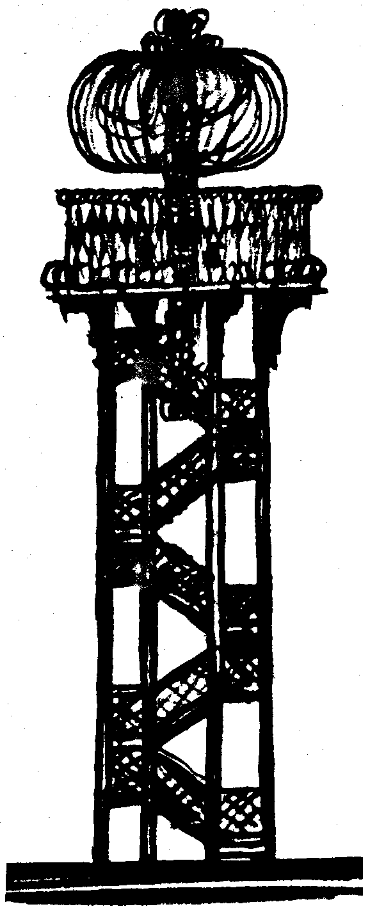
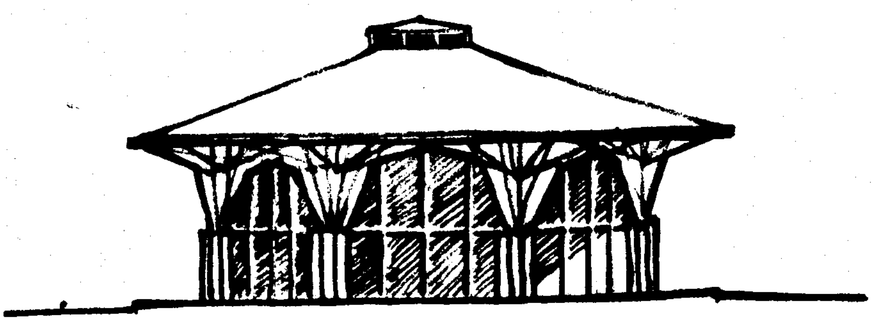
Pole Building Design



By Donald Patterson
Structural Engineer

B.E., State University of Iowa, 1922; C.E., 1930. Member American Society of Civil Engineers, American Society of Testing Materials, International Association for Bridge and Structural Engineering, American Railway Engineering Association. Donald Patterson has designed, constructed, and inspected all types of structures, from coast to coast, in which pressure treated round and sawn timbers are used. Because of his broad, diversified experience he is well qualified to have prepared this concise, dependable working manual. It is written for engineers by an engineer. It gives the correct design procedures for proportioning structural members of pole-type buildings of all sizes, kinds, and uses.



Sixth Edition

AMERICAN WOOD PRESERVERS INSTITUTE
2600 Virginia Ave., N.W., Washington, D.C. 20037

Reprinted, April 1970
Reprinted, August 1970

Vacation Village, a thirty-acre island resort in Mission Bay, near San Diego, is built on filled land, dredged up from the Bay. Pole-type construction serves not only as the frame work for the various buildings but also as the architectural accent.

PREFACE

PROPERLY designed, modern pole-type buildings owe their popularity primarily to their low cost, the ease and speed with which they can be erected. Recent surveys reveal that about 600,000 or more pole-type buildings now are in service in the United States and Canada. Their adaptation to a wide range of functional structures for industrial and commercial uses stems from more than 35 years of successful use in the farm field, where the several inherent advantages of this type construction have been demonstrated.

Construction experience has shown that building costs may be reduced by 25 to 50 percent or more by using pole-type design. Contract prices for completed buildings have ranged from 90¢ to $1.40 a sq. ft. for farm buildings. Industrial and commercial buildings have been erected for as little as $1.90 to $4.00 depending on location, labor costs and the type of finished building.

Superstructures of pole-type buildings are relatively light. Floor loads are supported independently by the ground and not by the frame of the buildings; hence elaborate, expensive masonry foundations are not required. Bases for the poles or columns, set to moderate depths in average soils, support these buildings safely even in regions where design provisions must be made for strong winds of hurricane force or for heavy snowfall.

Erection also has been simplified. The necessity of cutting or framing of structural members so common in conventional construction in many designs, has almost been eliminated. The simple lapping of commercial lengths of lumber obviates the necessity for any but the simplest cuts in members for roofs and walls.

Designers find that pole buildings may be modified or expanded with ease. They can be built to almost any desired dimensions or proportions. Buildings now in use range up to three acres of floor area. Warehouses and bulk storage buildings are designed for the use of all types of automotive trucks and other types of materials handling equipment. Poles are widely spaced to allow for easy movements of palletized loads and larger machines. Wide clear spans are provided by the use of simple, light-weight wood trusses.

Poles and lumber, even in ground contact, when properly pressure preserved in accordance with standard specifications, will last for 40 to 50 years and more even in areas of very severe exposure.

Simplicity of construction lends itself to many adaptions by designing architects and engineers. Furthermore, well designed and well built structures of this type have proved their resistance to many severe storms and hurricanes. No longer are they classified as temporary low-cost expedients because good design has proved their worth through many years of service.

ELEMENTS OF DESIGN

INTRODUCTION

Although the design of pole-type structures is basically very simple, civil or structural engineers should be employed in the design of most structures. The safety and satisfactory performance of these buildings, as with other types, depend on the proper and informed evaluation of loads, stresses, deflections, foundation capabilities and their relative influence and importance in any given case. Since the stability of the structure depends upon the integrity and quality of the foundation material, it is also recommended that an adequate foundation investigation be made retaining the services of a competent foundation engineer, if necessary.

These "Elements of Design" are intended to serve as a guide for engineers familiar with building design. The engineering concepts that are somewhat unusual or unique in pole-type buildings are stressed.

SUMMARY

The steps required in designing a pole-type building may be briefly summarized as follows: general features, such as overall length and width, spacing of poles in transverse bents, spacing of bents longitudinally in the building, height at the eaves, pitch and type of roof, and the kind of flooring to be used, as well as any special features such as wide bays. Unsymmetrical layouts or the possible suspension of particular loads from the roof framing, must be determined from the occupancy and use to which the building is to be put.

Having determined these characteristics of the structure, external loads to be applied must be considered. These will be obtained from local codes wherever available, but they may, in some cases, have to be matters of judgment on the part of the designer. In the case of wind loads, resort may be had to the map on page 35 for horizontal pressures. Character of the soil must be investigated, preferably by some positive exploration, such as the soil auger mentioned in connection with the Rutledge chart; Figure 1, page 6.

The roof and its framing are then designed by conventional methods, proceeding from minor members to the larger and more basic supports, coming lastly to the poles themselves. These must be analyzed, first for the required depth of embedment, then bending and direct loading. Both the outside poles, which are generally governed by bending, and the interior ones in which the bending forces are diminishing and direct stresses reach their maximum, should be investigated. Since results obtained may require depths of embedment or pole sizes other than those originally assumed, it may be necessary to run through a second set of calculations until the various results obtained are consistent throughout.

Not much work is involved in the design of even a large building of this type. A thorough analysis will be repaid by a structure economical throughout and with a large reserve of strength in some of its features, such as resistance to abnormal wind pressures.

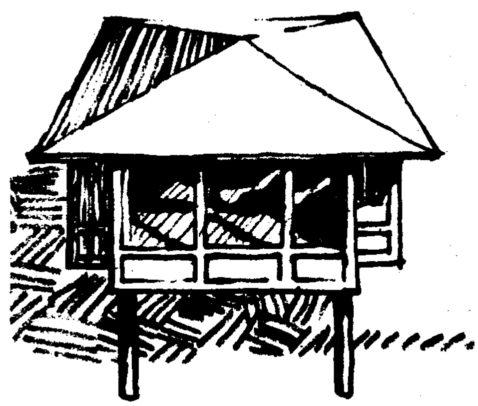
Specifications for Poles and Lumber

Pole grading and strength are given by the United States of America Standards Institute in its publication, "Specifications and Dimensions for Wood Poles." Table VI, page 45 in the Appendix is taken from that publication. It gives the dimensions for poles of Douglas fir and Southern pine. This USA bulletin also gives specifications and dimensions for all of the common varieties of timber used for poles, together with the general material, manufacturing and handling requirements. The USA stress values are adopted also by the National Electrical Safety Code.

Specifications and working stresses for sawed material are published by the lumber industry. The designer should refer to those applying to the species of lumber he intends using. Illustrative examples that follow have assumed the use of Douglas fir or Southern pine lumber and poles of these same species. Design analysis will give the required fiber stress which, in turn, will indicate the grade of lumber to be used. In buildings where the loads are comparatively light or the spans rather short, lesser grades more readily obtainable in small lumber yards may be used. In structures where loads are heavy, spans are relatively long, or where special framing arrangements or trusses are required, higher stress grades will be required.

Preservative Treatment

Poles and lumber that have been pressure treated in a closed cylinder to the recommended net retentions of preservative per cu. ft. of wood by a standard pressure process should be specified for columns and for frame lumber in contact with or close to the ground.



Selecting the proper method of treatment is just as important as selecting an effective preservative. The best preservative known will not prevent decay or insect damage to wood that is in contact with the soil or close to it if it has not been applied properly. The process used for injecting preservative into the wood must be one that, not only secures adequate depths of penetration to afford real protection, but also insures uniform diffusion of preservative through the treated area to avoid spotty treatments or thinly protected places where decay or insects may gain entrance to the untreated interior of the timber.

Wood preservatives fall into two broad groups, viz: (a), preservative oils or oil-borne chemicals, and, (b), water-borne salts. The recognized standard preservatives in the oil group are creosote, creosote-coal tar solution, creosote-petroleum solutions and pentachlorophenol.

Approved preservatives and recommended retentions of preservatives in pounds per cubic foot of wood are listed in the Recommended Treatments Section, pages 36 to 38. Poles and lumber to be used in direct contact with the ground are generally treated with creosote, pentachlorophenol or one of the three following water-borne salt preservatives: Ammoniacal Copper Arsenite (ACA), Chromated Copper Arsenate—Type A (CCA), or Chromated Copper Arsenate—Type B (CCA). All of the standard water-borne preservatives listed in Table 1, page 37 are suitable for use above the ground as well as creosote and pentachlorophenol preservatives.

Standard salts or pentachlorophenol in a light or volatile petroleum solvent are better adapted to lumber and building materials where painting is desired, or where odor must be eliminated—for example, in tightly closed compartments where sensitive foods are stored or processed. Wood treated with these preservatives should be allowed to dry prior to painting.

Items for which pressure treated materials should be used are poles or columns that are set in the earth, or lumber where construction details will permit a moisture content in the wood above twenty percent. These may include nailing girts, skirting, facia, balconies and porches exposed to the weather.

American Wood-Preservers' Association standards for the preservative treatment of lumber, plywood, poles and fence posts are designated in Table 1, along with the appropriate retentions for suitable preservatives. The AWPA standards for the preservative and the corresponding Federal standards are also included.

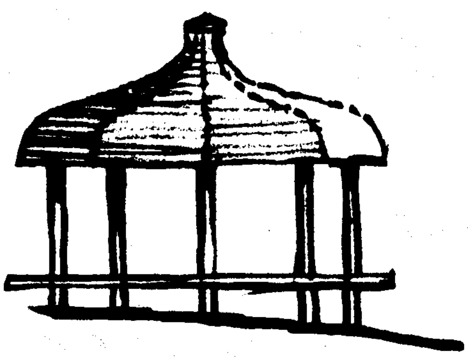
New Quality Control Standards have been developed by the American Wood Preservers Institute to assure users of properly pressure treated lumber and plywood. These Standards stipulate types, quantities and penetrations of preservatives needed to protect wood against termites and decay. Interested buyers need only specify items to be treated and the appropriate AWPI Standard (see pages 36 and 37). The AWPI Quality Mark on each piece of pressure treated lumber and plywood is evidence of treatment in accordance with these Standards.

Based on the actual service records of wood poles used in utility lines, a service record of at least fifty years can be expected for building poles treated with the recommended creosote, pentachlorophenol and salts preservatives in compliance with good and adequate specifications. Pole-type buildings have been approved where light frame structures are restricted because of fire hazard. Wood framing members are so widely separated that fire is unlikely to spread from member to member. In case, however, of high-hazard occupancies lumber can be pressure treated with fire retardant chemicals. Standard fire-retardant treatments and chemicals are approved by the American Wood-Preservers' Association and can be specified as shown in Table I.

Wind Pressure

There are two ways of selecting the wind pressure to be used in designing one of these structures.

If the structure is to be built in a city, requirements of the local building code must be followed. These usually will stipulate how the specified wind loads are to be applied and the pressure to be used. Conditions from which the code was derived will undoubtedly prevail over the surrounding area to a considerable distance.



The Outdoor Advertising Association of America, Inc., published in 1945 a map of the United States showing the recommended minimum design wind pressures, based upon data obtained from the United States Weather Bureau and American Standard Building Requirement A 58.1—1945. This map, page 35, shows general areas where design wind pressures of from 15 to 30 lb per sq ft are recommended as minimal. The highest values generally are confined to the Great Plains area and to the Gulf and Atlantic coasts. The map is in all probability the most reliable information on wind pressures for the country as a whole.

In the examples given later, maximum wind pressure has been assumed to be 20 psf, which is a common code requirement for buildings under 50 ft tall.

Because, in general, the principal load imposed on the frame will be the horizontal wind load, the designer should make every effort to assume a safe figure and, if possible, that of a nearby city building code. In the absence of such a code within a reasonable distance, the wind map on page 35, may be used.

Bearing Capacity of Soils

Estimating the bearing capacity of a soil or passive earth pressure is more difficult that determining applicable wind pressure. Characteristics of a certain soil may be determined, at the time of an investigation, with some degree of accuracy, but these characteristics may be altered by a later change in moisture content.

Professor P. C. Rutledge, following tests he conducted at Purdue University for the Outdoor Advertising Association of America, Inc., devised the chart, Figure I, for determining adequate depths of set for cantilever poles subject to lateral forces. This chart permits soil classification of a general nature to be made from a simple test on the site. This test consists of determining the force required to withdraw an auger from various depths. Soils are divided into five classifications, viz: very soft, poor, average, good and very hard. Values for these gradations range in lb per sq ft from 800 to 1200; to 2000; to 3050; to 4100 and to 4500 or above respectively.

This chart bases allowable unit soil pressure on the pull, in pounds, on an indicator auger of 1½-in diameter. In the chart, this pull is calibrated for two different types of soil; for sandy or gravelly soils, and for silts and clays. The first type permits higher allowable unit values than the second for the same pull on the auger. Reliability of the chart and its method of soil evaluation has been demonstrated by broad experience.

If the method of Figure 1, or some similar method, for determining strength of the soil is not available, it may be necessary to resort to visual inspection and to estimate value from samples taken from a shallow pit or from materials removed by a post hole digger or auger. The following descriptions may be helpful in such cases:

The United States Steel handbook, "Steel Sheet Piling," lists 27 soils, with their characteristics, based on the Coulomb-Rankine theories. Approximate allowable average unit pressure S_1 for these soils to be used in Figure 1 can be estimated when allowable unit pressures are plotted and compared with values in the chart. Allowance must be made, however, for the ultra-conservative passive pressures given by the Coulomb theory. In Table III, Appendix, page 43, approximate positions, lower third, middle third or upper third, of the five classes of soils in the chart, Figure 1, are for soils listed in the United States Steel handbook. Classifying the soil from a visual inspection and entering the chart from the approximate position indicated in Table III should furnish fairly reliable results for depth of embedment.

Soil Classification After Visual Inspection

The Outdoor Advertising Association of America, Inc., gives a general classification of soils for use when only a visual inspection is made, with the warning that the worst condition of moisture content should be anticipated. Good soils are described in the OAAA Engineering Design Manual, page 3, as, "Compact, well-graded sand and gravel; hard clay; well-graded fine and coarse sand; decomposed granite rock and soil. Good soil should be well-drained and in locations where water will not stand." For average soils, the description is, "Compact, fine sand; medium clay; compact, well-drained sandy loam; loose, coarse sand and gravel and medium clay. Average soils should drain sufficiently well so that water does not stand on the surface." Poor soils are, "Soft clay, clay loam, poorly compacted sand, clays containing a large amount of silt and vegetable matter. These soils will hold and absorb great quantities of moisture when wet. Usually, soils of this type are found in low-lying areas where water stands during the wet season."

Fitting a particular soil into the chart, Figure 1, requires considerable judgment. However, in conjunction with Table III this classification provides a general guide to the soils at a particular site.

Capacity of weak soils often can be improved by backfilling with soil cement or other suitable material. Wales or baffles that spread compression over greater soil areas in the upper two-thirds of the embedded depth may be used advantageously in many cases.

Attention should be called to the fact that, in general, soil in the upper two-thirds of a hole is of primary importance in judging allowable values, because unit pressure above the point of rotation is the governing factor in depth of embedment. When the auger-pull method of testing is used, results should indicate a soil's value at the estimated position of the bottom of the pole.

In the examples given later, because average soil is assumed, an allowable soil stress of 2,500 psf has been selected.

With the two basic assumptions of passive earth resistance and wind pressure made, vertical loads to be designed for also must be determined.

