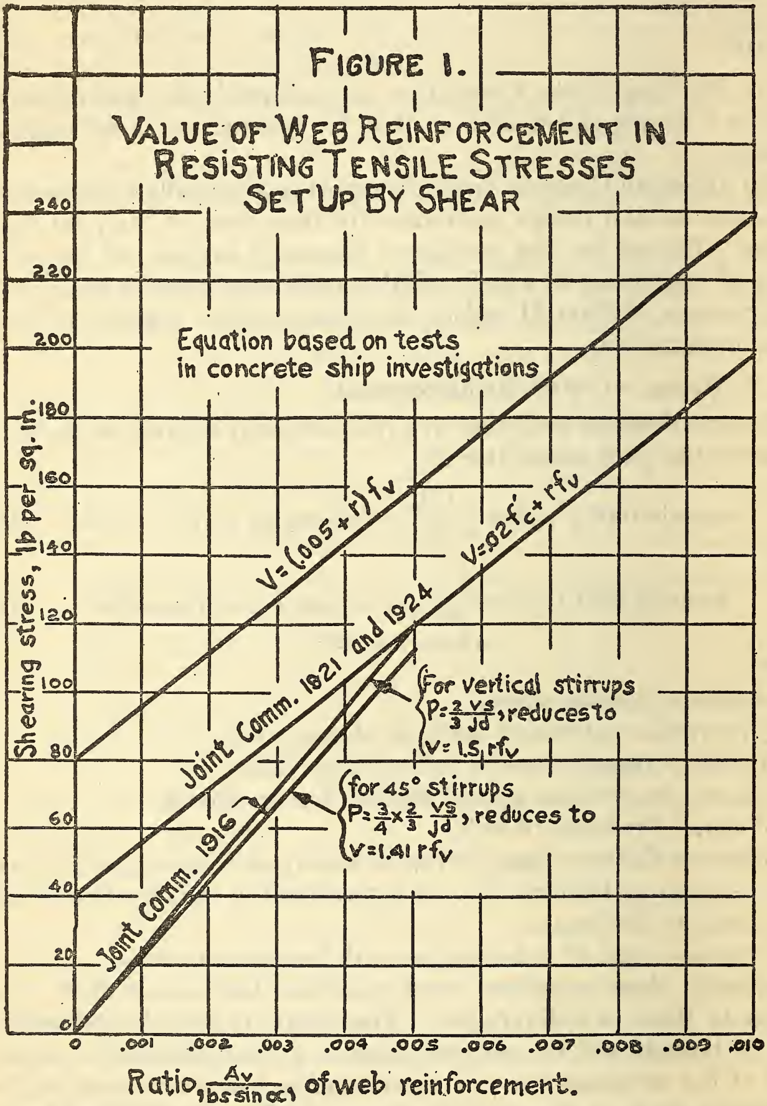


Figure 1 has been prepared for the purpose of showing relative values of a given quantity of web reinforcement in resisting the tensile stresses set up by shear as stated by these equations.

Equation (31) and Figure 1 bring out the fact that, as specified, a given quantity of steel is equally effective, in reinforcing for shear for all angles from 45° to 90°.

This equality is not peculiar to these specifications. Figure 1 shows that by the former joint committee's recommendations nearly the same percentage of reinforcement is required if the stirrups are placed at 45° as if they are placed vertically.

That a given percentage of steel was equally effective as web reinforcement whether placed at 45° or 90° was shown clearly by the tests made in the concrete ship investigations of the Emergency Fleet Corporation. Other angles were not used in these tests.

Equation (32) is a general equation for shear, in terms of the angle α and the other variables given. It is correct for all angles α in so far as the assumptions are correct on which the equation is based. For angles of 45° and 90° the values of sheering stress v given by the term

$$\frac{f_v A_v}{bs} (\sin \alpha \text{ plus } \cos \alpha)$$

are the same as are found by solving for v the familiar expression

$$A_v f_v \text{ equals } \frac{vs}{jd} \text{ equals } \frac{vbjds}{jd}$$

that is

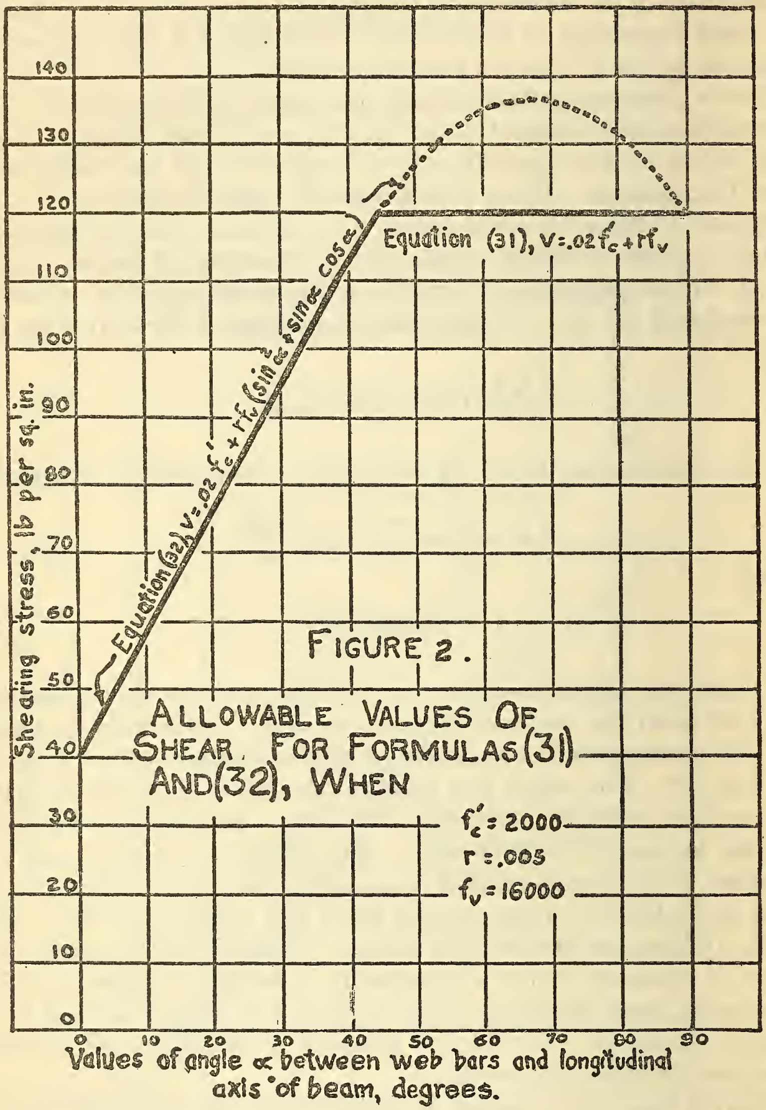
$$v \text{ equals } \frac{A_v f_v}{bs}$$

For these two angles the tests made in the concrete ship investigation confirmed the results of the analyses with considerable accuracy, but test results were not available for angles less than 45° or between 45° and 90°. For angles less than 45° the values given by equation (32) are both conservative and reasonable and the equation is recommended for use up to that angle. For angles between 45° and 90° equation (31) is recommended because the less conservative results there indicated by equation (32) have not been checked by test results and because it is simpler to use. Except for the greater simplicity of equation (31) it will probably make little difference which equation is used, since web reinforcement will seldom be used with angles between 45° and 90°, and since for these two angles the two equations give identical results.

Figure 2 shows the relation of these two equations when r equals 0.005 (0.5 per cent web reinforcement), f_v equals 16,000 pounds per square inch, and f'_c equals 2,000 pounds per square inch.

A technologic paper of the Bureau of Standards which is under preparation gives the derivation of the equations and gives the results of extensive tests of the shearing strength of reinforced concrete beams.

The formulas already stated give the area of reinforcement required without reference to the spacings. Neither formula includes a limit for the spacing, but such a limit is necessary in order to avoid having the web members placed so far apart as to permit diagonal cracks to



rise nearly to the top of the beam without crossing a web member. The formula for maximum permissible spacing, s equals $\frac{45\ d}{\alpha}$, seems to give reasonable values. Where shearing stresses greater than $0.06\ f'_c$ are used it seems well to limit the maximum spacing to two-thirds of these values.

Par. 8. Cast-Iron Columns.

There is evidence that prevailing code requirements governing the use of cast-iron columns have failed to keep pace with developments in the material and its handling, and that as a result this type of column either is unduly expensive to users or fails of its full economic utilization.

1. There is, as in the case of other materials, considerable variety in code restrictions. At least seven column formulas are in use, some based on a relation of length to radius of gyration and some on the ratio of length to least dimension.

The formula recommended in Part II is that most used. It gives values about midway of the maximum range, and the allowable stresses decrease somewhat more slowly with increasing $\frac{L}{r}$. The few test data available indicate a factor of safety of two or more for columns exceeding the prescribed limits of $\frac{L}{r}$.