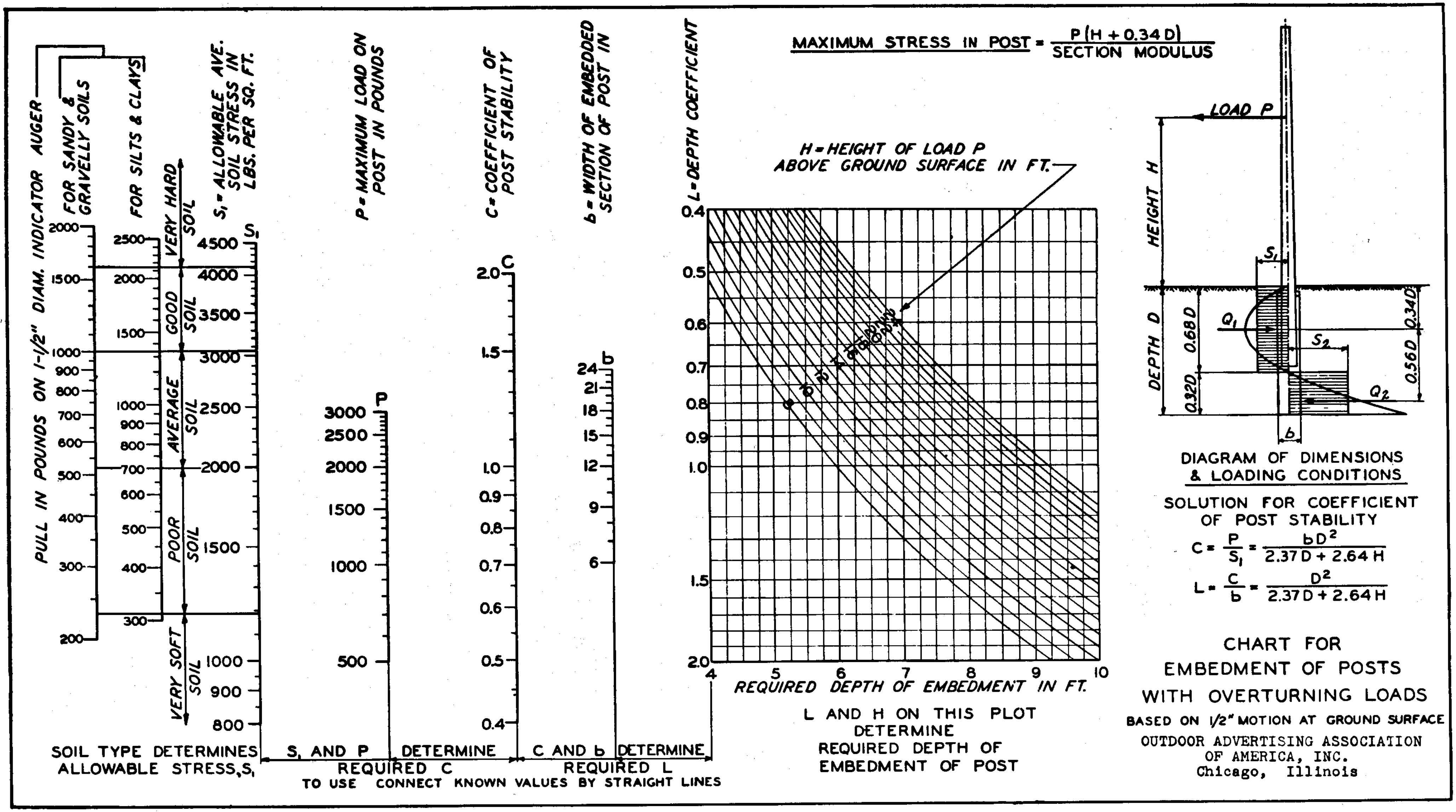
Dead load consists of the weight of all component parts of a building supported by any one pole. It is easily figured, once the spacing of poles is decided. Usually dead load in this type of structure will be small in comparison with other loads on the pole framework.

Some live loads, wind force and snow, for example, vary in different parts of the country. Here again the safest course is to follow values shown in building codes of cities in the area. In southern states, where snow is infrequent and light, snow load may be small, and often is ignored. In northern localities, where damp snow may pile up to a considerable depth on comparatively flat roofs, the live load will be relatively high.

In addition to snow load on roofs, the live load also may include monorails or other load-carrying devices in buildings designed for special purposes.

Inclined roofs will transmit vertical components of wind loads to pole frames and these must be added to dead and live loads. Building codes in general permit higher allowable unit stresses to be employed in designing for the combination of dead plus live plus wind loads. This increase ranges from one and one-quarter to one and one-half, with most codes

Chart for Embedment of Posts



SOLUTION FOR COEFFICIENT OF POST STABILITY

$$C = \frac{P}{S_1} = \frac{bD^2}{2.37D + 2.64H}$$

$$L = \frac{C}{b} = \frac{D^2}{2.37D + 2.64H}$$

CHART FOR EMBEDMENT OF POSTS WITH OVERTURNING LOADS

BASED ON 1/2" MOTION AT GROUND SURFACE

OUTDOOR ADVERTISING ASSOCIATION OF AMERICA, INC.
Chicago, Illinois

Figure 1

permitting the allowable unit stresses to be increased by one-third for such a combination of loads, provided, however, that basic unit stresses are not exceeded for dead load plus live load.

In design examples that follow, live load has been assumed to be 20 psf and the basic allowable unit stresses for dead load plus live load have been increased by one-third when wind loads have been added to the other two.

Embedment

Embedment of piles and poles, to develop lateral stability through passive earth pressure, has been considered for a long time. Early attempts to solve this problem generally were made in connection with the design of dock walls, bulkheads, and walls of cofferdams. These usually were based on methods of Rankine which, in turn, rested on Coulomb's theory of earth pressures. In most cases there were other horizontal supporting elements in addition to passive earth resistance against the embedded portion of a wall, such as anchored tie rods or cofferdam bracing. Considerations other than lateral stability often were major factors in determining depth of embedment.

It has long been realized that analyses based on Coulomb's formulae for earth pressures are too conservative. Field tests and laboratory experiments during the last 25 or 30 years have furnished data on the pressures exerted by, and the stresses induced in, laterally loaded poles in certain types of soil. Methods of analysis have been developed by different investigators in attempts to reconcile theory with results obtained from various controlled field or laboratory studies. In practice, the design of structures utilizing laterally loaded vertical elements embedded in the ground must be governed by assumptions, based on experiment, experience and service records of many similar structures.

Attaining lateral stability with minimum depth of embedment of poles is of primary importance in the design and construction of pole-type buildings. A method for determining required depth of embedment necessary for lateral stability of poles loaded horizontally will be given here. It has been used extensively and has a wide background of successful applications, checked by numerous full-scale and laboratory experiments.

Depth to Point of Rotation

In the specific problem of a single pole embedded in the ground and subjected to a lateral pull or thrust at some height above the ground, we are principally interested in depth of embedment, maximum bending moment, and the point where it occurs in the pole.

For many years the problem of pole embedment was solved by rule of thumb, using a fixed ratio of embedded depth to overall length of pole.

The depth of set required to prevent rotation of a cantilever pole acted on by a lateral force can be determined quite easily from the chart, Figure 1, when allowable soil stress, size of the pole, and height at which the lateral force acts are known.

The principal factor involved in determining proper depth of embedment is distance below surface of the ground to the point of rotation of the pole. This is the point where passive earth pressure changes direction, from one side of the embedded member to the other.

In poles with shallow depths of set, this point generally was found to be approximately two-thirds the embedment depth below surface of the ground. Variations were so slight as to be considered negligible because of the much greater uncertainty in soil pressure and other factors. In some cases percentage of depth to point of rotation was modified to be proportional to vertical area above that point divided by total vertical area. In poles, where taper is very slight, and the effect of other variables is taken into account, such a modification obviously is unwarranted.

In some work on poles, external moment above the ground surface has been taken as the maximum, ignoring the fact that the moment curve continues to increase for some distance below the surface. Some proportion of depth of embedment undoubtedly should be added to height above ground in computing maximum bending moment in the pole. If the point of rotation is at two-thirds the depth of embedment, an assumed added length of approximately one-quarter of the embedded depth for computing maximum bending moment in the pole would seem reasonable. This assumption has been made in examples that follow.

Embedment Chart

The nomographic embedment chart of the OAAA, Inc., was developed by Prof. P. C. Rutledge from tests that he made at Purdue and Northwestern Universities. Later tests, made for the same organization by Professors Walter L. Shilts, Leroy D. Graves, and George F. Driscoll at Notre Dame University, and by Dr. J. O. Osterberg at Northwestern University bore out reliability of the chart.

Different investigators used slightly different percentages of depth of embedment for the moment arms of earth pressure reactions, above and below the point of rotation. All used a depth to the point of rotation of approximately two-thirds the embedded depth. Such small differences in percentages have very little practical effect on parameters L and C, of the chart in Figure 1.

This chart, Figure 1, assumes a distance of nine-tenths the depth of embedment from the ground to the resultant of the lower earth pressure force. Using a depth of two-thirds the embedment from the ground line to the point of rotation, and assuming the lower soil pressure to have the shape of a semi-parabola curving away from that of the upper soil pressure, the distance given above would be seven-eighths instead of nine-tenths the embedded depth. Such a change would alter values obtained from the chart by about four percent at the most.

By blunting the lower portion of the parabola of earth pressure, as in Figure 2, page 9, the laws of statistics are complied with in the figure, and statements of various investigators that soil pressures below the point of rotation are, in general, unimportant, is brought out graphically. With the zero point at two-thirds the depth of embedment, passive resistances, as shown by areas between the parabola and axis of the pole, are equal. The center of gravity of the lower area is three-eighths of F above the bottom of the pole, or one-eighth the depth of embedment. This would make E equal to $5/24$, or 0.208 of D. As mentioned before, this would have little effect on values of the chart's parameters.

All other investigations made on poles with shallow embedments have tended to verify these assumptions. Small variations in the several factors used by different investigators, produce very slight changes in the equation or chart, which leads to the result sought, depth of embedment required to prevent objectionable deflection of the pole axis from its original position.

Equation for Depth of Embedment

Referring to Figure 2, and using the notation given there, the general equation for depth of embedment is derived as follows:—

P	is the horizontal thrust in	pounds
H	is the height above ground line of the horizontal thrust in	feet
B	is average diameter of embedded portion of the pole in	feet
A	is depth to point of rotation in	feet
Q_1	is the resultant of soil pressure above point of rotation in	pounds
S_1	is average soil pressure above point of rotation in	pounds per square foot

$$S_1 = Q_1/AB \qquad (1)$$

Q_2	is the resultant of soil pressure below point of rotation in	pounds
S_2	is the average soil pressure below point of rotation in	pounds per square foot

$$S_2 = Q_2/FB \qquad (2)$$

p	is passive earth pressure in	pounds per square foot
D	is depth of embedment in	feet

Embedment Diagram

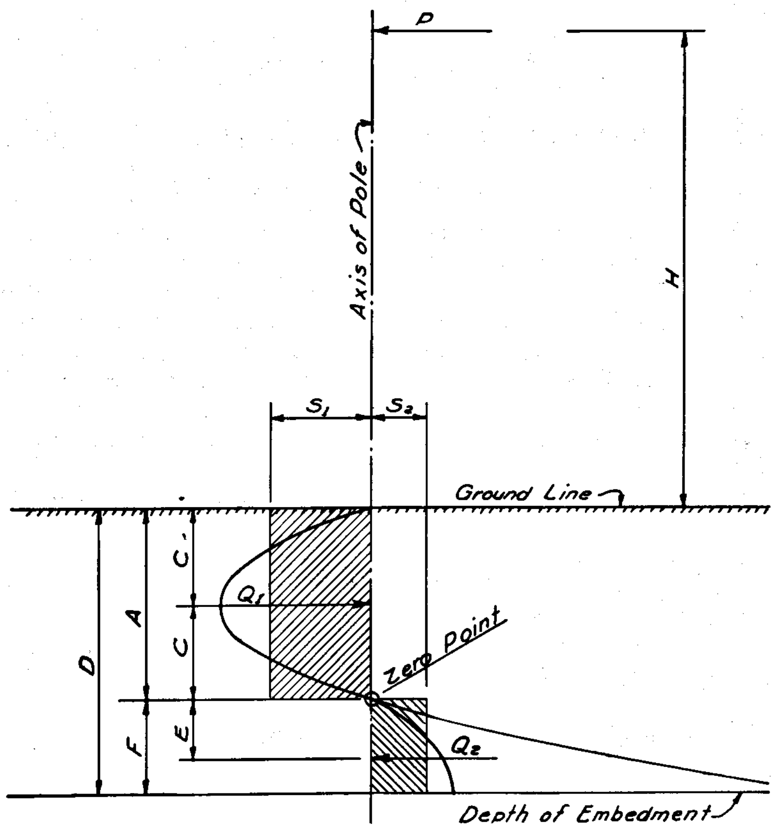


Figure 2

From the notation:

$$S_1 = \frac{Q_1}{AB} \qquad (1)$$

$$Q_1 = S_1 AB \qquad (3)$$

Taking moments about Q_1: $P\,(H + C) = Q_2\,(E + C)$ (4)

$$Q_2 = \frac{P\,(H + C)}{(E + C)} \qquad (5)$$

$\Sigma H = O$: $Q_1 = P + Q_2$ (6)

Substituting in (3),

$$S_1\,AB = P + \frac{P\,(H + C)}{(E + C)} \qquad (7)$$

$$S_1\,AB = \frac{P\,(2C + H + E)}{(E + C)} \qquad (8)$$

$$\frac{P}{S_1} = \frac{AB\,(E + C)}{(A + H + E)} \qquad (9)$$

Assuming that, $A = 0.68\ D$

$C = 0.34\ D$

$E = 0.22\ D$ (all in accordance with Rutledge and corroborated by Shilts, Graves and Driscoll)

Then:-

$$\frac{P}{S_1} = \frac{0.68\ DB\ (0.22\ D + 0.34\ D)}{(0.68\ D + H + 0.22\ D)} \qquad (10)$$

$$\frac{P}{S_1} = \frac{0.381\ D^2 B}{(0.90\ D + H)} \qquad (11)$$

$$\frac{P}{S_1} = \frac{D^2\ B}{2.37\ D + 2.64\ H} \quad \text{(See Figure 2)} \qquad (12)$$

Solving for D in the quadratic equation,

$$S_1 BD^2 - 2.37\ PD - 2.64\ PH = 0 \tag{13}$$

$$D = \frac{2.37\ P + \sqrt{(2.37\ P)^2 + 4 \times 2.64\ PHS_1B}}{2\ S_1\ B} \tag{14}$$

This equation is solved graphically for D by use of the nomographic chart in Figure 1, with insertion of the parameter $C = \frac{P}{S_1}$ and $L = \frac{C}{B}$

Given H, P, B and S_1, depth of embedment may be readily obtained from the chart or from equation (14). Height, H is obtained from the building plan. Thrust, P is computed from assumed wind pressure applied to the particular building. Width, B depends on diameter of the pole selected to resist bending moment induced by the wind force. Allowable average soil pressure, S_1 may be derived from any adequate soil test available at the site. Lacking the means of making any other test, a pit may be dug for a visual inspection and appraisal of the soil, since the depths involved will never be very large.

Factors involved in use of the chart, Figure 1, or in solving by formula, are discussed elsewhere in connection with the design, pages 22-23.

Design criteria for a post embedded in the earth and subjected to lateral thrust have been approved by the International Conference of Building Officials, and are included in the Uniform Building Code. The formula for determining depth of set and explanatory details of the recommendations appear in the appendix, page 42 to 43.

TYPICAL DESIGNS

There are three general patterns of structures for which pole-type framing is particularly well suited. For each pattern there are many variations, in number and spacing of bays, spacing of poles in the bays, pitch and height of roof. A typical structure of each pattern will be illustrated and general features common to all variations of the particular pattern will be set forth.

The three types are shown in outline in Figue 3, below. Type A has an even number of equal panels in each bent, with a pole on the center line. A driveway through the building can be placed on either or both sides of the center line and the bent can be restricted to four panels or extended to eight or ten panels, depending on requirements. Some buildings of this type have been built with three or five panels, with a driveway in the center panel, and either different heights at the eaves, or different roof pitches on the two sides, as indicated by dotted lines in the figure.

Typical Frame Layout

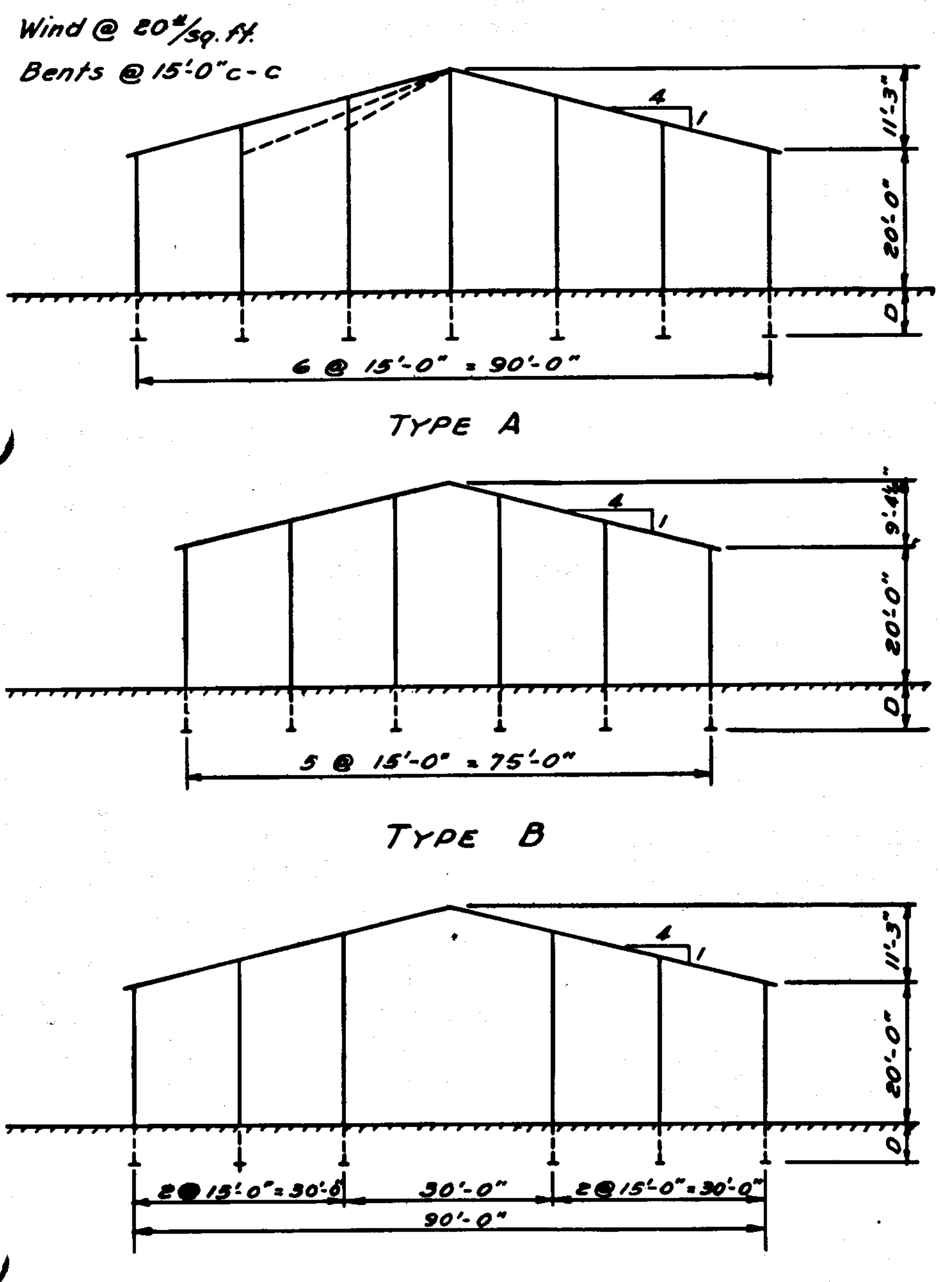


Figure 3

Type B, has an odd number of equal panels in each bent, providing a driveway in the center panel. Here again the bent may be reduced to three panels or increased to seven, nine, or more. With three panels and low eave heights, the design makes an ideal layout for stock barns, either on farms or at fairgrounds. Hundreds have been built for this purpose, as well as for warehouses with more than three panels per bent.

A great many pole-type buildings have been erected with a wide, clear opening in the central panel and several smaller equal panels on each side, as shown in Type C. The central panel may be twice the width of the side panels, or more, or less, but it usually is wide enough to require a truss over it. Many modifications of this arrangement are possible.

Features Applicable to all Designs

A few general considerations apply to all pole-type buildings. Spacing of poles in bents, and longitudinal spacing of bents in buildings, if made to use commercial lengths of lumber without waste, will effect marked economies in their overall cost. Other factors that govern spacings may force this economy to be sacrificed, but where it can be done, savings in cost make it worth while to work out economical distances between poles and between bents.

Roof Panel

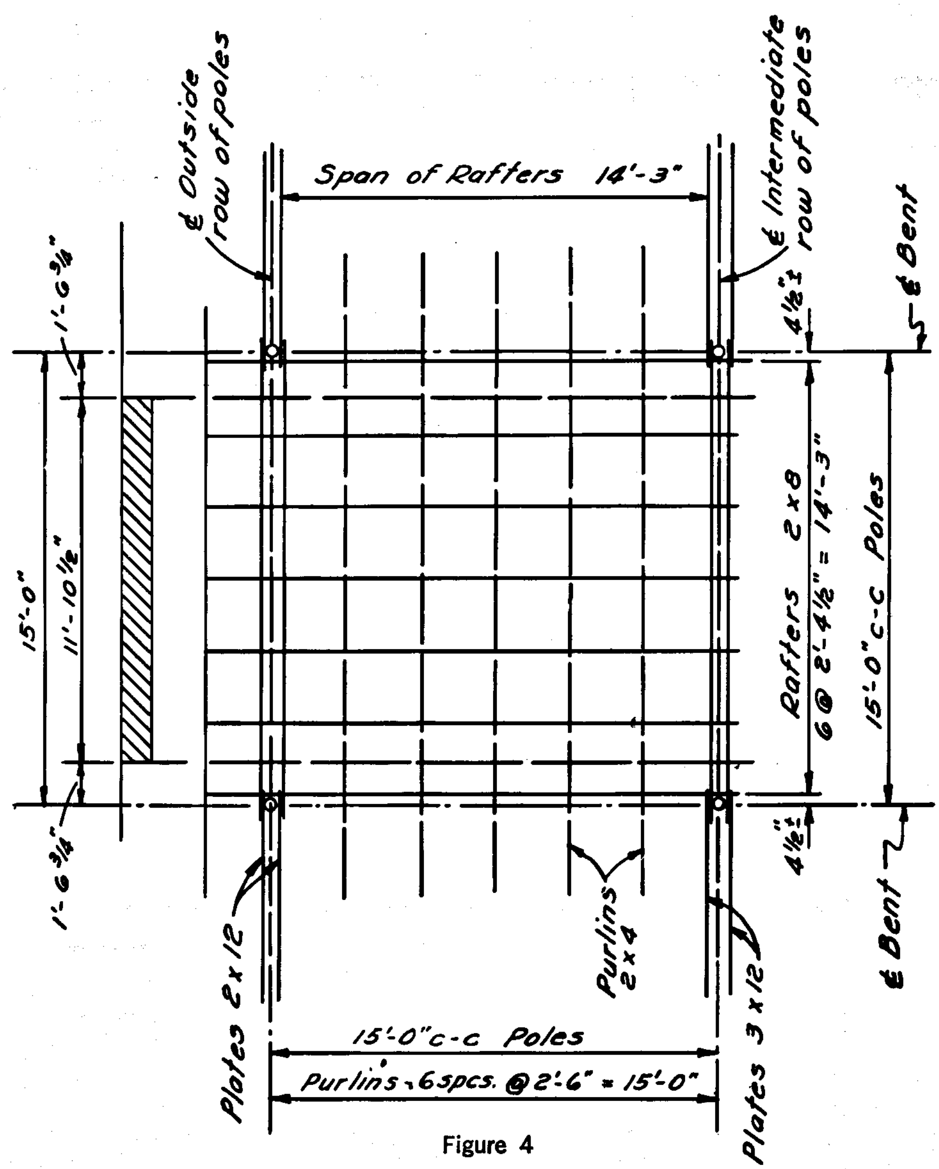


Figure 4

If the structure is low, or is located where wind pressures are moderate, light knee-braces may be used at pole tops. Taller buildings, and those subjected to near-maximum wind pressures require additional bracing. Tops of the poles in each bent should be fixed in order to reduce somewhat bending moments in poles and to ensure more equitable distribution of horizontal loads between poles in the bent.

Usually, bearing values under butt ends of poles will be unimportant. Vertical loads, including vertical components of the wind forces, generally are rather small, except in regions where maximum snow load may be expected. Most of the vertical load will be transmitted to surrounding soil through skin friction before reaching lower ends of poles. Investigators, both here and abroad, have found, in the case of piles, that in average soils, vertical loads are transferred to surrounding earth in the upper portion of each pile. In a soil with low bearing capacity, the chart, Figure 1, automatically will compensate for it and will show the increased depth of embedment required. Bearing values can be increased by backfilling the hole with concrete or soil-cement.

In cases where skin friction must be increased in order to carry vertical loads in the length of pole determined from embedment requirements, concrete encasement in the form of backfilling may be assumed to have a bond value with the wood pole of 30 lb per sq in at working stresses.

Skin friction also is effective against uplift in the case of a pole-type building that is relatively narrow and tall, or on the windward side of a building where uplift may develop on a pole through its connections to the roof structure. The American Association of State Highway Officials, in the case of piles, permits 40 percent of the allowable working load to be used for uplift against transient or temporary wind loadings.

A concrete mat under the pole butt will not increase bearing capacity unless thickness of the concrete is sufficient to withstand the punching shear of the pole, and bending in the mat. Depth of concrete never should be less than 12 in, and should be increased where heavy loads are transmitted. The same quantity of concrete placed as backfilling around the pole is a more effective method of increasing bearing capacity. The enlarged area in contact with the soil permits greater vertical loads, and enlarged diameter or breadth of the encased pole provides greater resistance to slight rotation from horizontal forces.

Design Computations

The following work is based on certain assumptions similar to those which must be made for the design of any particular building:

Maximum wind pressure, 20 psf. (This figure is in accordance with building codes of Chicago, Ill., Detroit, and Flint, Mich., the Wisconsin State Building code and others. It also is the wind pressure recommended by regional codes as applicable to the country as a whole.)

Live load; 20 psf for roofs with slopes less than 30°. (25 psf probably is the maximum that need be provided for anywhere.)

Allowable soil pressure, 2,500 psf. (This is "Average" soil in Figure 1.)

Lumber; commercial grades of Southern yellow pine or Douglas fir. A suitable stress grade to be selected from applicable grading rules.

The bulk of structurally graded lumber is cut from Southern pine and the West Coast species—Douglas fir and hemlock. Grading rules published by the two associations control grading of these species. Each contains a wide range of structural grades. These rules give all allowable working stresses, so that a satisfactory strength grade of lumber may readily be selected for any use. Current editions, "Grading Rules for Southern Pine Lumber," and "Standard Grading and Dressing Rules," are available from the Southern Pine Inspection Bureau, Southern Pine Association, New Orleans, La., and from the Western Wood Products Association, Portland, Ore., respectively.

Wood poles: Douglas fir or Southern pine with an ultimate fiber stress of 8,000 psi, USA rating.

Panels have been made 15-ft square, center-to-center of poles, to utilize 16-ft lumber. The assumed roof slope is 1:4 or a 1/8 pitch.

In applying working stresses, character of the loads must be considered. Dead load of a structure is always present and must be taken at full value. Live load, or snow load, although not always present, may last for a considerable time when it does occur. Wind load, on the other hand, is a transient, fluctuating load. When its maximum occurs, it is an extreme that lasts for only a brief period. These extremes seldom occur in conjunction with maximum live loads.

Where dead weights are very light in comparison with those imposed by maximum wind velocities, many texts recommend increasing working stresses 50 percent. This is conservative when the short duration of wind loads is considered. For sawed timbers an increase of 33⅓ percent in working stress usually is specified, but permissible working stresses for them automatically carry an increase of 10 percent or more for Normal Loading. Figures for round timbers on the other hand do not include an increase for Normal Loading. With the dead-to-live-load ratio usually found in pole-type buildings, an increase of 50 percent for wind appears justified in arriving at conservative design values for round timber poles.

Working Stresses for Combined Loadings

In the following examples normal working stresses are increased by one-third when computing the effect of combined loading, including wind pressure, which acts with maximum intensity only a few times, if at all, during life of a structure.

When dead, live and wind loads are combined, unit stresses may be increased by one-third. The vertical component of the wind load, therefore, must exceed one-third the sum of dead and live loads before it will affect the computations. Where live loads are light, and for steeply pitched roofs, vertical effect of wind load usually will enter into calculations. It always should be considered in the beginning, if only to dispose of it for the remainder of the work.

Roofs of the three types of buildings shown will be assumed to be of 26-gauge galvanized sheets on 2x4 purlins, spaced at 30-in centers, support-

Rafter Design

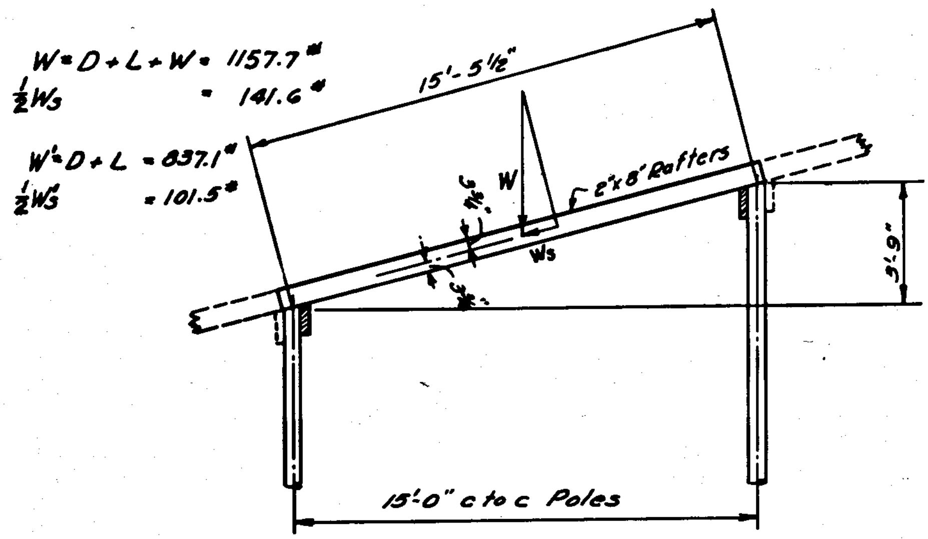


Figure 5

ed by 2x8 rafters at 28½-in centers, with a rafter on each side of each pole. Rafters will be supported on two 2x12 plates at the outer longitudinal rows of poles and on two 3x12 plates at the intermediate rows, one on each side of each pole.

The wind load on a sloping roof, based on the Duchemin formula is, $P_r = P \left[\dfrac{2 \quad \text{Sin } A}{1 + \text{Sin}^2 A}\right]$, where P is the assumed maximum wind pressure, A is the angle the roof makes with the horizontal, and P_r is wind pressure normal to the roof. From the formula, this amounts to 9.1 psf, when wind pressure is 20 psf and the roof pitch is ⅛. Vertical and horizontal components of this normal wind pressure are 8.9 and 2.2 psf respectively.

Vertical Loads on Frame

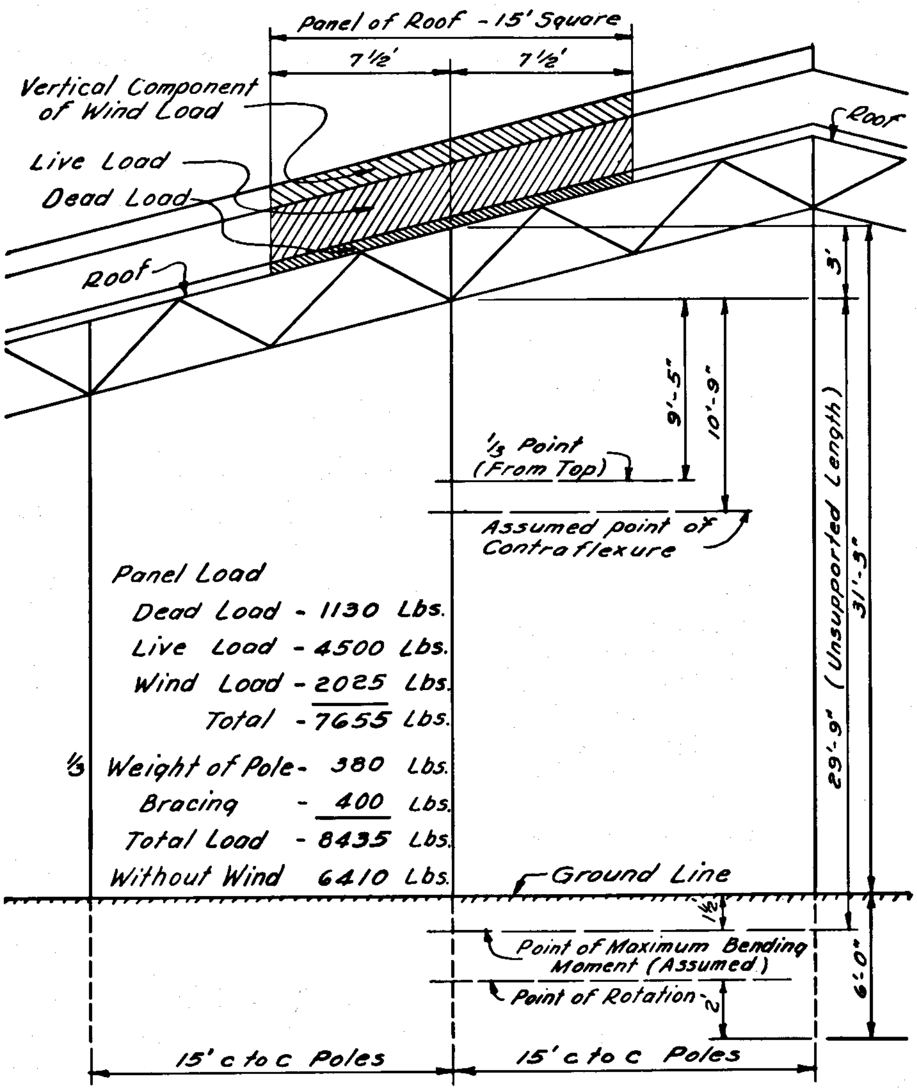


Figure 6

Rafters

Rafter loads are computed on a panel basis as follows:

Because there is a rafter on each side of each pole, and assuming the pole diameter is approximately 7 in at the top, span length of rafters will be 14 ft-3 in center-to-center of supports, and there will be six rafter spaces of 28½ in. in a panel.

Panel load (Figure 4):			
Dead load:	6—2 x 4 purlins—2.375 ft @ 3 lb/fbm	=	28.5 lb
	26-gauge sheets, 0.9 lb/ft x 2.375 x 15.0	=	32.1 lb
	2 x 8 rafter, 4.0 lb/ft x 16.0	=	64.0 lb
	Total dead load		124.6 lb
Live load:	20 psf x 2.375 x 15.0	=	712.5 lb
Wind load:	9 psf x 2.375 x 15.0	=	320.6 lb
	Total: Dead plus Live plus Wind load	=	1157.7 lb

Because of close spacing of purlins, this load may be considered as uniformly distributed over rafters.

$$M = 1157.7 \times 14.25 \times 12 \times 1/8 = 24{,}750 \text{ in-lb}$$

Referring to Figure 5, one-half the component of the vertical load parallel to the rafter amounts to 141.6 lb. Secondary moment in the rafter, due to this component, is

$$141.6 \times 7\frac{1}{2} \times \frac{1}{2} = 532 \text{ in-lb}$$

Total moment in the rafter is:

$$24{,}750 + 532 = 25{,}282 \text{ in-lb}$$

Section modulus $\frac{bd^2}{6}$ of the 2 x 8 S4S rafter is 15.23 in^3. Table V in the Appendix, page 44, gives the section moduli for a few of the more common sizes of dressed lumber used in buildings.