2. Many codes do not give limits for $\frac{L}{r}$. Others set maximum limits of from 60 to 95, these values applying in both cases to the same formula recommended in Part II. In view of the general improvement in casting practice in recent years and the quality of metal required by Part II, it is believed the increase to 120 is sufficiently conservative. It should permit considerable extensions in the use of such columns for high top stories or where loads are nominal.

3. All cast-iron columns should have three-eighths inch test holes drilled in the shaft 2 feet from each end, so that the thickness of the walls may easily be determined.

4. A number of codes limit the total height of buildings in which cast-iron columns are used, presumably on the grounds that neither the columns nor their connections are particularly adapted to meet the stresses caused in high buildings by wind pressure. The limits of height prescribed vary from 6 stories, 75 feet, or one and three-fourths of the least width of the building, to 10 stories, 100 feet, or three times the least width of the building. It is believed no economic hardships will result if the limit is placed at 100 feet, or twice the least width of the building. Cast-iron columns usually are associated with masonry bearing walls, and since these latter have little inherent stability a certain amount of rigidity is necessary in interior construction.

5. It is quite customary to require that cast-iron columns have a minimum lateral dimension of 5 inches and a minimum wall thickness of three-fourths inch. Strictly enforced, this prevents the use of modern cast-iron pipe machinery and practice for the production

of columns, and by making columns more or less special as compared to the volume of pipe production increases their cost. Pipe machinery is usually adjusted to a wall thickness in 4-inch sizes of from 0.4 to 0.5 inch, and overall diameters are thus from 4.8 to 5.0 inches plus. It is obvious that if the same amount of quality of metal is present in a given column, that having the thinner wall will have the greatest radius of gyration and, within reasonable limits, the greatest stability. A minimum wall thickness of 0.4 inch, or one-twelfth of minimum diameter, and a minimum lateral dimension of 4.75 inches, therefore, are suggested as combining safety with economy.

6. The few codes which specify a tolerance in wall thickness put it at one-eighth inch to one-fourth the wall thickness. In view of improvements in casting practice such a wide tolerance is no longer necessary, and in consideration of the recommended reduction in wall thickness requirements it is undesirable, especially since variation on one side of a column often is accompanied by a corresponding variation on the opposite side and the member, therefore, becomes doubly eccentric in its reaction to stress. A tolerance of not more than 0.08 inch is believed advisable. It is not too strict for columns cast vertically or by the centrifugal processes.

In view of standards suggested in other respects the acceptance of imperfect columns to be used under slightly lower stresses is not recommended. The extent and effect of imperfections can be determined only approximately, and there is the possibility that confusion will arise in the field as to the use of such columns.

Columns should not be coated or painted previous to their inspection, as this is apt to prevent detection of defects.

The American Society for Testing Materials Standard Specification for Cast-Iron Pipe and Special Castings, serial designation: A 44–04, requires that they shall be made of cast iron of good quality and of such character as shall make the metal of the castings strong, tough, and of even grain, and soft enough to satisfactorily admit of drilling and cutting. The metal shall be made without any admixture of cinder iron or other inferior metal, and shall be remelted in a cupola or air furnace. The castings shall be smooth, free from scales, lumps, blisters, sand holes and defects of every nature which unfit them for the use for which they are intended. No plugging or filling will be allowed.

Par. 9. Working Stresses in Structural Steel.

The question of the proper working stresses for steel to be used in the design of buildings resolves itself into two independent considerations: (1) What is the proper basic stress, with reference to the strength of the steel in common use? (2) What are the relative values of the different forms of stress?

(1) Basic Stress.—The basic stress is generally considered to be a direct axial stress in tension. To decide upon this it is necessary only to determine what grade of metal will actually be used and what factor of safety should be applied to the specified strength of this material.

Quality of steel.—With the first introduction of rolled steel on a commercial basis specifications commonly called for a "medium grade" having an ultimate strength in test specimens of from 60,000 to 70,000 pounds per square inch. During the last 20 years the use of this grade has given place almost entirely to the grade known as "structural," having an ultimate strength between 55,000 and 65,000 pounds per square inch. This strength is now called for by the standard specifications of the American Society for Testing Materials, the American Railway Engineering Association, and the American Society of Civil Engineers.

The American Society for Testing Materials recognizes two grades of steel which may be used for building purposes. The first, termed "structural steel for bridges," does not include metal made by the Bessemer process. The second, termed "structural steel for buildings," admits Bessemer steel on a par with open-hearth steel if the physical requirements for test specimens are met. It is understood that in this country the use of Bessemer steel for structural shapes has been practically discontinued; that the mills are accustomed to furnish and engineers and fabricators to require only open-hearth steel; and that as a consequence practically all steel supplied under specifications complies with the American Society for Testing Materials standard specification for bridge steel. In deciding upon working stresses, therefore, it is logical to assume that when steel is ordered to specification this is the grade which will be used.

The original choice of 16,000 pounds per square inch as a basic stress appears to have been predicated on the idea that this gave a "factor of safety" of four, the ultimate strength of steel then in use averaging about 64,000 pounds per square inch. If so, its original significance has long since disappeared through the general acceptance of lower ultimate limits. The unanimity with which the 16,000 pounds per square inch has been accepted, not only in this country but in Canada and Australia, and the persistence with which it has been retained are remarkable. Up to the last three or four years there has been little inclination to increase stresses for building construction above this base, but recently there has been a movement to secure the adoption of higher stresses in building codes. One of the arguments advanced for this increase is, that when the 16,000 pounds per square inch stress was first introduced the use of Bessemer steel was quite general, but that now the open-hearth product has practically superseded Bessemer in the rolled material used for structural work, and that the open-hearth product is much more uniform and reliable.

Structural steel according to the American Society for Testing Materials Specifications must have a yield point of at least 50 per cent of the ultimate strength, but in no case less than 30,000 pounds per square inch. If a maximum tensile stress of 18,000 pounds per square inch is adopted, as recommended in Part II, it affords a safety factor approaching 2.0 based upon the average yield point for test pieces as determined by recent extensive investigations by the American Society of Civil Engineers;[3] or a little over 3.0 referred to the specified minimum ultimate strength.

Factor of safety.—Neither of these factors, however, has a very definite meaning. The yield point is the intensity of stress at which the metal begins to undergo an appreciable permanent change in shape, but it is not by any means a point of failure in tension. In fact, since the yielding is in the direction of greatest stress, it tends to readjust the structure to its loads and, by correcting imperfections of workmanship or fit, to bring parts under less stress to the aid of those which have reached the yield point.

Neither is the ultimate tensile strength of small test specimens a true criterion of the limiting strength of a structure. Full size fabricated material can not be expected to develop the same strength per unit area as concentrically-loaded, parallel-edge specimens. The exact point of failure of tension members as actually used is not very clearly established. It is well above the yield point but below the ultimate of tensile specimens; that is, it lies between 30,000 and 65,000 pounds per square inch. An assumption of 40,000 pounds per square inch is thought to be on the safe side and not ultra-conservative. It gives a safety factor for full-sized members of 2.2 with a working stress of 18,000 pounds per square inch.

In considering the necessity of allowing so high a factor as this it must be remembered that close investigation shows the usual data on the tensile strength of small test specimens are not necessarily a close indication of the actual strength of the material represented. The customary method of acceptance testing, for example, specifies yield point determinations. Those familiar with mill testing methods know that it is difficult to get check results by different operators. The testing conditions, speed of operation of testing machine, and personal equation of the operator all seriously affect the results obtained. Mill tests rarely agree with those conducted more carefully in standard testing laboratories.

There is the further question of how well test specimens really represent the material supplied. Commonly in channels and I beams these are taken from the webs, the most worked, thinnest,

[3] See p. 7 of Report of Special Committee on Stresses in Structural Steel; January, 1925.

and therefore quickest cooled portion of the section. In plates, test pieces generally come from the edges. In the latter it is a rare thing to call for transverse specimens; that is, taken perpendicular to the direction of rolling. Careful tests on rolled sections show material at the fillet or root portions of the section of decidedly lower strength. Transverse test specimens also show lower strengths.

Again, yield point determinations do not give, except in a general way, information of the true behavior of a steel under stress. For similar grade that material having the higher yield point will possess the higher proportional limit. The relative value of the proportional limit can not be determined, however, from yield point tests of steel. In fact, it is doubtful whether any two specimens taken from the same grade of steel will show like stress-strain characteristics, and the most marked differences will come in the evaluation of the proportional limit stress. The proportional limit stress is at present practically ignored in acceptance testing. It is, therefore, given slight consideration in the determination of working stresses.

Whether this procedure is safe may be questioned. For example, it has been proven by test that if two specimens cut from the same sample of metal in adjacent positions are tested in compression, one without any other strain than compression, the other having been subjected to an overstrain in tension before being tested in compression, the former will show an entirely different stress-strain curve from the latter. The overstrain in tension will almost completely destroy the elasticity of the metal in compression.

The question of safety factors and maximum allowable stresses for design of steel structures has been much agitated in the last few years. A committee of architects and engineers retained by the American Institute of Steel Construction has recommended for general practice the basic stress in tension of 18,000 pounds per square inch, with other stresses in proportion. This schedule of stresses has been quite widely adopted, both in building codes and in private practice where codes do not control. Its more or less complete acceptance in over 40 cities, after considerable investigation and the canvassing of local technical opinion in numerous cases, may be accepted as indicating general approval by those experienced in construction.

A special committee also was appointed by the American Society of Civil Engineers in August, 1922, to consider working stresses for structural steel in buildings. The report of this committee, presented at the January, 1925, meeting of the society, recognizes the possibility of increasing the basic structural steel stress, due to static loads, not only to 18,000 but to 20,000 pounds per square inch. The precautions with which the American Society of Civil Engineers committee's approval of this latter stress is qualified, however, are believed such as to make its use unwise at present under the diversity of circumstances comprehended by a building code.

Undoubtedly, under present conditions, a somewhat closer evaluation of the relation between strength of test specimens and working stresses employed in structural design is possible than that upon which the 16,000 pound per square inch tensile stress originally was based. There is general agreement that present-day manufacturing processes result in a more uniform grade of material; that there is less danger, both by reason of the quality of material and the perfection of fabricating processes, that structural shapes will be injured in fabrication or erection; and that the knowledge of design by which the stresses in steel are accurately determined is more complete and widespread than during the earlier years in which the 16,000 pounds per square inch basic stress was generally used. The weight of engineering opinion, as obtained through submission of the committee's tentative report, favors the basic stress of 18,000 pounds per square inch for steel, the quality of which is definitely established. In the absence of experimental results to the contrary and in view of the conclusions arrived at by other recent investigations this consensus is accepted as authoritative.

Where evidence is lacking that the material meets the American Society for Testing Materials standard specifications for steel for buildings it is necessary to consider what working stresses may be allowed. Building codes should provide for this contingency, since in a large proportion of buildings of types in which structural steel is not an important factor, material is commonly purchased from the most available source, regardless of specification. It has been argued that under such circumstances it is unsafe to allow any definite schedule of stresses. On the other hand, there does not appear to be sufficient reason for not allowing the stresses generally in use for many years and which have produced satisfactory results.

(2) Relative Stresses.—In building codes, as well as in specifications for design, it is common to find an elaborate list of different working stresses for varying conditions of stress. The important ones are: (1) Direct tension, (2) direct compression; that is, axial compression in columns of moderate length in proportion to their stiffness, (3) fiber stress in beams under flexure, (4) shear in webs, (5) shear in rivets and pins, and (6) bearing upon rivets and pins. The last two are generally subdivided according to the way the rivets are driven, and often other working stresses are given, such as fiber stress in pins and rivets, and stresses upon bolts.

In comparing these tables in codes and specifications it appears that there is substantial unanimity of practice with two exceptions—the working stress for compression in short columns and rivet and bolt stresses.

Much experimental work has been done upon columns, but not enough to settle definitely the proper relation between tension and compression stresses. It is generally accepted, however, that the ultimate strength of columns of which the ratio of length to least radius of gyration does not exceed 60 is very close to the yield point of the metal in compression; and that this yield point is approximately the same as the yield point of tensile specimens. There is a very important difference, however, between the way tension members and compression members fail. When a tension member reaches its elastic limit it remains straight and stretches, but with the increase in length there is a gain in resistance. When a compression member, on the other hand, reaches its elastic limit, or very shortly thereafter, it buckles and gives way suddenly. For this reason, it is safe to consider the point of absolute failure of tension members as considerably above the yield point of the steel, while for compression members the yield point must be assumed as the point at which failure is likely to ensue.

In order, therefore, that compression members may be as safe, relatively, as tension members, with reference to their probable points of failure, it is necessary that the maximum compressive stress be less than that permitted in tension. In the more modern specifications are found the ratios of 75, 78, 83, and 90 per cent for this relation. In view of the uncertainty as to whether the factor of 2.2 in tension is actually obtained and considering also the weight of qualified opinion, the committee has adopted a maximum stress of 14,000 pounds per square inch in compression. This represents a ratio of about 78 per cent between maximum compression and tension, and approximates the ratio of 78⅛ per cent used by the American Railway Engineering Association.

As to values for rivets and bolts, working stresses are given in codes for a variety of conditions, such as shop driven, power driven, hand driven, inclosed bearing, uninclosed bearing, turned bolts, and rough bolts. While all these conditions have shades of difference, there does not appear to be any necessity for specifying different working stresses for all. The proportion of hand-driven rivets is decreasing rapidly and where inspection is careful the difference between rivets power driven in shop and field is negligible.

Column reduction formula.—When the slenderness ratio, $\frac{L}{r}$, becomes greater than about 60, a reduction in the stress should be made on account of the increasing possibility of bending. For this reason it is universal practice to set a limit of slenderness beyond which no stress shall be allowed. This limit is frequently placed at $\frac{L}{r}$ equals 120—sometimes at 120 for main members and at 160 for secondary members. Theoretically this limit might be much higher, but on account of the increasing danger of the column being accidentally bent out of direct line a restriction is usually considered desirable.

The general opinion among engineers reviewing the tentative report was inclined to favor the traditional limit of about 120 for $\frac{L}{r}$ of main members, with higher limits in most cases for bracing and other "secondary" members. With the reduction in stress imposed by the formula in Part II, however, and with proper care to insure alignment, the higher limit is believed safe for main members.

Columns more than usually slender are more subject to accidental bending, and should be shipped and handled with care. Continuous columns should be given preference wherever possible over those one story in height. In designing inclined or horizontal compression members of extreme slenderness, allowance should be made for stress due to weight of the member.

Fabrication and erection should be in accordance with the Code of Standard Practice adopted by the American Institute of Steel Construction which requires that compression members shall not have a lateral variation greater than 1 to 1,000 of the axial length between the points which are to be laterally supported.

At a limit of $\frac{L}{r}$ equals 160 it is shown by tests and by the consensus of specifications that the working stress may be conservatively placed at one-half that used for short columns. Between the values of $\frac{L}{r}$ equals 60 and $\frac{L}{r}$ equals 160 the reduction may be made proportional, which is the case when a "straight line" formula is used; or it may be made by means of a curve after the pattern of the Rankine formula. Both methods have their strenuous advocates, but the difference between the results obtained by the two methods is insignificant compared with the variation between the different formulas recommended by various authorities (see fig. 3). The straight line formula $18{,}000-70\frac{L}{r}$ gives a stress of 14,000 pounds per square inch when $\frac{L}{r}$ equals $57\frac{1}{7}$ and a stress of 6,800 pounds per square inch when $\frac{L}{r}$ equals 160. The Rankine type formula which coincides at these two points is

$$f \text{ equals } \frac{16564}{1 \text{ plus } \frac{L^2}{17828\, r^2}}$$

The greatest divergence between these two formulas is at about the value of $\frac{L}{r}$ equals 110, where it amounts to 437 pounds per square inch or about $4\frac{1}{4}$ per cent of the stress.

The formula f equals $18{,}000-70\frac{L}{r}$ is very simple, easily remembered, and is sufficiently above the average of those now generally in use to be consistent with the increase in tensile and other stresses.