Stress in the rafter:

$$f = \frac{25{,}282}{15.23} = 1660 \text{ psi}$$

Since working stresses may be increased one-third for wind forces, a grade of lumber with an allowable fiber stress of three-quarters of this amount is adequate. A suitable stress grade should be selected from standard grading rules. Omitting the Wind load, bending moment in the rafter is:

$$M = \frac{837.1 \times 14.25 \times 12}{8} = 17{,}900 \text{ in-lb}$$

One-half the component parallel with the rafters amounts to 101.5 lb and the secondary moment is:

$$101.5 \times 7\tfrac{1}{2} \times \tfrac{1}{2} = 381 \text{ in-lb}$$

Total moment is 17,900 + 381 = 18,221 in-lb

$$f = \frac{18{,}221}{15.23} = 1200 \text{ psi}$$

The above selected stress also is adequate for the foregoing loading condition. If roof slopes are flat enough so that 2x4 purlins can be placed on edge to span wider spacing of rafters, a saving of about ten percent in rafter material can be made by using 2x10 rafters on 3 ft-6¾-in centers instead of 2x8 rafters on 2 ft-4½-in centers of the example.

Horizontal shear in rafters:

$$H = \frac{3V}{2bh} \text{ where } V = 1157.7 \times \tfrac{1}{2} = 579.0 \text{ lb*}$$

$$H = \frac{3 \times 579.0}{2 \times 1\tfrac{5}{8} \times 7\tfrac{1}{2}} = 72 \text{ psi}$$

Where:

H = Maximum horizontal shear on neutral plane
b = Width of member
h = Height of member
V = Vertical shear

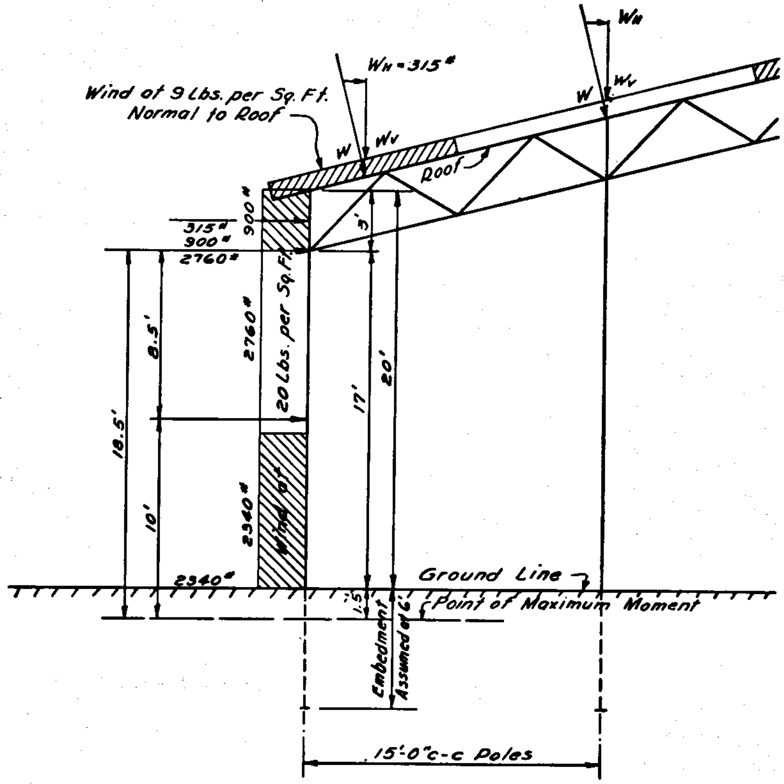
Omitting the wind load, horizontal shear is only 52 psi.

Allowable shearing stress for the grade selected for bending in extreme fiber, should be checked, of course. Practically any structural or stress-rated grade of Southern pine or Douglas fir will have an approved working stress in shear more than adequate for these calculated unit shears.

Design of Plates

Referring to Figure 4, it will be seen that only about 12 ft of the panel load actually contributes to bending moment in plates, because the two outside rafters are fastened directly to poles. Moments have been computed for the entire panel load, because the difference is only about four percent and on the side of safety. This is done because the entire panel load enters into pole computations and can be used for both pole and plate calculations. If it is desired to use the reduced moment for the plates, the formula is, $M = \frac{1}{8} W(2L-B)$, where W is the reduced panel load distributed over width B, and L is the span length. In the example given, W would be 5830 lb, L 15 ft, and B 11 ft-10½ in. Bending moment due to weight of plates would be computed separately and added to the other.

* Vertical shear is taken as the end reaction for the 1157.7 lb uniform load. U. S. Forest Products Laboratory, however, recommends omitting load within height of the beam from both supports for calculating horizontal shear on its neutral plane.



Horizontal Loads on Frame

Figure 7

Panel Load—Figure 4:

Dead load—2 x 4 purlins, 6 x 2.0 lb/ft x 16.0	=	192.0 lb
26-gauge sheets, 0.9 lb/ft x 15.0 x 15.0	=	202.5 lb
2 x 8 rafters, 7 x 4.0 lb/ft x 16.0	=	448.0 lb
2—3 x 12 plates, 2 x 9.0 lb/ft x 16.0	=	288.0 lb
Total Dead load	=	1130.5 lb
Live load— 20 psf x 15.0 x 15.0	=	4500.0 lb
Wind load— 9 psf x 15.0 x 15.0	=	2025.0 lb
Total, Dead plus Live plus Wind load	=	7655.5 lb

Because of close spacing of rafters, the load may be considered as uniformly distributed over plates.

$$M = 7655.5 \times 15.0 \times 12 \times \tfrac{1}{8} = 172{,}249 \text{ in-lb}$$
$$S = 57.86 \text{ in}$$
$$f = \frac{172{,}249}{2 \times 57.86} = 1488 \text{ psi}$$

Omitting Wind load, stress in an intermediate plate is:

$$M = 5630.5 \times 15.0 \times 12 \times \tfrac{1}{8} = 126{,}686 \text{ in-lb}$$
$$f = \frac{126{,}686}{2 \times 57.86} = 1094 \text{ psi}$$

Horizontal shear in intermediate plates:

$$V = \tfrac{1}{2} \times 7655.5 \times \tfrac{1}{2} = 1913.9 \text{ lb}$$
$$H = \frac{3 \times 1913.9}{2 \times 2\tfrac{5}{8} \times 11\tfrac{1}{2}} = 95 \text{ psi}$$

Omitting Wind load, horizontal shear becomes 70 psi.

Plates on outside rows of poles:

These outside roof panels, 17 x 15 ft in size, provide for a 2-ft overhang at the building edge. Reaction at outside plates is about 6/10 of panel weight, approximately equivalent to a panel 10 x 15 ft in size.

Panel Load—Figure 4:

Dead Load—2 x 4 purlins, 4 x 2.0 lb/ft x 16.0	= 128.0 lb
26-gauge sheets, 0.9 lb/ft x 15.0 x 10	= 135.0 lb
2 x 8 rafters, 7 x 4.0 lb/ft x 10	= 280.0 lb
2—2 x 12 plates, 2 x 6.0 lb x 16.0	= 192.0 lb
Total Dead load	= 735.0 lb
Live load—20 psf x 15.0 x 10	= 3000.0 lb
Wind load—9 psf x 15.0 x 10	= 1350.0 lb
Total, Dead plus Live plus Wind load	= 5085.0 lb

$M = 5085.0 \times 15.0 \times 12 \times \frac{1}{8} = 114{,}413$ in-lb

$S = 35.82$ in

$$f = \frac{114{,}413}{2 \times 35.82} = 1600 \text{ psi}$$

Omitting wind load, stress in an outside plate is:

$M = 3735.0 \times 15.0 \times 12 \times \frac{1}{8} = 84{,}038$ in-lb

$$f = \frac{84{,}038}{2 \times 35.82} = 1173 \text{ psi}$$

Horizontal shear in outside plates:

$V = 5085.0 \times \frac{1}{2} \times \frac{1}{2} = 1271.3$ lb

$$H = \frac{3 \times 1271.3}{2 \times 1\frac{5}{8} \times 11\frac{1}{2}} = 102 \text{ psi}$$

Omitting Wind load, horizontal shear becomes 75 psi.

Fastening Plates and Outside Rafters

These roof members will be bolted to poles, and the allowable stresses in compression, perpendicular to the grain, will govern.

Referring to Figure 4, page 12, an interior rafter supports a strip of roof panel 2-ft-4½-in, or 28.5-in wide. A 1 ft 6¾-in, or 18.75-in width of the outside strip is carried to the rafter at the pole line. The end reaction of this rafter is, therefore, $\frac{18.75}{28.5}$ or 0.66 of the end shear computed for an interior panel on page 16, (579 lb). This end reaction amounts to 385 lb when

Bending Moment Diagram

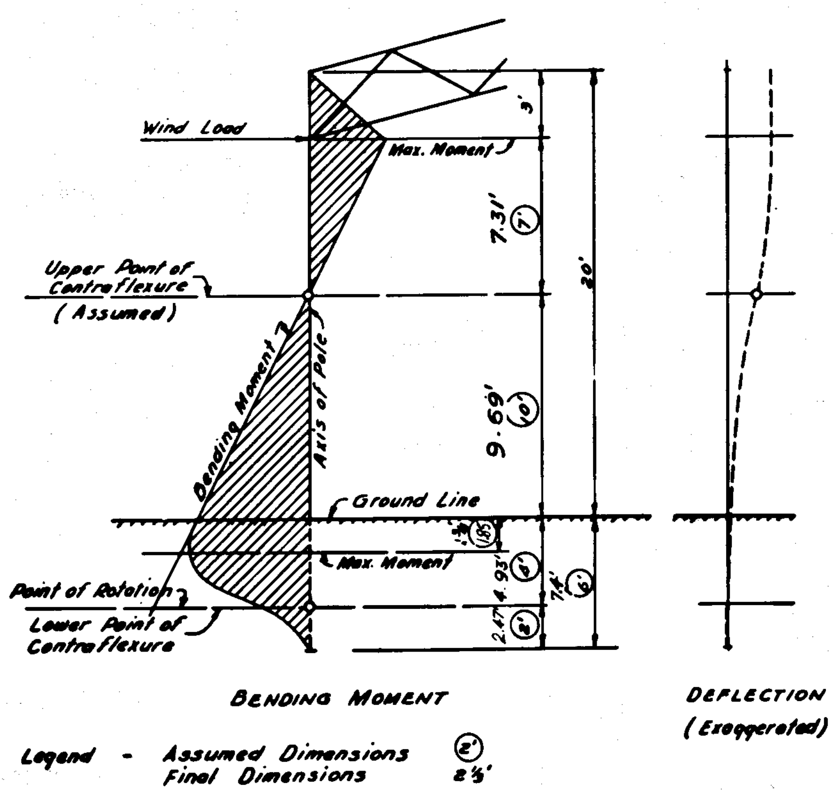


Figure 8

wind load is included, and 280 lb when wind is omitted. A ½-in diameter bolt through the 1⅝-in thick rafter sloping 1 in 4, will carry a single end load of 480 lb.

Plates on intermediate rows of poles, two 3 x 12 timbers, have an end reaction of 1410 lb each, without wind load, and 1915 lb each when wind load is added, (pages 17-18). At this connection where the plates are butt joined on 6 inch diameter poles, adequate bolting for this vertical load is impracticable. Four ½ inch bolts will be used to hold plate ends snugly against the pole, and a cleat to carry the vertical reaction will be bolted and nailed along the pole axis under each plate line. These cleats usually are 2x4 inch pieces 3 to 4 feet long. Bolts or nails attaching them to the pole act parallel to the grain of the wood and are embedded for their full length. For these plate reactions, two ½ inch diameter bolts through the pole and two cleats provide vertical support. Four or five 60d nails will also strengthen the connection. Shear on the section above the lower bolt should be checked against allowable longitudinal shear:

$$H = \frac{V}{bh} = \frac{1410}{2\frac{5}{8} \times 9} = 59 \text{ psi}$$

Plates on outside rows of poles, two 2x12 timbers, have an end reaction of 895 lb each, without wind loads, and 1215 lb each when wind load is added, (pages 17-18). Plate ends will be clamped to the pole with two ½ in diameter bolts, and 2x4 cleats nailed and/or bolted along the pole axis to support the vertical load. Shear on the section above the lower bolts should be checked against allowable longitudinal shear:

$$H = \frac{V}{bh} = \frac{895}{1\frac{5}{8} \times 9} = 61 \text{ psi}$$

(Note: see appendix for method of computing strength of bolted joints.)

The vertical load on an interior pole is next computed, Figure 6, page 15:

Round Timber Columns

Figure 6 has been drawn with an intermediate pole that equals height of the center one shown in Type A, Figure 3. This was done to obtain the effect of the vertical component of a full panel of wind load on the pole under consideration, when its length equals that of the center pole.

Dead + Live + vertical Wind load on an interior pole = 7655 lb

Dead + Live load on an interior pole = 5630 lb

Weight of a 40-ft, Class 3 or Class 4 pole = 1150 lb approximately.

One-third of this weight, 380 lb will be assumed to be that portion of the pole which is above the critical section, one-third the distance down from the top.

Another 400 lb will be assumed as a panel load due to bracing.

Total Dead + Live + Wind loads = 8435 lb

Dead + Live load only = 6410 lb

The pole has an unsupported length more than 11 times its least dimension and is therefore a column of intermediate class. Allowable unit stress in compression parallel to the grain in an axially loaded column of this class is determined from the formula,

$$\frac{P}{A} = \frac{3.6E}{\left(\frac{l}{r}\right)^2}$$

in which:

P is the total load on the pole, in lb.

A is the cross-sectional area of the pole one-third its length from the top, in sq in.

E is the modulus of elasticity, taken as 1,600,000 psi for Southern pine or Douglas fir. (E for other species shown in Table IV of Appendix).

3.6E for Southern pine or Douglas fir = 5,760,000.

l is the unsupported length, in inches.

r is the least radius of gyration, which is equal to one-fourth the diameter (inches), and should be taken at a point one-third the distance down from the top.

$$r^2 = \frac{d^2}{16} = \frac{C^2}{158}$$

C is the circumference in inches.

The slenderness ratio $\frac{0.289l}{r}$ should not exceed 50 and unit compressive stress at the small end of the pole should be checked. It should not exceed the allowable unit stress for a short column as given by the formula, $\frac{P}{A} = S$, in which P and A are the same as given above except that A is taken at the top of the pole instead of one-third down, and S is the resulting unit compressive stress parallel to the grain.

The cross-sectional area A and the least radius of gyration r of a pole at one-third its length below the top, are computed from the pole's circumference at that point. Dimensions at this third point are conveniently derived from circumferences at 6 ft from the butt listed on Table VI of the Appendix.

The tabulated dimension is the minimum permitted in a class. The average for the class is appreciably larger. Stresses computed on the basis of tabular dimensions are, therefore, considerably above those actually occurring in the average pole. Consequently, sizes selected for computed stresses are conservative.

The average taper for Southern pine and Douglas fir poles can be taken conservatively at 0.25 in. in circumference per foot of length. The pole in the first row right or left of center in Figure 6 is approximately 38 ft long with an unbraced length l of 29.75 ft, or 357 in.

The critical point at two-thirds the pole height in 24.8 ft, or approximately 25 ft above the bottom of the pole, and approximately 19 ft above the 6-ft mark for which Table VI shows circumferences of 36, 33½ and 31 inches respectively for 40-ft poles of Classes 3, 4 and 5. Circumferences at the critical point, assuming a taper of 0.25 in. in circumference per foot of length, are 19 × 0.25, or 4¾ in smaller than at the 6-ft mark, and for these poles are 31¼, 28¾ and 26¼ in. Corresponding values of r^2 are 6.2, 5.2 and 4.4 in.,

Analysis of Poles Under Truss Span

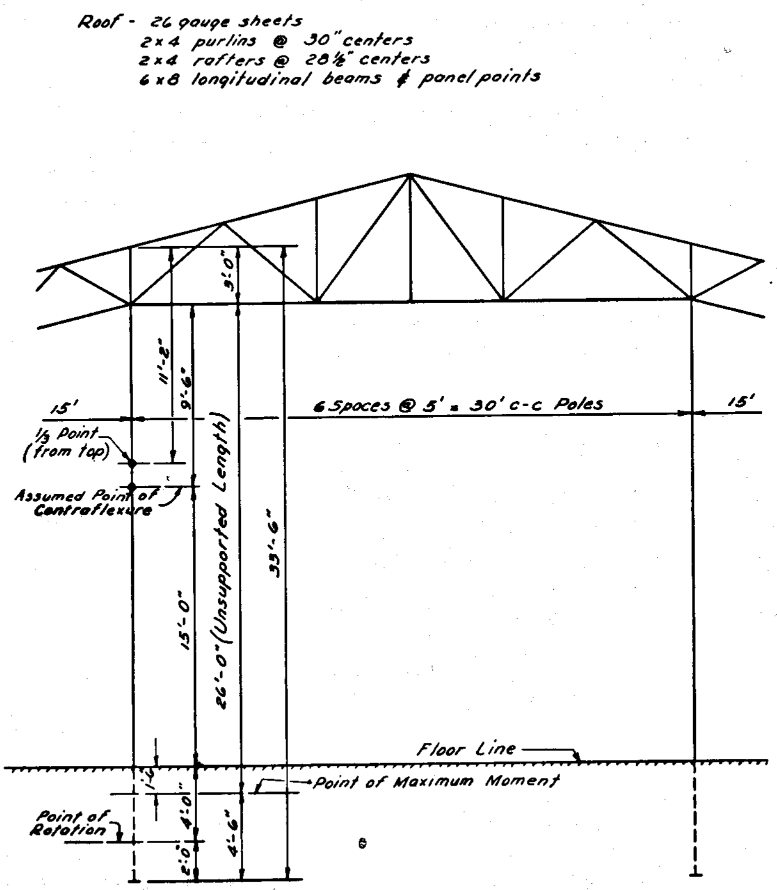


Figure 9

and the unbraced length shown in Figure 6 is 29.75 ft or 357 in. Substituting these values in the column formula, allowable unit stresses are found to be 280, 235 and 200 psi for these three classes of poles.