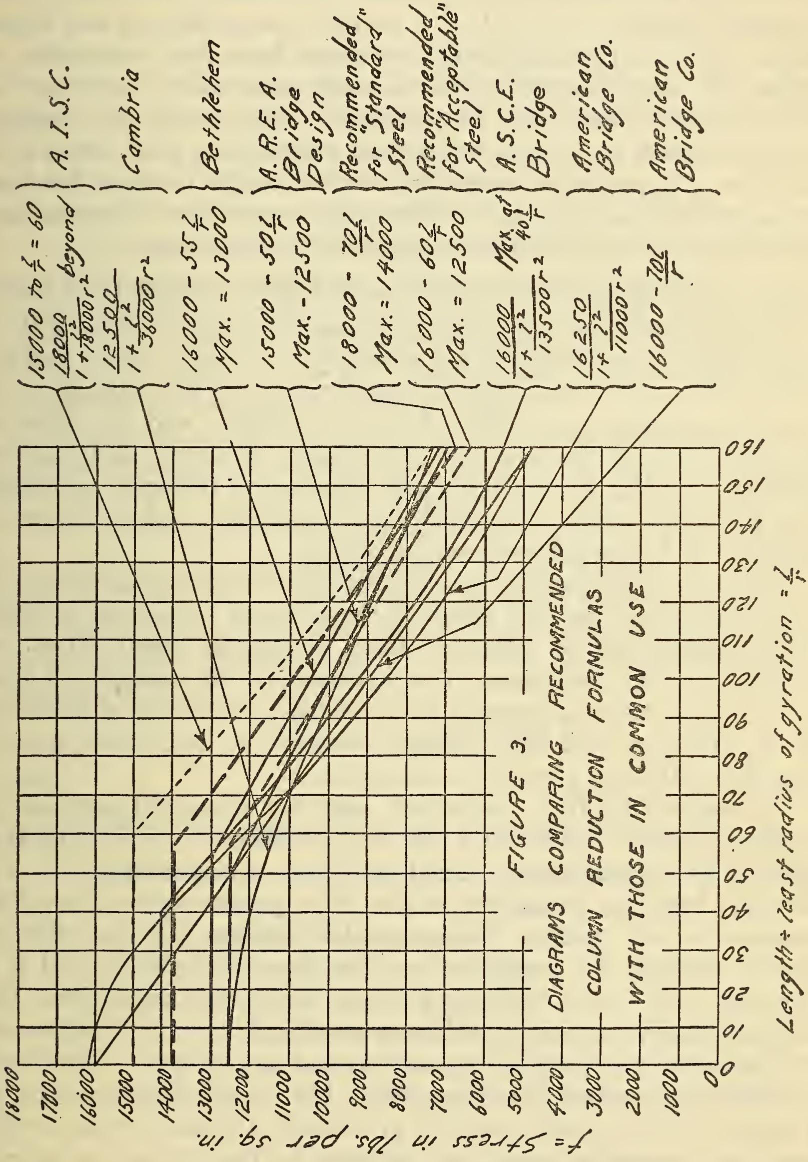


FIGURE 3. DIAGRAMS COMPARING RECOMMENDED COLUMN REDUCTION FORMULAS WITH THOSE IN COMMON USE

Formulas giving higher results are not recommended. Those plotted in Figure 3, the curves of which lie below that recommended by the committee, will give safe results, but in the interests of uniform code practice and of economy the committee urges that recommended in Part II be adopted.

Eccentric loading.—Any eccentricity of column loading should be taken into consideration, as the working stresses for columns are based upon the assumption of a purely axial stress. The effect of an eccentric loading is to produce a combination of flexure and direct stress. In case the maximum fiber stress from the combination is within the value allowed for the bending stress alone, however, the column section need not be increased. The reason for limiting column stresses to a figure lower than the bending fiber stress is to allow for a hypothetical eccentricity which might increase the fiber stress to the bending limit. Where the eccentricity is known and allowed for, the hypothetical assumption is unnecessary.

Par. 10. Factors Involved in Determining Safe Stresses for Timber.

The determination of working stresses for timber which will be safe under all circumstances and comparable in usefulness to working stresses for other materials is complicated by the variability of strength of clear wood, the influence of defects, and the differing strength of wood under various conditions of service and loading. Defects have the most pronounced influence on strength, variations due to service and loading conditions come next, and variations in strength of clear wood are least important.

Information obtained by research during the last 20 years has made it possible to account for many of the strength variations of structural timbers and to allow for them or control them. Data are available on the variations to be expected in the strength of clear wood, the decrease in strength due to defects, the effects of moisture, stress duration, and the average strengths of individual species, together with their strength characteristics.

In view of the fact that woods used for structural purposes in different parts of the country are of quite different species and that Part II is in form recommended for State or city adoption, it was thought best not to include in Part II a general table of working stresses for all species. Recommended stresses for the principal species suitable for structural use are given in Table 7, and it is expected that those revising building codes will select from this table the species used or which may possibly be used in their respective localities and place such a table in the code proper. The Forest Products Laboratory, Madison, Wis., has compiled information on numerous other species, and in practically all cases recommended working stresses for woods not included in Table 7 can be secured from the laboratory. When this is not possible, information should be obtained on locally used woods so that they may be classified in accordance with similar woods listed in the table.

Certain differences will be observed between the stresses here recommended and those specified in section 23 of the Building Code Committee's report on "Recommended Minimum Requirements for Small Dwelling Construction." These differences are primarily due to a change in grade, but in a few instances additional research has caused minor changes.

The structural grading rules published by the United States Department of Commerce in revised Simplified Practice Recommendation No. 16, "Lumber,"[4] have been substituted for those of the Southern Pine Association as representing a broader and more general agreement of the lumber industry.

A greater stress in compression perpendicular to the grain is allowed when the joists rests upon a ribbon or ledger board than when the ends rest upon or in masonry. This is because the stress is not at the extreme end of the joist and in the case of a slight overload they will not yield so much; the joists are spiked to the studding, thus gaining some support through the nails, and they are more certain of being dry and are consequently stronger.

Compression stresses parallel to grain for short columns are of considerable importance in most formulas for calculating the strength of columns of the intermediate-length class. A study of such formulas is being made in connection with a series of tests on long, short, and intermediate-length columns now under way at the Forest Products Laboratory. The tests indicate that a fourth-power parabola tangent to the Euler curve is a conservative representation of the law controlling the strength of columns of intermediate length. Its equation is

$$\frac{P}{A}=C\left(1-\tfrac{1}{3}\left(\frac{L}{Kd}\right)^4\right)$$

where

$\frac{P}{A}$ = allowable column stress, in pounds per square inch,

C = safe stress for the material in compression parallel to grain, in pounds per square inch, for short columns,

L = unsupported length of column, in inches,

d = least dimension of column, in inches,

E = modulus of elasticity (see Table 7),

K = a constant dependent on the modulus of elasticity and the maximum crushing strength parallel to grain, which in turn vary with the species and grade. (See Table 7 for K.)

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[4] U. S. Department of Commerce, Elimination of Waste Series, Simplified Practice Recommendation No. 16, "Lumber" (revised May 1, 1925), can be obtained from the Superintendent of Documents, Government Printing Office, Washington, D. C., for 15 cents, cash or money order.

That is, from the short block to the long column whose strength is dependent on the stiffness, there is a gradual falling off in the ultimate load. The falling off follows a smooth curve which is very flat at first and which curves sharply to meet the Euler curve at two-thirds of the ultimate compressive strength of the species in question. K in the formula represents the value of $\frac{L}{d}$ at the point of tangency.

Investigations have shown that the strength of square-end columns is practically identical with that of carefully centered pin-end columns up to a limit of $\frac{L}{d}$ equals 11; and that from that value onward any increase in strength in square-end columns over pin-ends is in very large measure dependent upon the condition of the bearing surface. On the whole, therefore, the information available as to the influence of square-end conditions on the strength of wooden columns does not, in the committee's opinion, justify increasing the stress over that for carefully centered pin-end columns.

The investigation of the strength of columns indicates that the Euler formula is quite accurate for long wooden columns with pin-end connections and that the maximum load which they will carry is dependent upon their stiffness. The Euler formula for rectangular columns with the factor of safety of 3 included reduces to

$$\frac{P}{A}=\frac{0.274E}{\left(\frac{L}{d}\right)^2}$$

TABLE 7.—*Working stresses for timber*

[Select grade,[1] beams and stringers, posts, and square timbers, joists and rafters for use under shelter in dry locations] [2]

Species	Allowable stress				Compression parallel to grain, intermediate columns; values of [5] K		Average for [6] modulus of elasticity. All grades
	Bending		Compression perpendicular to grain. All grades	Compression parallel to grain; short columns. Select grade			
	In extreme fiber.[3] Select grade	Horizontal shear.[4] Select grade			Select grade	Common grade	
	Lbs./in.^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}	*Lbs./in.*^{2}
Ash, black	1,000	90	300	650	26.4	29.5	1,100,000
Ash, commercial white	1,400	125	500	1,100	23.7	26.5	1,500,000
Aspen and large-tooth aspen	800	80	150	700	23.0	25.7	900,000
Basswood	800	80	150	700	23.0	25.7	900,000
Beech	1,500	125	500	1,200	23.4	26.2	1,600,000
Birch, paper	900	80	200	650	25.2	28.1	1,000,000
Birch, yellow and sweet	1,500	125	500	1,200	23.4	26.2	1,600,000
Cedar, Alaska	1,100	90	250	800	24.8	27.8	1,200,000
Cedar, western red	900	80	200	700	24.2	27.1	1,000,000
Cedar, northern and southern white	750	70	175	550	24.5	27.3	800,000
Cedar, Port Orford	1,100	90	250	900	23.4	26.2	1,200,000
Chestnut	950	90	300	800	22.7	25.3	1,000,000
Cottonwood, common and black	800	80	150	700	23.0	25.7	900,000
Cypress, bald	1,300	100	350	1,100	21.2	23.7	1,200,000
Douglas fir (western Washington and Oregon)[7]	1,600	90	345	1,175	23.7	27.3	1,600,000
Douglas fir (western Washington and Oregon), dense	1,750	105	380	1,290	22.6	24.9	1,600,000
Douglas fir (Rocky Mountain type)	1,100	85	275	800	24.8	27.8	1,200,000
Elm, cork	1,500	125	500	1,200	21.1	23.6	1,300,000
Elm, slippery and white	1,100	100	250	800	24.8	27.8	1,200,000
Fir, balsam	900	70	150	700	24.2	27.1	1,000,000
Fir, commercial white	1,100	70	300	700	25.4	28.4	1,100,000
Gum, red, black, and cotton	1,100	100	300	800	24.8	27.8	1,200,000
Hemlock, western	1,300	75	300	900	25.3	28.3	1,400,000
Hemlock, eastern	1,100	70	300	700	25.4	28.4	1,100,000
Hickory (true and pecan)	1,900	140	600	1,500	22.2	24.8	1,800,000
Larch, western	1,200	100	325	1,100	22.0	24.6	1,300,000
Maple, sugar and black	1,500	125	500	1,200	23.4	26.2	1,600,000
Maple, red and silver	1,000	100	350	800	23.8	26.6	1,100,000
Oak, commercial red and white	1,400	125	500	1,000	24.8	27.8	1,500,000
Pine, southern yellow [7]	1,600	110	345	1,175	23.7	27.3	1,600,000
Pine, southern yellow, dense	1,750	120	380	1,290	22.6	24.9	1,600,000
Pine, white, sugar, western white, western yellow	900	85	250	750	23.4	26.2	1,000,000
Pine, Norway	1,100	85	300	800	24.8	27.8	1,200,000
Poplar, yellow	1,000	80	250	800	23.8	26.6	1,100,000
Redwood	1,200	70	250	1,000	22.2	24.8	1,200,000
Spruce, red, white, Sitka	1,100	85	250	800	24.8	27.8	1,200,000
Spruce, Englemann	750	70	175	600	23.4	26.2	800,000
Sycamore	1,100	80	300	800	24.8	27.8	1,200,000
Tamarack (eastern)	1,200	95	300	1,000	23.1	25.8	1,300,000