Actual loads assumed for design of this column are 8435 lb with, and 6410 lb without wind. Actual load without wind, viz. 6410 lb results in axial stresses on a cross-section at the one-third point of 83, 98, 117 psi respectively for the three classes of poles considered. They are well below allowable stresses determined from the formula.

Circumferences at tops of the above class poles are $3\frac{1}{4}$ in, (13×0.25 in) smaller than at the critical point, one third the height, or in this case, 13 ft below the top. Accordingly, minimum top areas for Classes 3, 4 and 5 poles are respectively 62.4, 51.7 and 42.1 sq in. Unit compression in even the smallest of these poles under the design loading is 200 psi when wind force is included, and 152 psi when wind is omitted. These values are only a small fractional part of allowable short column stress for any pole species. Generally, any safe load on long pole columns will be well under that allowable for a short column with area equivalent to the pole top.

Any class of pole down to and including Class 5 is capable of carrying vertical loads at the highest part of the building. At the outside rows of poles, both vertical loads and unsupported length of pole are less, so any of these classes of poles will suffice. Finally, however, horizontal loads from wind pressure will determine sizes of poles in the outside rows.

Horizontal wind load, based on building codes of Chicago and Detroit, is taken at 20 psf of vertical wall. Wind load on a sloping roof, perpendicular to the surface from the Duchemin formula, page 14, is 9.1 psf. The horizontal component for the assumed 1:4 roof slope is 2.2 psf.

With a height of 20 ft at the eaves, tops of poles will be fixed by truss bracing 3-ft deep in the plane of the bent, and by knee braces in the plane perpendicular to that of the bent. It will be assumed, as explained under embedment, that maximum bending moment in a pole will occur at approximately one-quarter of its embedded depth below the ground surface. This may be taken, temporarily, at $1\frac{1}{2}$ ft below the surface.

Distribution of Load To Columns

Horizontal loads, then, on the outside poles of the bent are as follows, (Figure 7):

$$20 \text{ psf} \times 15 \times 3 = 900 \text{ lb @ top}$$
$$20 \text{ psf} \times 15 \times 17 \times 10/18.5 = 2760 \text{ lb @ top}$$
$$20 \text{ psf} \times 15 \times 17 \times 8\tfrac{1}{2}/18.5 = 2340 \text{ lb @ bottom}$$
$$2.2 \text{ psf} \times 15 \times 9.5 \text{ (roof)} = 315 \text{ lb @ top}$$

Total horizontal load applied at top, 3975 lb.

With rigid bracing at the tops of the poles, it is assumed that the force at any pole top will be distributed through the bracing to other poles in the bent. Because pole and bracing connections are not completely rigid and since they naturally tend to wear with repeated loadings, the portion of the load on any one pole which is transferred to others in the bent will diminish as their distances from the loaded pole increase. While this distribution may extend, with decreasing effect, to a number of poles, it is probably of no great effect beyond four or five poles. In the present example, the distribution has been limited to the four poles on the windward side of the frame, which would appear to be reasonable and conservative. Assuming the decreasing effect to be in the ratios 4 — 3 — 2 — 1 as the poles considered are further from the one loaded, we obtain factors of 4/10, 3/10, 2/10, and 1/10 of the wind load on the outside pole for the four windward poles, and factors of 3/12, 4/12, 3/12 and 2/12 for the same poles when the load considered is that at the top of the pole next to the outside one. The same two groups of factors, reversed, will apply to the next two poles loaded. In the case illustrated the loads from wind on the interior poles are not very large, although on roofs with steeper pitches these load increments would be considerably greater.

Using these coefficients, loads to be distributed at tops of windward poles are broken up as follows:

	Pole 1 Outside	Pole 2	Pole 3	Pole 4	Total
		Load in lb at Top of Pole			
Load from Pole 1	1595	1190	795	395	3975
Load from Pole 2	125	165	125	80	495
Load from Pole 3	80	125	165	125	495
Load from Pole 4	25	50	75	100	250
	1825	1530	1160	700	5215

Point of Contraflexure in Columns

If poles were not tapered, the point of contraflexure would be at mid-height. However, poles are frustums of cones, tapered approximately 0.25 in per ft in circumference and the moment of inertia, $\frac{C^4}{64\pi^3}$ increases at an accelerated rate from top to bottom. In a Class 2 pole, 40-ft long, as the diameter increases from top to butt from approximately 8 in to 13 in, the moment of inertia increases from 200 to 1400 in^4. The effect of this taper on a pole fixed at each end, with a horizontal thrust applied at the top, is to move the point of contraflexure somewhere above mid-height.

Here it will be assumed that the point of contraflexure occurs at a point two-thirds the height from the point of rotation to bottom strut of the bracing at top of the pole. This appears to be reasonable. Referring to Figure 8, it will be evident that lowering the point of inflection will decrease the lower bending moment and increase the upper. An assumption must be made and the two sections checked against their respective bending moments, and, if necessary, an adjustment made in this height until stresses are in approximate agreement at the two sections. This check will be made in the present example. Assuming a six-ft depth of embedment, this distance from point of rotation to bottom of bracing is 21 ft. Applying this two-thirds factor to the 21 ft of pole, places the point of contraflexure 14 ft above the point of rotation or 10 ft above the ground line.

To convert effect of 2340 lb of wind reaction at the ground line to an equivalent force at the point of contraflexure, multiply it by the factor $\frac{4}{14}$. It then amounts to 670 lb at a height of 10 ft above the ground. Total load, therefore, to be applied to the pole 10 ft above surface of the ground is 2495 lb.

Embedment of Columns

Since allowable soil stress has been assumed to be 2500 psf for these examples, the only other factor needed to determine embedment is b, breadth of the embedded pole. For poles 30-ft long, of Classs 2, 3, 4 and 5, diameters, derived from minimum circumferences at six ft from the butt, given in Table VI in the Appendix, are 10.82, 10.19, 9.39 and 8.75 in respectively. Entering the chart, Figure 1, at 2500 on the soil scale (S_1), and using a load of 2495 lb on the P scale, a factor of approximately 1.0 (computed, this is 1.002), is obtained on the C scale. Starting from this point and using four values of b for the four classes of poles, 10.8, 10.2, 9.4 and 8.8 in respectively, points on the L scale are obtained, ranging downwards from 1.11 to 1.36. These values, when projected horizontally to the right to height of load curve corresponding to 10 ft, then from that intersection downward vertically to the depth-of-embedment scale, give values of approximately 6.9, 7.2, 7.5 and 7.8 ft respectively, for the four classes of poles. A Class 2 pole, with an indicated embedment of 7 ft instead of the assumed 6 ft, if refigured, would give an approximate required embedded depth of 7.1 ft. The other three classes of poles would require 7.4, 7.7 and 8.0 ft respectively.

Maximum bending moment on a pole, from the above figures is 2495 lb $\times$ 11.5 ft = 28,695 ft-lb, or 344,340 in-lb.

Round timbers have two distinct advantages from a strength standpoint. A circular timber has a form factor of 1.18. This means it is eighteen percent stronger in bending than a rectangular timber of similar grade with the same section modulus. A round timber pole or pile, in practically all cases, possesses a very high proportion of the basic strength of its species, because knots have only half the effect on strength of natural round timbers that they do on sawed sections. Results of numerous tests show that full size round timber poles develop practically the full bending strength of clear wood. Strengths in bending and compression parallel to the grain, in round timbers where limbs have not been trimmed so closely that excessive cross grain is exposed, are reduced very little by knots. The average pole too has a greater circumference than is indicated by the minimum for classes and lengths given in Table VI (Appendix). Dimensions of an average pole of each class, therefore, are appreciably larger than the tabulated minimum.

Where dead weights are very light in comparison with those imposed by maximum wind velocities many texts recommend increasing working stresses 50 percent for wind forces. This is conservative when short duration of wind loads is considered. For sawn timbers an increase of 33⅓ percent in working stress usually is specified, but permissible working stresses for stress-grade lumber automatically carry an increase of 10 percent or more for Normal Loading, whereas test figures for round timbers do not include an allowance for Normal Loading. With the dead-to-live-load ratio usually found in pole-type buildings, an increase of 50 percent for wind appears justified in arriving at conservative design values for round timber poles.

Working Stresses for Cantilever Poles

In selecting safe working stresses in bending for cantilever poles conforming to USA Standards Institute specifications and dimensions, a high strength grade of wood can be assumed. Few if any knots occur in the lower half of a pole where the greatest stress develops. An allowable stress of 1900 psi for extreme fiber in bending is conservative for a comparable grade of sawed timber of Southern pine or Douglas fir. Because of the circular form factor this permissible stress can be increased 18 percent. The resulting 2125 psi can be increased by 50 percent for wind loads, which are of short duration. This gives a working stress of 3200 psi and provides a factor of safety of approximately 2½ when compared with the USA ultimate value of 8000 psi for those species in cantilever bending. Similar working stresses for other pole species in buildings of this type can be obtained by applying this factor of safety of 2½ to their USA ratings. The Outdoor Advertising Association of America, Inc., Engineering Design Manual uses 3780 psi working stress in bending for Southern pine and Douglas fir poles.

An allowable fiber stress of 3200 psi in bending for poles of Southern pine or Douglas fir, with a safety factor of 2½, as previously obtained, will be used in these examples. The section modulus for a circular section is:

$$S = \frac{\pi D^3}{32} \text{ and is equal to } \frac{M}{f}$$

$$D^3 = \frac{32M}{\pi f} \text{ or } D = \sqrt[3]{\frac{32M}{\pi f}}$$

$$D = \sqrt[3]{\frac{32 \times 344{,}340}{3200\pi}}$$

Because pole circumferences are more easily measured, dimensions of sections usually are given in terms of circumferences rather than diameters. The above formula for the section modulus of a circular section in terms of the circumference is:

$$S = \frac{C^3}{32\pi^2} \text{ which is equal to } \frac{M}{f}$$

$$C^3 = \frac{32\pi^2 M}{f} \text{ or } C = \sqrt[3]{\frac{32\pi^2 M}{f}}$$

$$C = \sqrt[3]{\frac{32\pi^2 \times 344{,}340}{3200}} = 32.4 \text{ in}$$

This 32.4-in circumference is approximately that of a Class 3 pole at $1\frac{1}{2}$ ft below the ground line, or at the point used in figuring maximum bending moment in the pole. A Class 3 pole was found to require an embedment of 7.4 ft instead of the assumed six.

With a 7.4-ft depth of set, height of the point of inflection above the ground line becomes 9.7 ft, or 11.5 ft above the point of maximum bending moment—about the same as before. The Class 3 pole, therefore, is adequate so far as lower bending moment is concerned.

Maximum Bending Moment

The bending moment also is a maximum for the smaller diameter of a tapered pole, at bottom of the bracing. The distance of this point above the point of contraflexure is 7.31 ft, and the bending moment is 2495 lb × 7.31 ft, or 18,150 ft-lb, or 217,800 in-lb. The pole diameter for the required resisting moment is 8.8 in, which is approximately the diameter of a 30-ft Class 3 pole at this point.

This gives a rough check on the assumption that point of inflection will occur two-thirds the height from point of rotation to the bottom strut of the top bracing. For the given pole, stresses due to bending are approximately equal at the two sections.

In computing circumferences at points of maximum bending, distances should be figured from the dimensions 6 ft from butt ends of poles.

Repeating this procedure for poles in rows next to the outside ones, horizontal thrust is 1530 lb, all concentrated at tops of the poles. Height of pole above the ground line is 23 ft-9in. Height from the bottom of bracing to the point of rotation is 24 ft-9in. The point of contraflexure is 16 ft-6 in above the point of rotation, 12 ft-6 in above the ground line, or 14 ft-0 in above the point of maximum bending moment. Bending moment of a pole in this row is, 1530 lb × 14.0 ft × 12 = 257,040 in-lb, which, substituted in the formula for C, page 27, gives a required circumference of 29.4 in which corresponds to a Class 5 pole 35-ft long. Using the chart, Figure 1, and these figures, required depth of embedment is 6.1 ft. It already has been determined that a Class 5 pole will satisfy requirements for vertical loads on poles in all rows.

The two intermediate rows will need 35-ft poles and the center row 40-ft poles. It probably would be advisable to make embedment depths the same as those of outside rows of poles, for the sake of uniformity, and to take better care of vertical loads through skin friction. Generally, size and embedded depth of poles in outside rows will be governed by horizontal loads, and those in intermediate rows by vertical loads. Both types of loading should be considered, however, except where smaller classes of buildings are being planned.

Where excessive wind forces indicate a need for large poles, it may be well to consider some expedients for reducing size of poles needed to carry horizontal thrust.

Restraint Provided by Floor Slab

Many warehouses will be built with concrete floors for various reasons pertaining to operation. Such floors, by restraining poles at the ground line, tend to raise maximum bending moment in poles to a point near the surface, and to increase slightly height of the point of inflection above the surface, while decreasing amount of the bending moment. A concrete floor also reduces the total horizontal thrust taken by poles in an outside row. The slab directly takes the wind reaction otherwise taken by the pole itself in bending. A note of caution is necessary here, because the slab surrounding poles must be capable of taking, in bearing, the entire thrust of the wind against them. This may require a thickened slab adjacent to the poles, some reinforcement for distributing the thrust, or both. Savings in the size of poles needed effected by use of a slab may be a deciding factor in determining the type of floor to be used in a building.

Another way of reducing horizontal thrust on poles of a bent is to insert intermediate poles in outside rows, between the bents. This not only reduces thrust on outside poles of the bent, but thrust distributed through them to interior poles. This expedient need be used only in buildings with high walls, where long, large poles otherwise would be necessary.

The bottom strut of top bracing in the outside pole of a bent can be made stiff enough to transfer more of the principal horizontal thrust to interior poles. Where bents are high, bent spacings wide, or wind loads large, this means may be used to reduce thrust on outer poles and to distribute it to other poles in proportion to their capacities.

Poles for Tall Buildings

Reducing unsupported length of long poles in tall buildings, will, in some cases, permit the use of smaller poles to carry vertical loads. The poles would have to be braced in both directions at about their inflection points. Horizontal struts between poles in the plane of the bent also would aid materially in distributing wind loads among all poles of a bent.

It may appear self-evident that vertical loads alone would be critical for inner rows of poles, yet it always is advisable, except in the case of small, low structures, to check the next to the outer row for horizontal loads, as was done in the preceding example.

Although dimensions and form used in the case just discussed were taken from Type A, Figure 3, the method used and the result obtained apply to the other two types, with a few modifications. In frames of Types B and C, it can be assumed that horizontal loads affect poles beyond the center line, if the bracing or truss over the central bay is designed to carry the thrust between tops of the poles.

In buildings of Type C, in which a wide central bay is spanned by trusses, analysis of poles supporting trusses under vertical loads will be basically the same as Types A and B. There will be, however, approximately 1½ times as much vertical load, on these poles as on those of the other two cases, by reason of the longer center span. The relationship between lengths of the central span and side spans will vary according to use requirements of the building. Hence, loads from the truss should be derived from the structure rather than from a ratio of span lengths. Roof framing over the wide span will have to be computed for each case.

Roof framing and the truss will be figured later. Loads on the pole will be assumed as follows:

One-half panel load from the 15 ft side span, including bracing	4025 lb
One-half truss load from the central span	8095 lb
One-third weight of pole	380 lb
Total load on pole at critical section	12,500 lb

A Class 4 pole, 35 ft long, **Figure 9**, with unsupported length of 26 ft or 312 in, has a circumference at the one-third point of 27 in. The radius of gyration at the same point is 2.15 in. The modulus of elasticity, E is 1,600,000 psi. Allowable unit stress on the section is from the formula:

$\frac{P}{A} = \frac{3.6\ E}{(l/r)^2}$ is 273 psi. The actual unit stress is $\frac{12{,}500}{58.09}$

or 215 psi.

Rafters on Trussed Roofs

Analysis of rafters and plates for the type of roofing assumed for this building will be similar for each of the three types of frame. Application of vertical and horizontal loads to supporting poles also will be similar. Wind load on the roof is analyzed to see whether the vertical component adds more than the allowable one-third to total dead and live loads on the structure. If so, these vertical components of wind load on the roof are added to the other two classes of load and basic unit stresses augmented by one-third are used in proportioning required framing members. If wind loads are less than one-third the total dead and live load, they can be neglected as vertical forces on the structure. Vertical loads are applied to the longest pole in the frame and its size determined. Horizontal forces, from wind, are computed and distributed between poles of each bent. Both the outside row of poles and the row next to the outside are analyzed with the applied forces to determine size and length of poles required. A comparison is made with the size obtained from consideration of vertical loads. In the case of wind pressures greater than 20 psf, it will be necessary to apply the horizontal thrust, as it is assumed to be distributed, to all poles of the bent to make certain that each will carry its portion of horizontal loading. This completes work on the bent proper for whichever type of frame has been adopted, except for determining bracing at the top of poles and trusses in Type C.

Transverse Bracing

Figures 10, 11 and 12 are outlines of typical transverse bracing that can be adapted to any one of the three types of frame. These systems are somewhat similar and their use will be determined by the same considerations for any of the frames selected.

If a building is low, or horizontal wind pressures are moderate, light knee braces between pole tops and adjacent rafters will be sufficient. In this case, poles in outer rows should be designed to take all of the horizontal thrust on the side of the building. Only the horizontal component of wind load on the roof goes to inner rows of poles. Most of the vertical load on the roof will be taken by inner rows of poles. Outer rows carry only half panels of live and dead roof load, plus dead load of the outside walls.

If panels of the bents are narrow, knee braces also may be short. Wider panels require longer knee braces at an angle of about forty-five degrees to the vertical. Knee braces should meet rafters somwhere near quarter-span points. For light knee braces, timber sizes will be nominal, usually 2×4's or 2×6's. They should be arranged in pairs and should brace poles in both directions, that is, in the planes of both rafters and plates. These braces help fix tops of the poles and thereby increase their effectiveness in resisting any transverse or longitudinal horizontal forces that may be transferred to them from other panel points, through the rafters and plates of the roof framing, and through the bracing itself. Their connections to poles, rafters and plates may be made by nailing.

Another means of effecting the same purpose, in the case of relatively short panels, is to use cross-bracing between pole tops. In this case the

Transverse Bracing Type A Frame

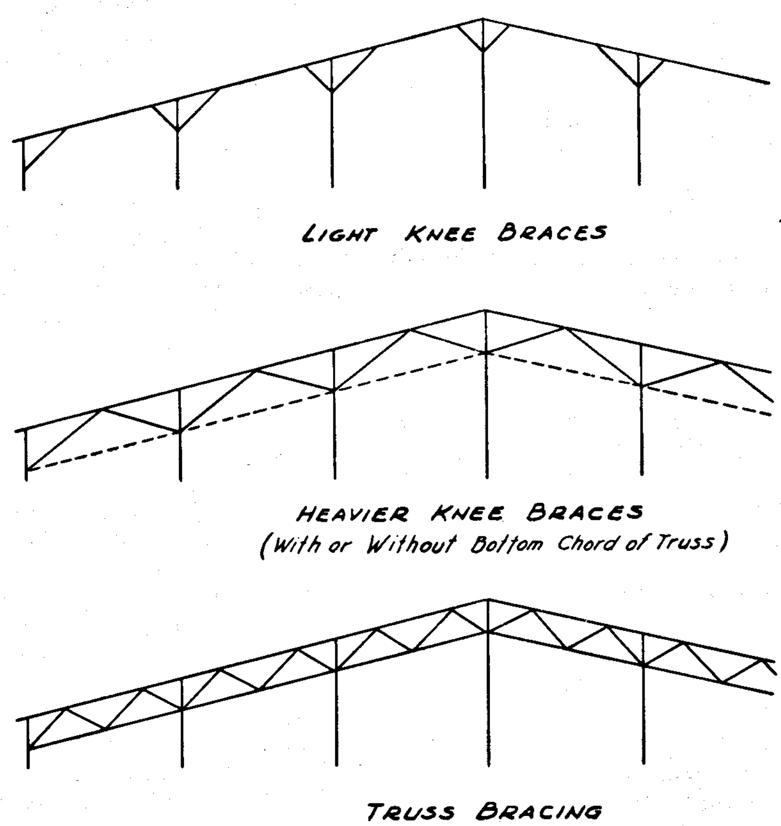
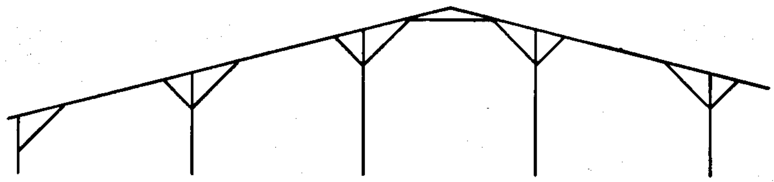
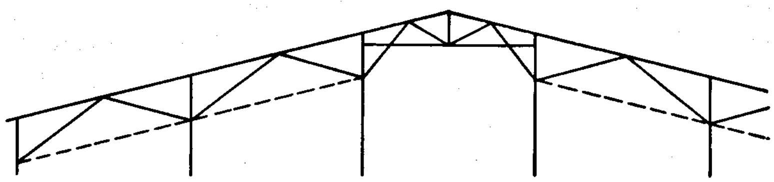


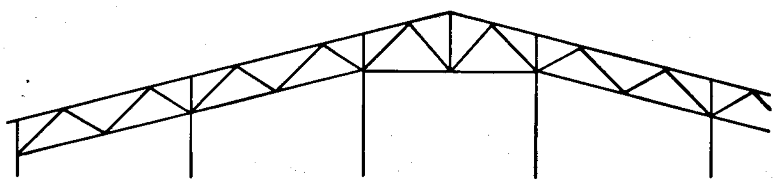
Figure 10



LIGHT KNEE BRACES



HEAVIER KNEE BRACES
WITH CENTRAL TRUSS
(With or Without Bottom Chord of Truss)



TRUSS BRACING

Figure 11

Transverse Bracing Type B Frame

bracing members will be in tension and their sizes will not depend on slenderness ratios $\left[\frac{L}{D}\right]$, as in the case of knee braces in trusses which follow. To be effective, their lower ends should be connected to poles far enough down to give as much depth to the panel as possible. These cross bracing members generally should be 2-in-thick lumber. Nailing, as in the case of light knee braces, usually will be adequate for any such member.

In buildings where transverse and longitudinal pole spacings are larger, where pole heights are greater, or where wind pressures are more extreme, provision must be made to transfer a considerable portion of the horizontal load from the pole to which it is immediately applicable to other poles in the bent, and also, longitudinally, to several poles from ends of the building in each row. This can be done in several ways: (a) by attaching longer knee braces to quarter points of the rafters or plates; (b) by attaching longer, heavier knee braces to mid-points of roof members; (c) by using shorter, lighter, triangular panels of bracing between rafters, or plates, and parallel members acting as the lower chords of trusses between pole tops. These types of bracing are shown in Figures 10, 11 and 12.

Design of Sawed Timber Columns

The allowable load in compression on a piece of sawed timber depends on its slenderness ratio, L/D, which should not exceed 50. In the case of 2-in material (1⅝ in dressed thickness), this maximum value of the L/D ratio restricts the unbraced length for which such material may be used as a strut to 81 in. For thicker timbers this maximum length increases to 11 ft-0 in for 3-in material (2⅝ in dressed), and to 15 ft-0 in for 4-in material (3⅝ in dressed).

The formula for allowable unit stress in compression parallel to the grain for long columns, in which the range of L/D is from about 28 to 50, is:

$$\frac{P}{A} = \frac{0.329E}{(L/D)^2}$$ For the following examples,

Transverse Bracing Type C Frame

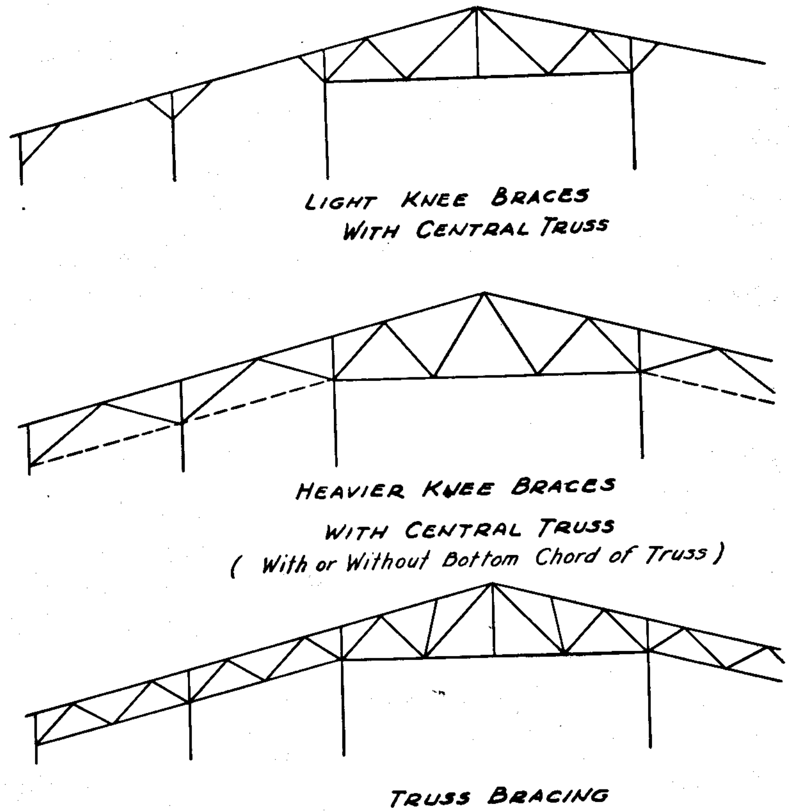


Figure 12

based on the use of Southern pine or Douglas fir, E is taken as 1,600,000 psi. The allowable unit stress in compression parallel to the grain, c, for short columns up to L/D equals 11, is taken in the following examples at 1,000 psi. Values for other species may be obtained from timber handbooks or from regional grading rules. Using this formula, the allowable unit compressive stress in long columns will vary from 667 psi where L/D is 28, down to 211 psi where L/D is 50, the variations being inversely as the square of L/D.

Knee bracing, using 2-in lumber, is limited to an unspported length of 6 ft 9 in. This length, in turn, restricts the distance down from top of a pole to its lower bracing connection to approximately 3½ ft. This applies if the roof slope is one to four, as given in these examples, and the brace makes an angle with the pole of about forty-five degrees.

Where a large horizontal force, due to high wind pressure or a large vertical panel of outside wall, is to be distributed to interior poles of the transverse bent or to longitudinal rows of supports, more adequate bracing must be provided. Longer knee braces may be used, dropping the lower end

Heavy Knee Brace Design

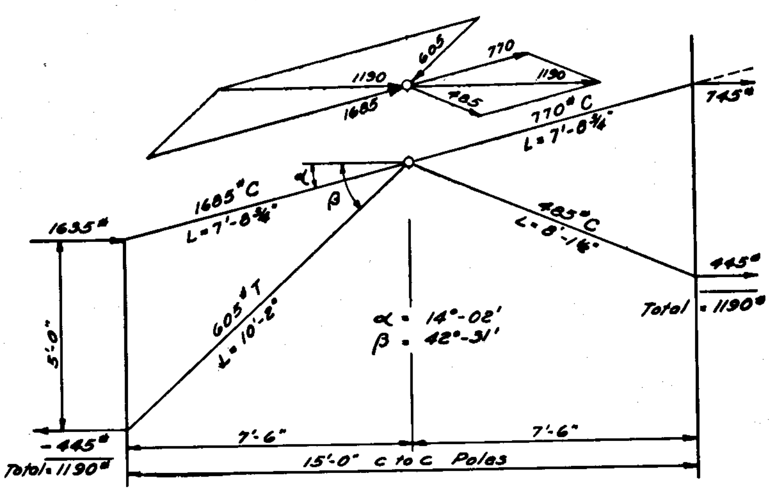


Figure 13

connection down the pole and connecting the upper end to mid-point of the rafter.

In the preceding example (see page 22), a force of 1190 lb was to be transmitted from the outer pole of the bent to the inside one. This was based on an assumed wind pressure of 20 psf, 15-ft pole spacings in both directions, and a height of 20 ft at the eaves. Referring to Figure 13, this force of 1190 lb applies to bracing in the outside panel. It diminishes in panels nearer center of the building.

Loads Carried by Rafters

Total horizontal force in the panel is given as 1190 lb. Analyzing it and forces around the point where the two braces meet rafters at center of the panel, shows forces distributed as follows: rafters at left of the point, 1685 lb in compression; those at the right, 770 lb in compression; in the brace, located lower left, 605 lb in tension; in the brace, located lower right, 485 lb in compression. These forces are in equilibrium about the point.

For each two rafters adjacent to poles, shown in Figure 13, unsupported length in compression is approximately 7 ft-9 in. Therefore, instead of 2×8 dressed sizes used for intermediate rafters that support the roof, it will be necessary to use 3-in dressed, or 2-in rough timbers for these rafters at the poles. One piece of 3×8 dressed timber is good for an axial load of 4150 lb (19.69 sq in × 211 psi), for an unsupported length of 11 ft-0 in ($2\frac{5}{8}$ in × 50). For a given length of 7 ft-9 in, with an L/D of 93 divided by $2\frac{5}{8}$, or 35.5, the allowable load of 4150 lb is increased by the ratio 2500/1260 (inversely as the squares of L/D), to an allowable 8230 lb for one, or 16,460 lb for two rafters. If 2×8 rough lumber is used, maximum unsupported length for L/D equal to 50 becomes 8 ft-4 in, and the allowable load in compression, 3375 lb. For the 7 ft-9 in unsupported length, with an L/D of 93 divided by 2, or 46.5 the allowable is increased by the ratio 2500/2162 to 3900 lb for one or 7800 lb for two rafters.

Stress Diagram—Dead Plus Live Loads

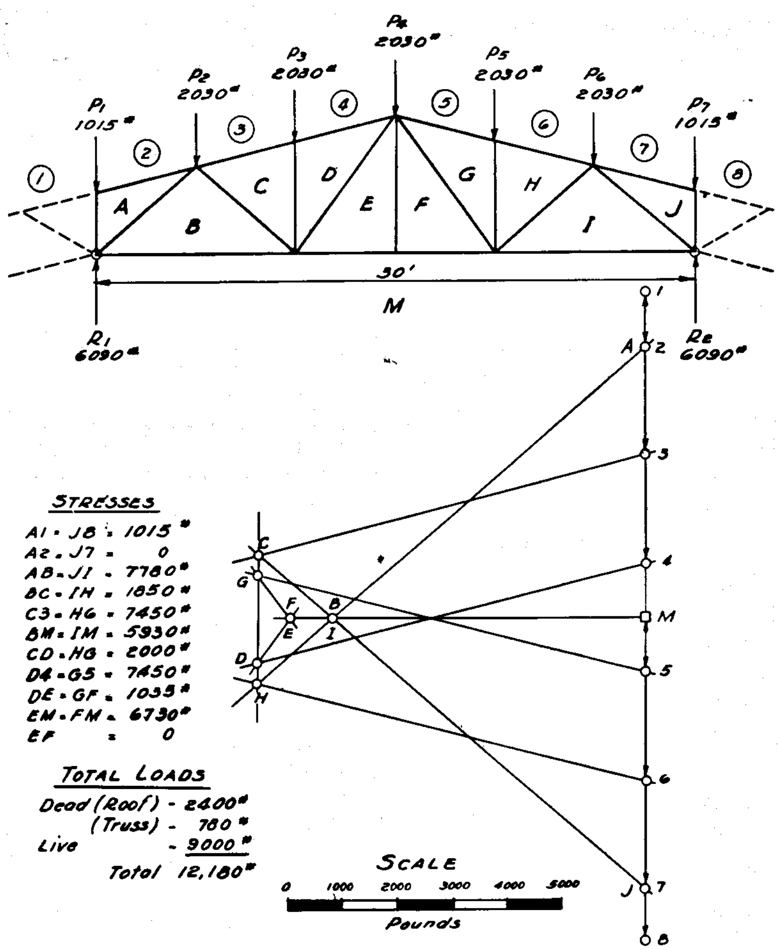


Figure 14

Because rafters carry a portion of roof load, they must be adequate for the combination of bending with direct compression. Rafters adjacent to pole tops do not carry a full panel roof-load, (See Figure 4), as was computed for intermediate rafters. The panel width here is 1 ft-6¾ in instead of 2 ft-4½ in. Figuring the panel load in the manner shown on page 15, using a heavier rafter and shorter panel width, the panel loads are:

Dead load	—	119.8 lb (3 × 8 rafter)
Live load	—	468.0 lb
Wind load	—	210.8 lb
Total load, D + L + W		798.6 lb

Wind loads must be considered in this case because the total of dead plus live plus wind loads is more than 1⅓ times that for dead plus live loads alone. The bending moment in the rafter is,

$$M = 798.6 \times \tfrac{3}{4} \times 14.25 \times 12 \times \tfrac{1}{8}$$
$$= 12{,}804 \text{ in-lb}$$

For the secondary moment, (See Figure 5), the component parallel to rafters amounts to 96.8 lb and the moment due to it is:

$$96.8 \times 7\tfrac{1}{2} \times \tfrac{1}{2} = 363 \text{ in-lb}$$

Total moment is 12,804 + 363 = 13,167 in-lb

$$s = 24.61 \text{ in}^3$$

$$f = \frac{13{,}167}{24.61} = 535 \text{ psi}$$

Stress Diagram— Wind Loads

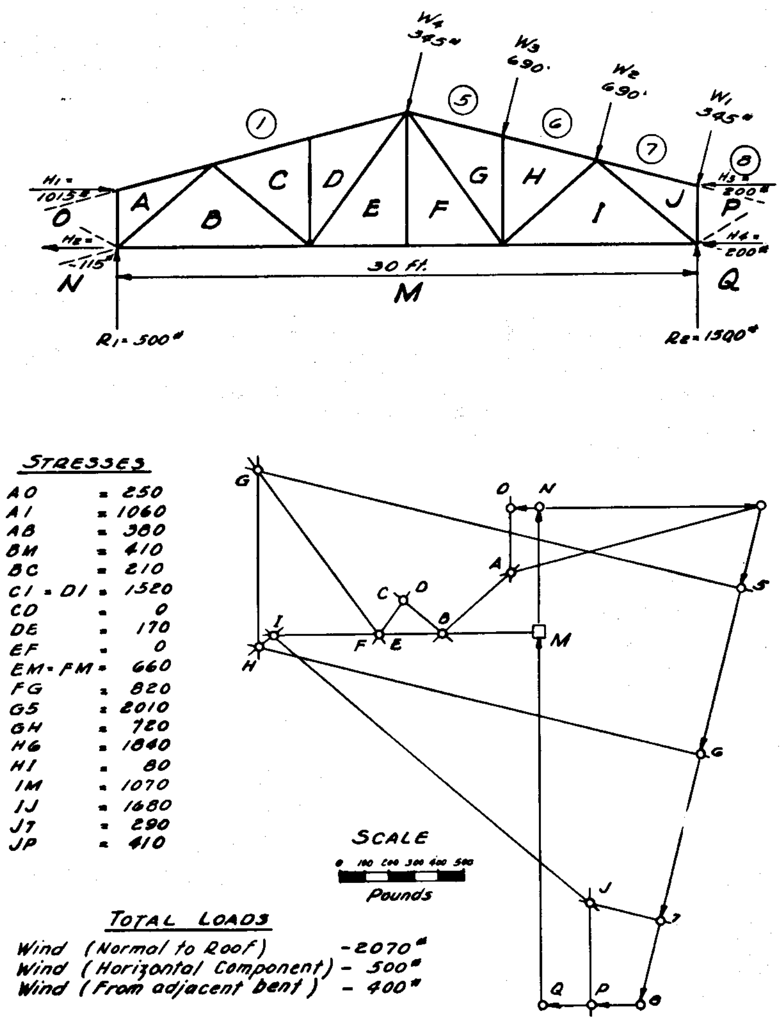


Figure 15

For rough 2×8's, dead load amounts to 103.8 lb, total load to 782.6 lb, and bending moment in the rafter is:

$$M = 782.6 \times 3/4 \times 14.25 \times 12 \times 1/8 = 12{,}548 \text{ in-lb}$$

The component parallel to rafters amounts to 95.0 lb, and the secondary moment is:

$$95.0 \times 7\tfrac{1}{2} \times \tfrac{1}{2} = 356 \text{ in-lb}$$

Total moment is 12,548 + 356 = 12,904 in-lb

$$s = 21.33 \text{ in}^3$$

$$f = \frac{12{,}904}{21.33} = 606 \text{ psi}$$

For combined flexure and compression, $\frac{M/S}{f} + \frac{P/A}{c}$ must not be greater than one, where:

$\frac{M}{s}$ = Actual stress in bending, extreme fiber.

f = Allowable stress in bending, extreme fiber.