[1] Specifications for the select and common grades were published in Simplified Practice Recommendation No. 16 (Revised), May 1, 1925. Working stresses for the common grade of Douglas fir (western Washington and Oregon), and southern yellow pine are 75 per cent of those given for the select grade (not dense); for other species 80 per cent of the select grade, except in compression perpendicular to grain and modulus of elasticity, which are the same for all grades. The difference in the percentage between the select and common grades of Douglas fir and southern yellow pine as compared with other species is due to the fact that the rate of growth requirement for the select grade applies only to these two species.

[2] See introductory paragraph, Part II.

[3] Stress in tension along the grain. The stresses in extreme fiber in bending are identical with those recommended for tension parallel to grain.

[4] Shearing stress for joint details. The shearing stress for joint details for all grades may be taken as 50 per cent greater than the horizontal shear values given in the table.

[5] K is for use in Table 8 for determining the safe stress for columns of intermediate length. It is the $\frac{L}{d}$ at which the F. P. L. fourth power parabola becomes tangent to the Euler curve.

[6] Values for modulus of elasticity are average for species. In figuring safe loads for long columns a factor of 3 must be applied to them.

[7] If material of common grade of southern yellow pine or Douglas fir conforms to the density requirements of the American Society for Testing Materials the stresses in compression (both parallel and perpendicular to grain), in extreme fiber in bending and in horizonal shear may be increased by 15 per cent of the value listed for the select grade.

Source: U. S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, Wis. (Revised), January 20, 1926.

TABLE 8.—*Strength of columns of intermediate length in per cent of strength of short columns* [1]

Values of K[3]	Ratio of length to least dimension in rectangular timbers [2]																	
	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
	P. ct.	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P.ct.*	*P. ct.*
21	96	95	93	91	89	86	82	78	73	67	-----	-----	-----	-----	-----	-----	-----	-----
22	97	96	95	93	91	88	85	81	77	72	67	-----	-----	-----	-----	-----	-----	-----
23	98	97	95	94	92	90	87	84	81	77	72	67	-----	-----	-----	-----	-----	-----
24	98	97	96	95	93	92	89	87	84	80	76	72	67	-----	-----	-----	-----	-----
25	98	98	97	96	94	93	91	89	86	83	80	76	72	67	-----	-----	-----	-----
26	99	98	97	96	95	93	92	91	89	86	83	80	76	72	67	-----	-----	-----
27	99	98	98	97	96	95	93	92	90	88	85	82	79	74	71	67	-----	-----
28	99	98	98	97	96	95	94	93	91	89	87	85	82	79	75	71	67	-----
29	99	99	98	98	97	96	95	94	92	91	89	87	84	82	79	75	71	67

[1] These percentages are values of the expression $\left(1-\frac{1}{3}\left(\frac{L}{Kd}\right)^4\right)$ for various values of K and $\frac{L}{d}$. When the $\frac{L}{d}$ exceeds K, the column is in the Euler class and must be designed by the formula $\frac{P}{A}=\frac{0.274E}{\left(\frac{L}{d}\right)^2}$.

[2] *Columns not rectangular.*—This table can be used for other than rectangular columns, the $\frac{L}{d}$ being equivalent to $0.289\frac{L}{r}$ where r is the least radius of gyration of the section.

[3] Values of K are found in Table 7 opposite the species and under the grade.

SOURCE: U. S. Department of Agriculture, Forest Service, Forest Products Laboratory, Madison, Wis., January 18, 1926.

Table 8 is given to simplify the use of the fourth-power parabolic equation for determining the permissible load for columns of intermediate length. It is a tabulation of values for the expression

$$\left(1-\frac{1}{3}\left(\frac{L}{Kd}\right)^4\right)$$

The values for this expression are given as percentages, which when applied to the allowable stresses in compression parallel to grain for short columns (Table 7) will give the permissible stresses for columns of intermediate length with the respective ratios of length to least dimension.

Use of Table 8 to determine safe load on columns of intermediate length

To determine the load a given column will safely carry:

First. Obtain from Table 7 the value of K for the species and grade of timber used;

Second. Determine its $\frac{L}{d}$ or slenderness ratio; that is, its length divided by its least dimension; [5]

[5] If the $\frac{L}{d}$ is found to be 11 or less, the column is considered as "short" and the load it may support is determined by multiplying its area by the allowable stress for short columns. (See Table 7.) If, for a column of given size and length, the $\frac{L}{d}$ exceeds K, then the column is in the Euler formula class, which means its carrying capacity is dependent on it stiffness, and its strength must be determined by the formula

$$\frac{P}{A}=\frac{0.274E}{\left(\frac{L}{d}\right)^2}$$

Third. At the side of Table 8 find the horizontal line beginning with the value of K just determined;

Fourth. In this line and vertically below the $\frac{L}{d}$ of the column under consideration will be found the percentage which, when applied to the allowable compression stress for short columns of that species and grade (see Table 7) will result in the permissible stress for the given column;

Fifth. Multiply the actual area of the column by the stress just found. This will give the total load the column will safely support.

Example: Required to find the safe load for a Douglas fir or Southern yellow pine column, common grade, 8 by 8 inches in nominal section, and 14 feet long. The slenderness ratio $\frac{L}{d}$ should be based on the actual size, which is 7.5 by 7.5 inches:

$$\frac{14 \times 12}{7.5} = \frac{168}{7.5} = 22.4$$

In Table 7 the value K for the common grade of Douglas fir or Southern yellow pine is given as 27.3. In Table 8, in the line opposite $K=27$ and below the slenderness ratio of 22, we find the percentage 85. Referring again to Table 7, the allowable stress for short columns of select grade is found to be 1,175 pounds; the stress for common grade being 75 per cent of this, or 880. The permissible stress for the intermediate column with $\frac{L}{d}$ equal to 22, is, therefore, 880×85 per cent, or 748 pounds per square inch. This is the allowable working stress, and multiplied by the column's cross-section area, 7.5 by 7.5, or 56.25 square inches, gives the safe total load for the column—42,200 pounds.

Use of Table 8 to determine the required size of a column of given intermediate length to support a given load

The following procedure affords a close approximation of the necessary size: [6]

First. Determine the cross section of a square timber necessary to sustain the load if the timber were a short column of the intended species and grade.

Second. Determine the slenderness ratio of that size of column for the given length.

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[6] Except when $\frac{L}{d}$ is less than 11, the solution of a quadratic equation is necessary, even with a straight line formula, to determine directly and accurately the required column size for a given load and length. A similar method would have to be pursued in the solution of the problem by the fourth power parabola equation. It is consequently easier to resort to the trial method for solution.

Third. From Table 7 ascertain the proper K for the species and grade.

Fourth. In Table 8 find the percentage for this $\frac{L}{d}$ opposite the proper K for the species and grade. The area found for the short column divided by this percentage gives the necessary cross-sectional area for the required column (column of given length).

It is understood, of course, that the least dimension of the required column must be equal to the least side of the trial short square column in order to leave the slenderness ratio unchanged. A square column of the adjusted area will carry slightly more than the given load. The dimensions of a square column of the adjusted area will equal the square root of the area. For rectangular columns, one dimension will be the same as the side of the trial column and the other will equal the required area divided by that dimension.

Example.—Required to find the necessary cross-sectional size for a 14-foot Douglas fir column, common grade, to carry a 50,000 pound load. The allowable stress for a short column of that species and grade is 880 pounds per square inch (75 per cent of the stress for select grade in Table 7).

First. $\frac{\text{Load}}{\text{Stress}}=\frac{50,000}{880}=56.80$ square inches cross-sectional area necessary for a short (trial) column.

Second. $\sqrt{56.80}=7.5=$side of the short square trial column,

$$\text{Trial } \frac{L}{d}=\frac{\text{given length}}{\text{side of trial square}}=\frac{14\times 12}{7.5}=\frac{168}{7.5}=22.4,$$

Third. $K=27.3$ from Table 7,

Fourth. If $\frac{L}{d}=22$ and $K=27$ then from Table 8 the working stress is 85 per cent of that allowable for the short trial column and the adjusted area of the required column is $\frac{56.80}{0.85}$ or 66.8 square inches.

Fifth. $\frac{\text{Adjusted area}}{\text{Side of trial column}}=\frac{66.8}{7.5}=8.9$; therefore, a rectangular column that is 7.5 by 8.9 inches in size will carry the load.

The dimension of a square column of equivalent area equals $\sqrt{66.8}$ or 8.16 which is slightly larger than necessary. The next lower commercial size is 7.5 by 8.5 actual dimensions, which as is evident will not carry the load without exceeding the permitted stress. Hence, the nearest commercial sizes which will carry the load safest are 8 by 10 (7½ by 9½ actual) or 9 by 9 (8½ by 8½ actual).

If the trial $\frac{L}{d}$ is greater than K, then determine the necessary size of a short square column using two-thirds the safe working stress for that species and grade; that is, proceed through the same steps as were used in determining the original trial column and trial $\frac{L}{d}$ but with two-thirds the allowable stress used before. If the trial $\frac{L}{d}$ thus determined is still more than K, then the column is definitely in the Euler class. If, however, this second trial $\frac{L}{d}$ is less than K, then as a trial use the next larger $\frac{L}{d}$ in Table 8 and solve as an intermediate column.

The foregoing examples have been treated mathematically in detail with the purpose of illustrating quite fully the application of the fourth power parabola and the Euler curve to the determination of column sections. It is realized that for the purposes of building design and building code enforcement a simpler and more direct statement of requirements is necessary. Table 4 following section 5–5 in Part II, therefore, has been worked out on the basis of the formulas heretofore discussed. In addition to the working stresses for short columns it gives the safe working stresses for intermediate and long columns of various species grouped according to the values of K and the respective working stresses for short columns. Safe working stresses for columns of $\frac{L}{d}$ intermediate between the values given in Table 4 may be obtained by interpolation.

Par. 11. Variables Affecting Strength of Clear Wood.

The clear wood from trees of different species varies in strength. The part of the tree from which the wood is cut and the characteristics of the tree also influence strength. Such variations, however, especially differences in strength between different species, are neither so great nor so important in comparison with other factors as is generally believed. The differences in the strength of the various species are usually taken care of by specifying working stresses for individual species. The enormous amount of testing necessary to determine the average strength of the clear wood of the various native species has been largely completed, and the results for 126 species have been published.[7] Data on approximately 35 additional species are available but not yet published. There is ample information as to the average strength of clear wood of species commonly used in building.

Variations within a species, however, are such that it is not uncommon to find one piece twice as strong as another, although both are sound, clear, and straight grained, and at the same moisture content. Such a variation is usually associated with a difference in density

[7] These data are given in U. S. Department of Agriculture Bulletin 556, "Mechanical Properties of Woods Grown in the United States."

and may be accounted for either by a difference in position in the tree, tree characteristics, or a combination of both.

Figure 4 is a frequency curve for modulus of rupture of Sitka spruce. It is based on the results of 1,374 tests of material from seven different localities. This graph is representative of the normal variation of most species, in which the most probable value is some 6 or 7 per cent below the arithmetic mean; 25 per cent of the material is more than 14 per cent below the mean; and 25 per cent more than 12 per cent above the mean. These differences are usually associated with differences in density.

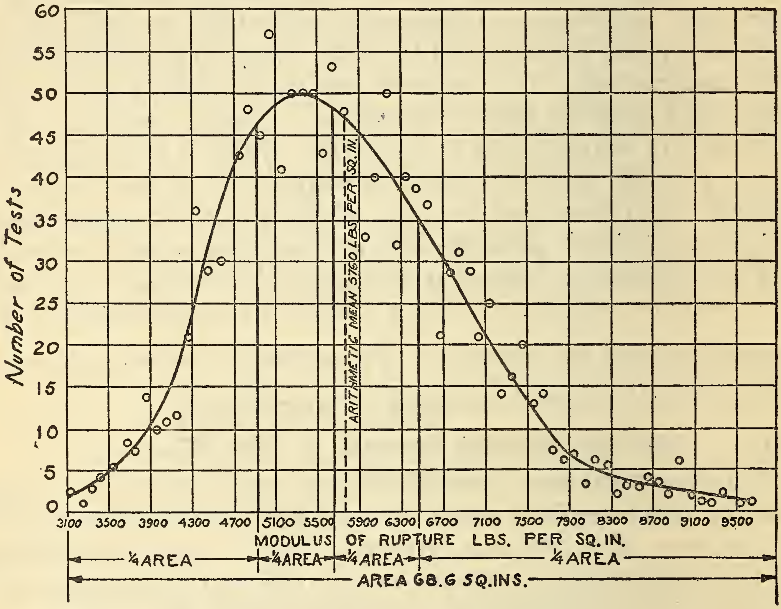

FIG. 4.—*Variability in modulus of rupture of clear wood, green Sitka spruce*

U. S. Department of Agriculture, Forest Service, Forest Products Laboratory.

Although density is the best indication of the strength of clear wood of all species, no satisfactory visual method of estimating it has been devised except for Douglas fir and southern yellow pine, in which the percentage of summer wood present is a good criterion of density. Specifications for high quality dense material of these two species, which produce most of our structural timber, are published in the Department of Commerce Simplified Practice Recommendation No. 16 (revised). The present specifications of the American Society for Testing Materials [8] are being revised so that they will be in absolute conformity. Percentage of summer wood is not generally applicable to other species as a test of quality.

[8] Given in A. S. T. M. "Tentative Standard 1920, serial D 23–20 T for Douglas fir; for Southern pine in serial D 10–15 A. S. T. M. Standard 1921." The latter standard applies to bridge and trestle timbers, but the principle may be used for other grades.

Par. 12. Influence of Defects on Strength of Timber.

As already remarked, the greatest variation in the strength of timbers results from defects. Defects often occur which reduce the strength of the piece to practically zero.[9] The control of defects in accordance with their influence on strength is, therefore, essential before safe and practical working stresses can be assigned to timber.

Data on the influence of defects on strength of wood have made possible the preparation of grading rules by which structural timbers are classified in accordance with their strength and such control is accomplished. Unfortunately, most grading rules formerly in use for structural timbers were very general in their application, and did not limit defects in accordance with their effect on strength. As a step toward improving this condition, the Forest Service has lately prepared basis rules for grading structural timbers,[10] embodying the fundamental principles of strength grading. Various committees of the American Society for Testing Materials and the American Railway Engineering Association, as well as the Central Committee on Lumber Standards, have accepted the principles therein set forth, together with certain additions agreed to by the Forest Service as the basis for strength grading. As a result of the work of these committees, the Central Committee on Lumber Standards has adopted select and common grades for beams and stringers, and select and common grades for joists and plank, the select grade to insure 75 per cent and common grade 60 per cent of the green strength of clear wood. These rules are published in United States Department of Commerce Elimination of Waste Series, Simplified Practice Recommendation No. 16, "Lumber," revised May 1, 1925.

Par. 13. Moisture Conditions of Service.

The strength of wood is influenced largely by its moisture content and, therefore, by the moisture conditions of service, which have an important bearing also on decay and checking.

The loss of moisture accompanying seasoning differs in its effect upon strength according to the size of the material. Clear wood in small sizes increases in strength with loss of moisture, the increase being much greater for some properties than for others. Thus, compressive strength of pieces 2 by 2 inches in cross section may increase 100 per cent or more from the green to the air-dry condition, and bending strength 50 per cent or more, whereas impact and work to rupture values seldom increase by more than 20 per cent and may even show a loss in some species.

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[9] Discussion of defects and their influence is given in the 1924 Proc. of the A. S. T. M. under the title "Structural Timbers," by J. A. Newlin and R. P. A. Johnson.

[10] U. S. Department of Agriculture Circular 295, "Basic Grading Rules and Working Stresses for Structural Timber," by J. A. Newlin and R. P. A. Johnson.

Studies of moisture effects on strength have shown large timbers to be about as strong in a green as in an air-dry condition, and have also shown the increase in strength with drying which can be expected in higher grades of dimension and in small clear material. With larger pieces, the checking which accompanies drying increases in severity, especially as knots increase in number and size. Thus, clear dimension material 2 to 4 inches thick increases in strength about 25 per cent on drying, but material of common grade increases only about 8 per cent. Owing to injury from checking, the lower grades of large timbers and about 25 per cent of large timbers of higher grades show no increase in strength with drying, although the average strength of the latter is raised slightly.