$\frac{P}{A}$ = Unit direct stress induced by the axial load.

c = Allowable stress in compression parallel to grain.

Here, M/s divided by f amounts to 535/1200 = 0.45 or to 606/1200 = 0.51, which leaves 0.55 or 0.49 respectively, for the fraction P/A divided by c.

Total allowable load in compression on the two rafters is 16,460 lb. This load multiplied by 0.55, or 7800 by 0.49, gives the maximum allowable load in compression in combination with bending stresses from the roof load. The result will be 9055 lb or 3820 lb, depending on whether 3×8 dressed or 2×8 rough lumber is used for rafters.

Substituting these values and checking the second of these, the 2×8 rough:

$$\frac{P/A}{c} = \frac{\dfrac{3{,}820}{2 \times 16.00}}{211 \times \dfrac{2500}{2162}} = \frac{119.3}{244.0} = 0.49$$

The actual load is only 1685 lb. If the roof itself is securely fastened to these rafters, it will be found that dressed 2×8 lumber will be adequate.

Rafter Braces

Braces located to the left of a rafter's center are in tension, and a pair of 2×4 pieces will suffice. Braces located to the right of the mid-point are in compression. Because of their unsupported length of 8 ft-2 in, it is necessary to use material at least 2-in thick, either 3×6 dressed, or 2×6 rough. Actual unit loads are nominal for such pieces at this point. Their thickness is determined by limiting value of the slenderness ratio, L/D.

Dressed 2×6 lumber can be used for these braces if a piece of 2-in-thick material 3 to 4-ft long is securely spiked to upper edges of each pair of braces along their mid-length.

Members are arranged in pairs and should be bolted. Calculations for the required bolts to transmit stresses to poles and rafters are similar to those previously given for roof framing. Braces in compression should be kept on the inside, next to rafters. Tensile braces should be outside, since eccentricity in them is not so serious as in compression members. Similar bracing may be used in rows of poles, longitudinally of the building, to fix pole tops in that direction and to distribute wind loads from ends of the structure.

Ties may be provided between poles at lower ends of knee braces to take some of the tension required at these points to fix their tops in a vertical direction, as shown by dotted lines in Figures 10, 11 and 12. These bolted ties also should be arranged in pairs, and may be 2×4, or 2×6 material. When these ties are used, depth of the bracing can be reduced. Shorter diagonal bracing, arranged as two or three triangles (shown in the lower diagram of Figures 10, 11 and 12), can replace the single triangle shown for deeper bracing. With shallower bracing, diagonals may be nominal sizes, 2×4's or 2×6's. Lengths should be kept within the range of 2-in dressed lumber in compression, not to exceed 6 ft-9 in unsupported lengths, for a maximum slenderness ratio of 50.

Roof Trusses

For wide central bays, as Type C, Figure 12, roof trusses usually must be provided for the spans. Framing a roof over trusses will differ from that shown in Figure 4, and used for narrower panels of Types A and B, and the side spans of Type C, Figures 10, 11 and 12.

Purlins on top of trusses at panel points support rafters and give lateral support to the top compression cord of the trusses. Estimating the panel load as before, with the same spacing of rafters, and applying it to the shorter span between roof purlins, reveals that 2×4's are strong enough for the rafters. Then, computing the panel load supported by a purlin, the required section modulus is found to be approximately 36 in^3. The computations are shown, Figure 16. Because these purlins, resting on the top chord of panel points of trusses, are not vertical, their section moduli in an inclined position, with loads acting vertically downwards, must be found. Figure 16 shows a graphical method for obtaining the section modulus of rectangular sections subject to unsymmetrical bending. The same results may be readily obtained algebraically.

Analysis of Purlins on Trusses

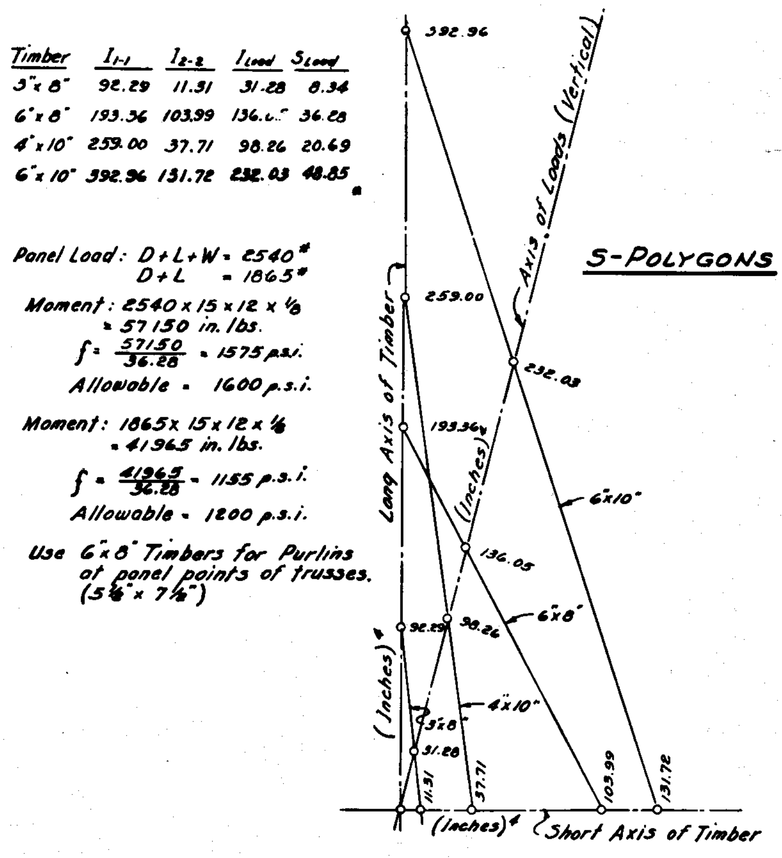


Figure 16

Size of the beam that will support rafters over the truss has been determined. Loads to be applied to it at the panel points are next obtained as shown previously, (Figure 4), for panel load determinations in connection with roof framing. Panel load on the truss:

Roof Framing Panel Loads

Dead load	
2 × 4 purlins	64 lb
26-gauge sheets	67 lb
2 × 4 rafters	77 lb
6 × 8 beam	192 lb
Total from roof	400 lb
Truss (assumed)	130 lb
Total Dead Load	530 lb
Live load (20 psf)	1500 lb
Total Dead plus Live load per panel	2030 lb
Wind load (9.2 psf, normal to roof)	690 lb
Wind load (horizontal force from adjacent panel)	400 lb

Figure 14 shows dead and live loads applied to trusses, at panel points. It gives a graphical determination of stresses in various members of the truss from the load diagram, and a tabulation of truss members, with their respective stresses. Figure 15 shows a similar application of loads, also a graphical determination and tabulation of stresses due to wind forces on the truss. A table can be prepared to show stresses due to dead plus live loads, and to dead plus live plus wind loads. It would determine which condition of loading should govern the design of each individual member. In this case, stresses due to all loads, including the wind, have been multiplied by the factor 3/4. This allows for a one-third increase in working stresses when designing members where wind forces have been included in finding total stress. The figures provide a direct comparison between total stress in a member caused by dead plus live loads, and total stress in the same member caused by dead plus live plus wind loads. The larger of the two should be used and allowable unit stresses chosen for the particular species and grade of lumber selected.

In this case, wind loads affect only certain web members and end panels of top chord members. This might have been foreseen because wind loads, applicable to just one-half the span of a truss, are only about one-third as large as the sum of dead and live loads at each panel point. Their effect on chord members must be relatively less than that of equal loads applied throughout the span. Caution should be observed, however, in making such generalizations. It is advisable to run through the analysis of wind loads in each instance, making direct comparisons of actual stresses obtained in members. Little work is involved and both stress diagrams can be made on the same layout. In regions where the assumed live load is less than that used here, or where the wind pressures are greater, or both, it will be found that stresses due to wind loads are large enough to require consideration in the design of all truss members.

Table of Stresses

Table A, on the following page, shows results obtained from the two stress diagrams. In practice it is necessary to tabulate figures for only one-half truss, unless its outline is unsymmetrical or there are special, unsymmetrically placed live loads. Large values obtained in the side of a truss away from the wind loading will be produced in opposite members on the other side by reversing direction of the wind.

From the table of stresses it would seem that only eight separate members need be considered. End verticals, A1 and J8, are, of course, replaced by tops of supporting poles. Top chord members, C3, D4, G5 and H6, are all the same. With a total stress of 7,450 lb in compression, and a length of 5 ft-2 in, it will be found that two 2×8's are good for 2 × 2570 lb at 6 ft-9 in. For a length of 5 ft-2 in the L/D ratio is 62 divided by 1⅝, or 38.2. The 2×8's, 5 ft-2 in long are, therefore, good for 2 × 2570 × 2500/1459, or 8800 lb. Similarly, the end post, AB or IJ, whose length is 6 ft-7 in requires two 3×6's, good for 2 × 3115 lb at 11 ft-0 in, or 2 × 3115 × 2500/906, or 17,200 lb at 6 ft-7 in, where the L/D ratio is 30.1. Top chord of panel A2 or J8, is stressed only through an unbalanced loading from the wind. This stress amount is 795 lb. These sections, 5 ft-2 in long, will be made up of two 2×4's, good for 2 × 1240 × 2500/1459, or 4250 lb where the L/D ratio for their

Table A **Stresses—From Diagrams, Figures 14-15, Pages 29 and 30**

MEMBER	D + LL	WL	D + L + WL	¾ TOTAL	USE
END POSTS					
AB	+7,780	+ 380	+8,160	+6,120	+7,780
IJ	+7,780	+1,680	+9,460	+7,095	+7,780
TOP CHORDS					
A2	C	+1,080	+1,060	+ 793	+ 795
C3	+7,450	+1,520	+8,970	+6,725	+7,450
D4	+7,450	+1,520	+8,970	+6,725	+7,450
G5	+7,450	+2,010	+9,460	+7,095	+7,450
H6	+7,450	+1,840	+9,290	+6,965	+7,450
J7	0	+ 290	+ 290	+ 220	+ 220
BOTTOM CHORDS					
BM	−5,930	− 410	−6,340	−4,755	−5,930
EM	−6,730	− 660	−7,390	−5,540	−6,730
FM	−6,730	− 660	−7,390	−5,540	−6,730
IM	−5,930	−1,070	−7,000	−5,250	−5,930
DIAGONALS					
BC	−1,850	− 210	−2,060	−1,545	−1,850
DE	+1,035	+ 170	+1,205	+ 905	+1,035
FG	+1,035	+ 820	+1,855	+1,390	+1,390
HI	−1,850	− 80	−1,930	−1,445	−1,850
VERTICALS					
A1	+1,015	+ 250	+1,265	+ 950	+1,015
CD	+2,030	0	+2,030	+1,520	+2,030
EF	0	0	0	0	0
GH	+2,030	+ 720	+2,750	+2,060	+2,060
J8	+1,015	+ 410	+1,445	+1,085	+1,080

lengths is 38.2. The central diagonal, DE or FG, which is one of the web members deriving its maximum stress condition under wind loading, is 8 ft-5 in long, with a stress of 1380 lb. It will consist of one 3x6, good for 3115 x 2500/1482, or 5300 lb. Its L/D ratio is 38.5. Similarly, the verticals, CD or GH, 5 ft-6 in long, with a maximum stress of 2040 lb, also due to unbalanced loading from the wind, may be composed of two 2x4's, good for 2670 lb.

Remaining members, bottom chords BM, EM, FM and IM, and diagonals BC and HI, are in tension. Their stresses are nominal. Bottom chords probably should be two 2x6's, to provide areas necessary for connections. For diagonals BC and HI a single 2 x 4 will suffice.

Joint connections of this truss can be bolted. The top chord joint between D4 and G5, and that between end post AB and top chord C3, should be cut to bear, not to transfer stress, but to insure good alignment of these compression members. Stresses at these joints should be carried by bolted splice plates. Web members should be framed inside the chord members and filler blocks used to pack each joint tightly between chord parts. Purlins should be fastened rigidly to top chords at panel points to assure unsupported lengths of top chord members that were assumed in their design. Longitudinal sway bracing between trusses should be provided in about every third bay lengthwise of the structure.

If trusses are used to span a much wider central bay, or to support heavier dead and live loads and greater horizontal wind forces, it undoubtedly will be necessary to use some type of timber connectors for joint connections. It is not within the scope of this publication to go into the design of such connections, but detailed information is readily available.* Basic analysis will be the same as has been shown here, and applications to pole-type

* Timber Engineering Company, 1619 Massachusetts Avenue, Washington, D. C. 20036

frames will be similar to those assumed examples. Possible variations in frames based on the three simple types shown in Figures 10, 11 and 12, are without limit but the underlying principles governing their design are the same for all.

Uplift Resistance

In designing any roof for a pole-type building it is well to take advantage of the inherent value of poles to resist uplift by giving roof framing throughout the structure a positive connection to tops of poles. With the roof securely fastened down, embedded poles will safely withstand a very large uplift force. For further information on this subject see, "How to Prevent Storm Damage to Farm Buildings," Fact Sheet 86, published by the University of Maryland, College Park, Md. and "Houses Can Resist Hurricanes," U.S. Forest Service Research Paper FPL 33, Forest Products Laboratory, Madison, Wisconsin. The Factory Mutuals' service bureau has extensive experience with hurricane damage along the Atlantic coast, particularly in New England. Its bulletins on protection against wind damage to mill, factory and warehouse structures, stress the importance of adequate anchorage of roofs against uplift. In pole-type buildings, a very large resistance against uplift is already available at the roof level. It should be credited where such forces are to be expected.

RECOMMENDED MINIMUM DESIGN WIND PRESSURES
(Pounds per Square Foot)
(Based upon data obtained from U.S. WEATHER BUREAU and AMERICAN STANDARD BUILDING REQUIREMENTS A58.1-1945)

Outdoor Advertising Association of America, Inc.

Treating Specifications

The specifications which are universally recognized and quoted as the standards for the pressure treatment of wood poles and lumber are those of the American Wood-Preservers' Association and those included in Federal Specification TT-W-571; Wood Preservative, Recommended Treating Practice. The latter is based on the AWPA Book of Standards. The net retentions of preservative may be varied somewhat to provide for different conditions of severity. However, the net retention should never be less than those stipulated in AWPA standards. A guide providing recommended specifications for building products pressure treated with proven preservatives is included in Table I, page 38.

AWPA Standards for Pressure Treatment

AWPA Standard C1 is basic for all timber products and is used in conjunction with the standard for a particular item of material. The pressure treatment of lumber, for example, should comply with Standards C1 and C2, the treatment of poles with C1 and C4. Because C1 is designated as a part of the other AWPA Commodity Standards, it need not be designated when preparing a specification for a given product. The specification for the treatment of construction poles, for example, need only designate that poles shall be treated in accordance with AWPA Standard C4 with a minimum retention of ten pounds of creosote per cubic foot or .50 pounds of pentachlorophenol per cubic foot.

Lumber and Plywood Quality Control Standards

To assure high standards of quality and to facilitate the specifying and procurement of preservative treated materials the American Wood Preservers Institute, in conjunction with the U. S. Forest Products Laboratory and the Federal Housing Administration, has developed Quality Control Standards that can be adopted by reference. These Standards require that the products be subjected to the prescribed system of control, both from the producer and the control agency, such as the American Wood Preservers Bureau, who check product quality at the plant and at destination. These products are identified by the AWPI Quality Mark. Products produced under this system of control are described in the following listed standards:

Standard AWPI LP–2	American Wood Preservers Institute Standard for Softwood Lumber and Plywood Pressure Treated with Water-Borne Preservatives for Above-Ground Use.
Standard AWPI LP–3	American Wood Preservers Institute Standard for Softwood Lumber and Plywood Pressure Treated with Light Petroleum Solvent-Penta Solution.
Standard AWPI LP–4	American Wood Preservers Institute Standard for Softwood Lumber and Plywood Pressure Treated with Volatile Petroleum Solvent (LPG)—Penta Solution.
Standard AWPI LP–22	American Wood Preservers Institute Standard for Softwood Lumber and Plywood Pressure Treated with Water-Borne Preservatives for Ground Contact Use.

Although all of these standards are used in pole building construction, cleanliness, paintability, color and odor will effect the selection of the type of preservative for a particular project. The various preservatives and the corresponding AWPI Standard, where one occurs, are discussed below.

Water-Borne Salt Preservatives

These preservatives provide clean, odorless, paintable, non-irritating pressure treated lumber for all types of construction. Specify AWPI LP–2 for above ground applications. For lumber to be placed in the ground and subject to leaching, AWPI LP–22 should be specified.

Penta (Pentachlorophenol)

Highly toxic to fungi and insects . . . insoluble in water and thus permanent . . . most widely used oil-borne preservative. Penta dissolved in the light petroleum solvents described in AWPI LP–3 provides clean and colorless surfaces.