Timber in wet or damp locations is subject to deterioration from decay. Under such conditions lower stresses are recommended in order to offset the effects of depreciation and to avoid the danger which might result from failure to replace the member frequently.

Par. 14. Conditions of Loading.

Variations in strength with different conditions of loading result principally from difference in the duration of stress. The studies on the effect of duration of stress on strength of wood make is possible to know the factor of safety under varying conditions from impacts of a fraction of a second to quiescent loads of several years' duration. The variation in strength with duration of stress can be figured approximately by the rule that a known modulus of rupture will increase about 10 per cent if the duration of the stress is decreased to one-tenth of the original period.[11] Thus, a timber will carry 10 per cent more load for 1 hour than it will for 10 hours.

Loads which are carried without failure for a few seconds may, therefore, cause failure if applied for a long period. For example, a locomotive on a smooth track can pass safely over a bridge at 60 miles per hour, giving the ordinary impact for such a speed, when the same locomotive might cause failure if left in a position to cause a maximum stress for a considerable length of time.

Par. 15. Working Stresses for Timber.

The working stresses given in Table 7 are based on the general conclusion of the Forest Service, drawn from tests of structural timbers, that defects, such as knots, shakes, checks, and cross grain, have approximately the same influence on the strength of timbers from all species. Therefore, working stresses in a given case can be arrived at by adjusting the strength of clear wood in accordance with the known effect of permitted defects and the duration of stress, at the

[11] This rule of thumb was obtained from H. S. Grenoble's work on "Effect of Rate of Loading on Strength of Wood," and from dead load and impact tests. Some of this information is given in the 1909 Proceedings of the American Society for Testing Materials under the title "Some Results of Dead Load Bending Tests of Timbers by Means of a Recording Deflectometer," by H. D. Tiemann.

same time making allowance for probable variations of individual timbers from the average. As a matter of fact, characteristic differences in the nature and prevalence of defects in various species have been taken into account both in the wording of the rules and in the stresses permitted. Thus, for some species in which the limbs are borne in whorls rather than alternately, the rules are so worded as to exclude timbers in which whorls of knots would cause greater weakening than the allowable single knot. Again, if one species runs characteristically lower in the grade than another, consideration has been given that fact in the assignment of working stresses.

The differences in strength due to species are taken care of in the table by giving separate stresses for each species; difference in effect of seasoning on dimension and timbers of larger size is provided for by allowing slightly larger defects in dry locations for dimension timbers 4 inches and under in thickness. The stresses in Table 7 apply only for timber used in dry inside locations. Where timbers are exposed to moisture or to weather, design stresses should be decreased. (See Appendix, par. 13.)

The stresses shown in Table 7 are for long-time loadings.

Except in case of compression perpendicular to the grain these working stresses are based rather on ultimate than elastic limit values, although both are considered. There has been considerable discussion among engineers as to whether elastic limit or ultimate values should be used as the basis for safe working stresses. There is undoubtedly an element of safety in the stress which a timber is able to take after the elastic limit is passed. Those species which show a great difference between the elastic limit and the ultimate stress show also a large deflection before failure occurs, so that in one species a load may be applied slightly beyond the elastic limit and ample warning received before failure, while with another species the same loading condition would cause failure without apparent warning. When elastic limit values are used as a base no consideration is given to the differences in strength which exist in species between elastic limit and ultimate strength. In addition to this nonuniformity of behavior the elastic limit values are subject to more numerous test errors, to a greater "personal equation" of those operating testing machines, and to larger errors in calculation of stress values than are ultimate strength determinations. These facts are believed to justify the use of ultimate values as the primary base for safe working stresses in timber.

Par. 16. Factors of Safety for Timber.

Much information is available on the character and magnitude of factors which affect the strength and variability of timbers and the likelihood of their failure under various loading conditions. These factors will be discussed for bending, compression, and shear stresses under their respective headings. (Appendix, pars. 17 to 21, inclusive.) This information, however, is not sufficient alone for arriving at factors of safety or safe working stresses. At times the loading assumed is the maximum which can occur and contains the factor of safety, none being used in determining working stresses. At other times the loading assumed is of a magnitude that is likely to occur and is accompanied by a relatively low working stress. In the first instance the occupant of the building must keep within the design load by an unknown percentage in order to be certain that no failure will occur. Under the second assumption he can place the design load on the member with perfect assurance.

In the stresses shown in Table 7 it is assumed that small overloads are likely to remain on the timbers for long periods of time, and that loads two or three times the permitted load are likely to continue only for short periods.

The working stresses for bending and compression given in Table 7 are for timbers which meet the requirements of the select grade of the Central Committee on Lumber Standards. Considering that timbers of the select grade develop 75 per cent of the strength of the clear wood, the average timber has a factor of safety of 2¼ under long-time loading, 4 under 5-minute loads, and 6 under impact loads. One timber in 100, the clear wood of which is more than 36 per cent below the average bending strength for the species, has under long-time loading a factor of safety of about 1½.

The endeavor has been to establish in Table 7 a correct and fair relation between the species and to make the stresses safe under all conditions of loading and service. However, because of the number and nature of variables included in the factor of safety for wood it is difficult to make a correct comparison with the factors of safety for other materials. It is hoped that this discussion of these variables will clarify the methods used in arriving at safe working stresses for wood.

Par. 17. Bending—Stress in Extreme Fiber.

Below are given the factors of safety which exist under various conditions with the working stresses for extreme fiber in bending as shown in Table 7. About 1 time in 100 a timber is found, the clear wood of which is more than 36 per cent below the average bending strength for the species. If this 1 timber in 100, because of defects, develops only 75 per cent of the strength of the clear wood then it would be expected to break at one and one-half times the stress given under a load of approximately 10 years duration.

The average timber has a factor of safety of 2¼ on long-time loading, considering that it develops only 75 per cent of the strength of clear, green wood. Using the same stresses the factor of safety of the average timber under a 5-minute loading is about 4, while for impact it runs up approximately to 6.

Par. 18. Bending—Horizontal Shearing Stresses.

There are very wide variations in the shear stresses developed in structural timbers which fail by horizontal shear. These variations are due largely to the effect of checks and shakes which have a greater influence than their appearance would indicate. This tends to justify the use of a high factor of safety. On the other hand, the timbers with highest quality of clear wood are the most subject to shakes and develop the checks most injurious to strength in shear; after failing in shear, however, these timbers can usually carry their design load with safety because of their high bending strength.

The establishment of factors of safety and safe working stresses for shear is further complicated by an error in the fundamental assumptions used in developing the shear formula. When the formula is applied to checked beams with the load approaching the supports a large error results. In extreme cases the actual stress developed is only about one-third that indicated by the formula. Under other conditions the formula is about right. This error in the formula has led bridge engineers to question the shear stresses in Table 7. However, since the working stresses are about correct for uniform loading, it does not seem advisable to raise them even though they are too low when applied to concentrated moving loads. It seems desirable rather to correct the formula or to make allowances in the assumption as to loading. Before much can de done toward this it will be necessary to make an extensive investigation, for while it is relatively easy to see the error in the shear formula, to correct it is quite complicated.

Par. 19. Bending—Stiffness.

The modulus of elasticity values shown in Table 7 are not safe working stresses. They are approximately the average stiffness of the species when green and are intended for use in computing the average deflection of beams. As defects have little effect on the stiffness the modulus of elasticity values are applicable to all grades.

As timber dries out it becomes somewhat stiffer, and as a consequence the deflection will not be as great as indicated by the use of the values given in the table. In long-time loading timbers will sag somewhat, receiving a permanent set. This set, which may be considered as independent of the deflection, is usually about equal in amount to the deflection except where timbers are overloaded, when the sag becomes much more pronounced.

In cases where the deflection of beams is limited to a definite fraction of the span a simple relation exists between span and depth which determines when stiffness is of importance and when stress in extreme fiber is the important factor in load-carrying capacity. If, as is commonly the case, specifications require that the deflection shall not exceed 1/360 of the span in uniformly loaded beams, and if it is assumed that the ratio of allowable stress in extreme fiber to modulus of elasticity is 1 to 1,100, which for the select grade is about correct for all of the more important species, then when the span in feet is less than one and one-fourth times the depth in inches the stress in extreme fiber will govern the design; when more the stiffness will govern. With the common grade the divison between where stress rules and stiffness rules is at a span in feet one and one-half times the depth in inches.

Par. 20. Compression Parallel to the Grain—Short, Intermediate, and Long Columns.

The factors influencing determination of the so-called factor of safety for compression parallel to the grain are the same as those in bending but differ in their effect on the strength. Knots and cross grain are about one-half as injurious to compressive as to tensile strength and checks and shakes have very little effect on compressive strength. The elastic limit is closer to ultimate strength in compression than in bending and the loss of strength with increased duration of stress is considerably less. The variation in strength of clear wood is about the same for compression as for bending.

Features of design, other than strength, usually call for posts considerably larger than are required to carry the actual loads. It is seldom, therefore, that timbers used as posts are subjected to their maximum stress in permanent construction although it frequently happens in false work and temporary construction. For loads with a duration of stress of about five minutes the factors of safety should be about the same as for bending. For impact loading the factors of safety are somewhat less and for long-time loading somewhat more.

The factor of safety of 3 used with the Euler formula is necessary to care for the effect of crooks and defects and for overloading. It has been maintained that since the fiber stress at elastic limit is never passed in going to the maximum load of long columns the factor of safety could be about one-half that for short columns. The modulus of elasticity of timber is its most variable property and the one most difficult to estimate visually. Tests of clear wood and the laws of probability indicate that 1 timber in 100 will have less than 56 per cent of the average stiffness. It is believed, therefore, that a factor of safety of 3 should be used for long columns.

The effect of eccentric loading on the strength of columns of long, short, and intermediate lengths is about the same. The reduction in the maximum load which a column will carry due to an eccentric loading is not as great as might be expected.

The importance of crooks and eccentric loading of columns has generally been overestimated. Timber when subjected to combined bending and compression, whether the bending is from eccentricity or otherwise, develops a higher stress at both elastic limit and maximum load than when subjected to compression only.[12] For example, in loading a short block between plates supported and loaded by knife-edges, the load obtained when the line of action of the force was such as to give zero stress on one side was about three-fourths the load obtained with a true axial loading and not one-half, as would be the case if the stress under the two conditions was identical. This example is not intended to convey the idea that unrestricted crooks and eccentricities are permissible or that higher stresses are permissible in eccentrically-loaded columns, but is for the purpose of relieving anxiety as to influence of imperfect end conditions and the influence of crooks such as are common in most columns.

No definite recommendation can be made from the data available as to the influence of defects on column strength. However, sufficient information has been obtained to show that the injurious effect of defects on long columns is much less than is generally assumed. Defects, such as are permitted in the Central Committee on Lumber Standards grade of select posts and square timbers, have little influence upon the strength of long columns.

Par. 21. Compression Perpendicular to Grain.

The compression stresses perpendicular to grain in Table 7 are based on elastic limit rather than ultimate strength. This is because the nature of the materials is such that it is practically impossible to obtain a maximum either in test or in service even though the whole surface be covered.

Usually compressive loads perpendicular to grain are applied over a relatively small portion of the surface and the stresses are consequently very much distributed before they reach any considerable depth in the beam where most of the shakes and checks occur. The compressive strength perpendicular to grain of wood in and around knots is greater than that of clear wood. For these reasons the effect of defects on this property is not important and the same stresses are given for all grades.

If the moisture content is high, near 20 per cent, about 1 timber in 100 when loaded up to the stress specified for dry locations would be expected to mash under the bearing to an extent that in time would necessitate replacement. The average timber, however, would carry the permitted stresses for an indefinite time without mashing. It is

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[12] Tests showing this are presented in Report No. 188 of the National Advisory Committee for Aeronautics, "The Influence of the Form of a Wooden Beam on Its Stiffness and Strength," by J. A. Newlin and G. W. Trayer.

seldom that crushing across the grain endangers the safety of a structure.

Par. 22. Composite Columns.

A type of composite column that has come into general use in several sections of the country, ordinarily referred to as the "Lally column," consists of a steel pipe filled with concrete. This column, however, is entitled to recognition as a safe structural element in building construction only when extreme care is taken in its manufacture to secure by proper shop work a complete filling of the pipe with a dense 1 : 1½ : 3 gravel or crushed rock concrete. Only new standard heavy-weight mild-steel pipe should be used. Unless such care has been exercised the column has no greater strength value than an unfilled steel tube used as a column. In multi-story construction special connections must be designed and used. This column offers considerable fire resistance without exterior protection, but when encased in fireproofing materials it can be given a correspondingly greater fire resistance classification. The committee recommends for the properly designed and fabricated column of this type safe working loads in accordance with the formulas for composite columns in the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

Par. 23. Welded Connections.

The welding, either electrically or by the use of gas, of all kinds of connections is being advocated by some engineers engaged in steel construction. The advantages claimed for this method over the usual riveting process are economy in cost of erection, lighter weight in the connections, less noise during construction, greater rigidity, and increased strength. Although tests have been made and welded connections have been used in some actual structures, the experience and information thus far available are too limited to justify recommendations. An investigation of this subject is contemplated by one of the national technical societies, the outcome of which should be awaited before conclusions are drawn.

Par. 24. Construction Resistive to Earthquakes.

Suggestions have been received from people living in localities subject to earthquakes that the committee should recommend requirements for construction to resist such shocks. Efficient and rational design of structures to resist earthquake shocks must be based upon reasonably accurate assumptions as to the nature and magnitude of the forces which are to be resisted. At best such assumptions for any particular locality are based upon a guess as to whether past experience may be repeated or how much greater severity of shock may sometime occur. Knowledge of the forces produced by certain extents and periods of motion is accumulating, but no means has been devised for predicting the extent of motion or the periodicity of future quakes.

In localities where earth shocks occur frequently buildings should be designed to resist them. The necessity for such provisions is not so apparent in other parts of the country where the intervals between serious shocks, as judged by past records, indicate periods frequently several times greater than the ordinary life of a building. However, accepting the judgment of seismologists that earthquakes are likely to occur from time to time on the East Coast as well as on the West Coast, it is doubtless the part of wisdom to make provision in the design of at least all large or monumental buildings to resist such forces. The subject, however, is related more closely to loads, foundations, and details of connections and bracing than to working stresses, and therefore does not logically belong in this report. For this, and other reasons previously mentioned, this committee refrains from making recommendations for such design.

Committees of architects, engineers, and earthquake experts were organized in California subsequent to the Santa Barbara earthquake to study the problem of building construction in its relation to earth shocks. They included men of recognized authority upon the subject, and they had for study the records of all important earth shocks of recent years in this and other countries. The information derived from the investigations of these committees furnished the data for the principles of design to resist earthquake shock which are the basis of special building ordinances adopted in the cities of Santa Barbara and Palo Alto. These ordinances, while agreeing in conception as to what should be accomplished, are not entirely in harmony as to methods of calculation and design. They are recommended to those interested in such regulations as the best available treatment of the subject to date.

DEPARTMENT OF COMMERCE
WASHINGTON

PUBLICATIONS IN RELATION TO HOUSING AND MUNICIPAL REGULATION

[These publications may be obtained from the Superintendent of Documents, Government Printing Office, Washington, D. C., payments to be made by money order or New York draft; currency at sender's risk. Postage stamps or foreign money not accepted.]

BH1. Recommended Minimum Requirements for Small Dwelling Construction.

By the Building Code Committee: Ira H. Woolson, chairman; Edwin H. Brown, William K. Hatt, Rudolph P. Miller, J. A. Newlin, Ernest J. Russell, and Joseph R. Worcester. 30 illustrations. 108 pages. Government Printing Office, Washington. Price, 15 cents.

BH2. Recommended Minimum Requirements for Plumbing in Dwellings and Similar Buildings.

By the Subcommittee on Plumbing: George C. Whipple, chairman; Harry Y. Carson, William C. Groeniger, Thomas F. Hanley, and A. E. Hansen. 100 illustrations. 250 pages. Government Printing Office, Washington. Price, 35 cents.

BH3. A Zoning Primer.

By the Advisory Committee on Zoning: Edward M. Bassett, Irving B. Hiett, John Ihlder, Morris Knowles, Nelson P. Lewis, J. Horace McFarland, Frederick Law Olmsted, and Lawrence Veiller. 12 pages. Government Printing Office, Washington. Price, 5 cents.

BH4. How to Own Your Home.

By John M. Gries and James S. Taylor, with a foreword by Herbert Hoover. viii plus 28 pages. Government Printing Office, Washington. Price, 5 cents.

BH5. A Standard State Zoning Enabling Act.

By the Advisory Committee on Zoning. 12 pages. Government Printing Office, Washington. Price, 5 cents.

BH6. Recommended Minimum Requirements for Masonry Wall Construction.

By the Building Code Committee: Ira H. Woolson, chairman; Edwin H. Brown, William K. Hatt, Albert Kahn, Rudolph P. Miller, J. A. Newlin, and Joseph R. Worcester. 57 pages. Government Printing Office, Washington. Price, 15 cents.

BH7. Minimum Live Loads Allowable for Use in Design of Buildings.

By the Building Code Committee. 38 pages. Government Printing Office, Washington. Price, 10 cents.

BH8. Recommended Practice for Arrangement of Building Codes.

By the Building Code Committee. 29 pages. Government Printing Office, Washington. Price, 10 cents.

Seasonal Operation in the Construction Industries.

Summary of report and recommendations of a committee of the President's Conference on Unemployment, with a foreword by Herbert Hoover. viii plus 24 pages. Government Printing Office, Washington. Price, 5 cents. (Department of Commerce Publication.)