Gas-Borne Treatment

A relatively new development, Penta dissolved in liquid petroleum gas, as described in AWPI LP–4 provides a clean and paintable product which can be used where natural finishes are desired.

Penta in Oil

Penta dissolved in fuel oil can be used for lumber, fence posts and poles where color, odor, and the lack of paintability are not objectionable. See Table I, page 38 for recommended treatment.

Creosote

Used where protection against decay and attack by termites and other wood-destroying organisms is of primary importance; where painting is not required, and odor not objectionable. These chemical compounds are highly toxic to wood-destroying insects and marine organisms. See Table I for recommended treatments.

Sample Specifications

In order to obtain lumber products pressure treated in accordance with recognized industry standards and subjected to the rigid quality control procedures, the specifier should designate the species and grade of lumber required and that the lumber shall conform to AWPI LP–2, for example, and that it bear the AWPI Quality Mark. This mark, branded on the lumber, will indicate the AWPI Standard under which treated, the type of preservative, the month and year of treatment and identity of the treating company.

If the AWPI Quality Control Standards and Quality Mark are not utilized, the specifier should designate the species and grade of lumber required, the type of preservative desired and that the preservative treatment be performed in accordance with AWPA Standard C–2 to the net retention designated in Table I. The following is a typical specification for preservative treated lumber: the lumber shall be Southern pine or Douglas fir Construction grade, S4S, pressure treated to a net retention of .40 pounds of pentachlorophenol per cubic foot in accordance with AWPA Standard C–2.

Poles are subject to different grading requirements than sawn timber. Since the specification universally used, USASI 05.1—1963, was prepared basically for utility poles rather than construction poles some modifications may be desired, particularly to achieve greater pole straightness. A typical specification for construction poles follows: all poles shall be (insert species desired), class 6, 20 feet long, conforming to USA Standard 05.1 with the additional provision that a straight line drawn from a top edge to a bottom edge of the pole shall at no point along the pole fall more than one inch for every ten feet of the pole length from the pole surface and sweep shall be in one direction and one plane only. The poles shall be preservatively treated to a net retention of ten pounds of creosote per cubic foot in accordance with AWPA Standard C–4.

TABLE I—RECOMMENDED SPECIFICATIONS

Preservative	Coal-tar Creosote	Pentachlor-ophenol (1)	ACA (2)	CCA Type A (3)	CCA Type B (4)	FCAP (5)	ACC (6)	CZC (7)	TREATMENT Federal Specification TT-W-571
Preservative Fed. Spec.	TT-C-645	TT-W-570	TT-W-549	TT-W-550	TT-W-550	TT-W-535	TT-W-546	TT-W-531	
Preservative AWPA Spec.	P-1	P-8	P-5	P-5	P-5	P-5	P-5	P-5	
Minimum Net Retentions In Pounds Per Cubic Foot (By Assay)									
Product:									AWPA Standard
Lumber & Plywood, no ground or water contact	8	0.40	0.30	0.35	0.25	0.35	0.50	0.75	C-2 (Lumber) C-9 (Plywood)
Lumber & Plywood in ground or fresh water and all important structural timbers	8	0.40	0.50	0.75	0.42	NR (8) 0.50	NR (8) 1.00	NR (8) 1.00	C-2 (Lumber) C-9 (Plywood) C-28 (Laminated)
Poles	10	0.50	0.60	1.00	0.60	—	—	—	C-4
Posts (fence)	6	0.30	0.50	0.75	0.42	0.50	1.00	1.00	C-5
Fire Retardant Treatment by Performance:	Material shall have no greater flame spread than 25 when tested in accordance with ASTM E84 and in test extended to 30 minutes duration it shall have no greater flame spread than equivalent of 25 and no evidence of significant progressive combustion. All lumber 2 inches and less in thickness shall be dried to an average moisture content of 19% and plywood to 15% after treatment. All material shall bear the UL label with the letters FR-S.								C-20B (Lumber) C-27B (Plywood)

NOTES

(1) Pentachlorophenol may be prepared as a solution of heavy petroleum oil (AWPA Spec. P-9); or as a solution of mineral spirits and waxes (Fed. Spec. TT-W-572), known as Water Repellant Penta, or Penta W-R; or as a solution of volatile petroleum solvent (AWPA Spec. P-9), trade name: Cellon. Recommended use other than as a solution of heavy petroleum oil is limited to conditions requiring cleanliness or paintability.

(2) Ammoniacal Copper Arsenite (Trade name: Chemonite)

(3) Chromated Copper Arsenate, Type A (Trade name: Erdalith-Green salt)

(4) Chromated Copper Arsenate, Type B (Trade name: Boliden K-33)

(5) Fluor-Chrome Arsenate Phenol, Type A (Trade name: Tanalith-Wolman Salts)
Fluor-Chrome Arsenate Phenol, Type B (Trade name: Osmosalts)

(6) Acid Copper Chromate (Trade name: Celcure)

(7) Chromated Zinc Chloride

(8) Occasional exposure to rain or constant exposure to ground in arid regions.

General: All fabrication, cutting, boring, etc. possible should be performed before treatment. After treatment, pressure treated wood products should be cared for in accordance with AWPA Spec. M-4.

Recommended penetrations vary with the species. See AWPA Specifications. The purpose of this summary it to provide a ready reference to minimum recommended preservative retention levels and to provide an index to detailed treatment specifications.

Colors: Creosote: black; pentachlorophenol in heavy oil: brown to black; Penta W-R and Cellon: essentially clear and paintable; water borne salts and fire retardants: shades of green and brown and paintable.

APPENDIX

Correct Embedment for Pole Structures

By Edwin E. Kinney
Chief Engineer, Engineering & Research Div.
Outdoor Advertising Association of America, Inc.

While members of the Outdoor Advertising Assn. of America, Inc. have made use of the Rutledge Chart for the design of cantilever supported outdoor advertising displays for the past 20 years, it is more recently that others have discovered the usefulness of this nomograph for the design of guardrail supports, airplane nose hangars, pole-type buildings, and other cantilever structures. The nomograph is helpful in designing any structure requiring the utilization of the lateral support of the soil.

Previous to the advertising industry's adoption of the cantilever method, most outdoor advertising displays were supported on truss type A-frame supporting systems. A basic difference between these two methods of support is illustrated by a utility company's use of ground anchors to brace poles, as opposed to the cantilever embedment of the poles themselves. In the first case the bearing value of the soil provides the key to the design of the anchor. In the second case the lateral soil resistance and the depth to which the pole is placed in the ground is the controlling factor in the stability of the poles.

Prior to about 1938 one of the few pieces of literature which shed light on the design of a cantilever embedded upright was the J. F. Seiler (formerly field engineer, American Wood Preservers Institute) pamphlet published by *Wood Preserving News* in November 1932.

In 1938, the OAAA became interested in learning more about the safety and use of this type of construction, and engaged Dr. P. C. Rutledge, professor of civil engineering at Purdue University, to investigate the stability of anchor type foundations and also to investigate and make laboratory and field tests of the lateral stability of cantilever uprights. This work resulted in the production of a chart for the determination of maximum anchor resistance, dated April 15, 1940.

It was not until further work was completed in early 1947 at Notre Dame University, with Dr. Rutledge as consultant, that a nomographic chart pertaining to the design of cantilever uprights was made available to the OAAA (Figure 1, page 6). The report on which the nomograph was first based is entitled "A Report of Field and Laboratory Tests on the Stability of Posts Against Lateral Loads" by Walter L. Shilts, Leroy Graves, and George Driscoll, University of Notre Dame. This report was reprinted as a part of the proceedings of the Second International Conference of Soil Mechanics and Foundation Engineering, Rotterdam, 1948.

The first use of the nomograph in actual design of advertising display structure uprights was made shortly after the development of the nomograph by Dr. Rutledge in December 1947.

At the present time, 22 years since the design of the chart, over 80 percent of the new structures put up by members of the OAAA make use of the cantilever principle. The chart is designed to limit movement of the upright at the groundline to not more than one-half inch under full design load, and makes use of a predicted allowable average soil stress both above and below the point of rotation of the upright, based upon field tests in a range of sandy and gravelly soils as well as a full range of silts and clays. Actual failure of a cantilever foundation occurs at loads upwards of five times the design load because of the limiting value of the half-inch groundline deflection used in the nomograph.

To further extend the usefulness of the Rutledge Chart, a series of full-scale field tests was undertaken by the OAAA in a clay soil of excellent strength in Lima, Ohio, in 1950. A similar series at a location near Fort Wayne, Ind., where a lacustrine and clay soil provided a weaker test material, was completed later the same year. Both tests supported and extended the information presented in the 1947 Rutledge nomographic design chart. Dr. Rutledge, then chairman of the Civil Engineering Dept. at Northwestern University, Evanston, Ill., acted as consultant to the OAAA and directed the field tests and the analysis of strain gage and deflection data. This work resulted in a series of maximum soil stress, deflection, and average soil stress computations, and graphs of the individual tests at varying embedments. These data, which are as yet unpublished, are available and in use by the association.

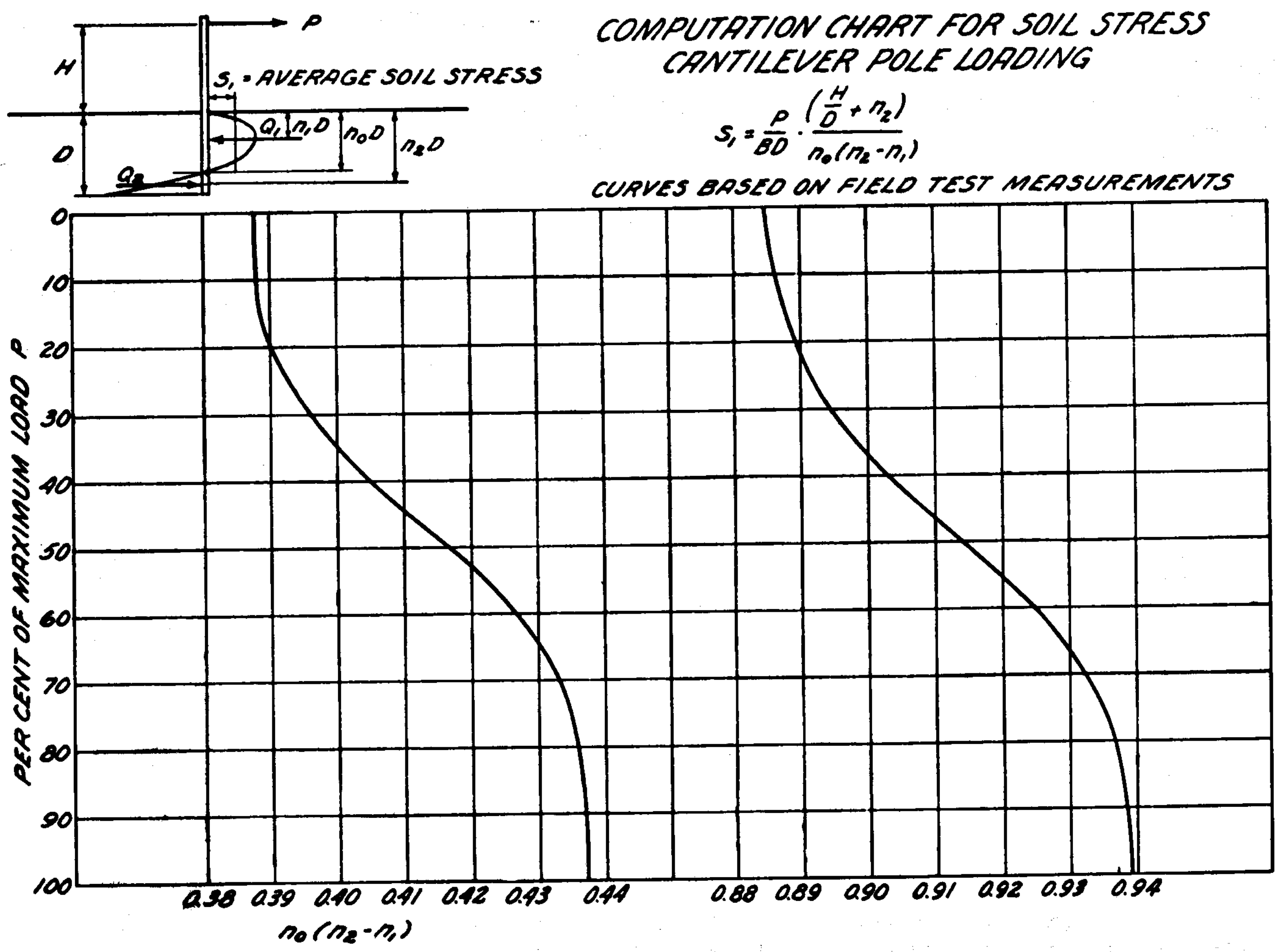
To use the Rutledge Chart an estimate is made of the soil type, either according to the following classification or by an auger indicator test which has been correlated to the allowable average soil stress in pounds per square foot. Soils will vary in their classification during the year due to changes in moisture content. The worst condition should be estimated. The following are definitions of soil within the categories of Good, Average, and Poor.

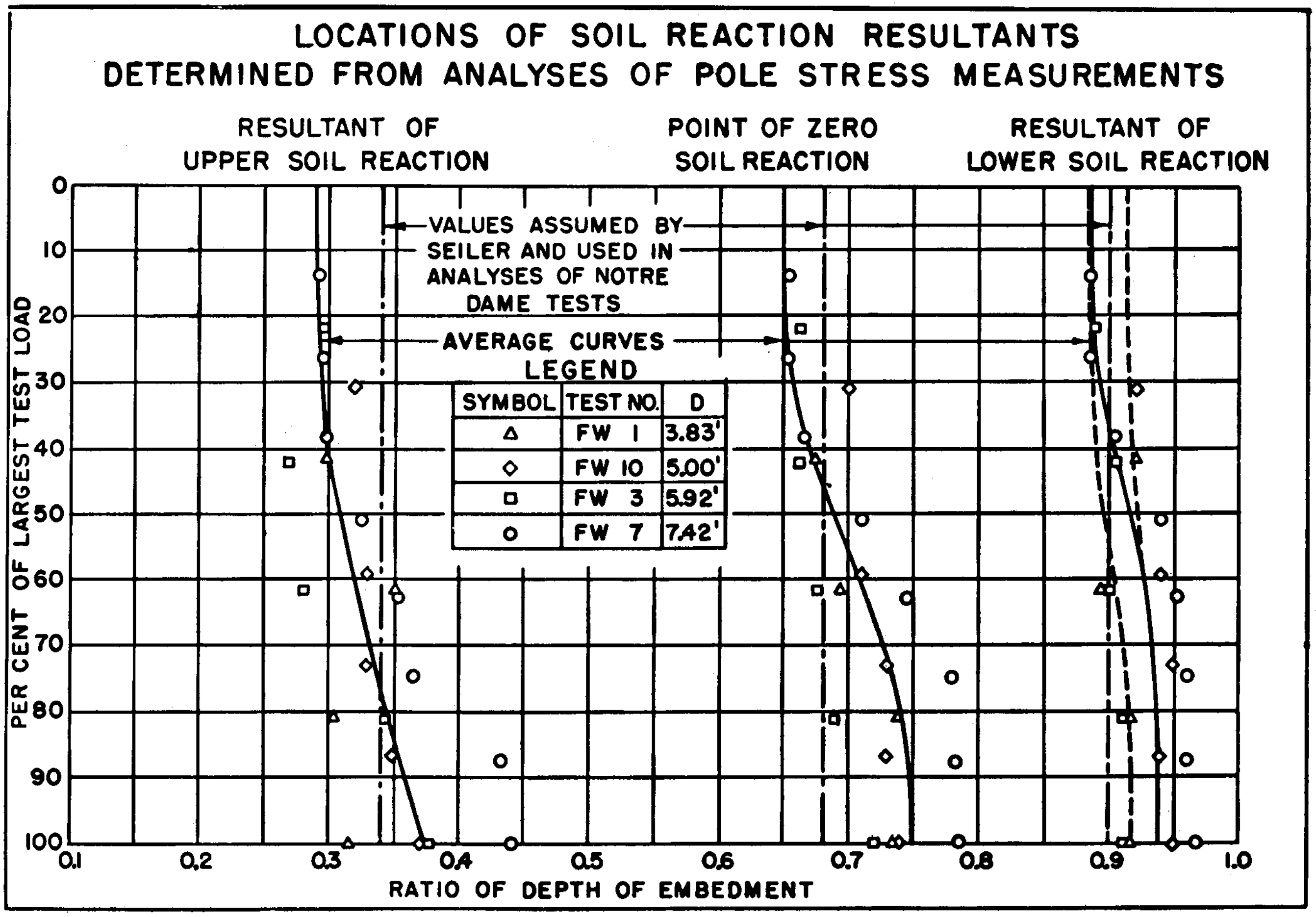
Good—Compact well-graded sand and gravel, hard clay, well-graded fine and coarse sand, decomposed granite rock and soil. Good soil should be well-drained and in locations where water will not stand. Allowable average soil stress—4,000 lb per sq ft.

Average—Compact fine sand, medium clay, compact well-drained sandy loam, loose coarse sand and gravel, and medium clay. Average soils should drain sufficiently well so that water does not stand on the surface. Allowable average soil stress—2,500 lb per sq ft.

Poor—Soft clay, clay loam, poorly compacted sand, clays containing a large amount of silt and vegetable matter. These soils will hold moisture and absorb great quantities of moisture when wet. Usually, soils of this type are found in low-lying areas and in areas where water stands during the wet season. Allowable average soil stress—1,500 lb per sq ft.

The indicator test is performed by turning a 1½-in indicator auger into the ground at the selected site to a depth of 1 ft and measuring the pull in pounds required to return it 6 in. This is repeated at 1-ft intervals until a profile of the soil at 1-ft intervals to the required depth has been estab-





lished. The values obtained can then be related to the Rutledge nomograph according to the scale at the extreme left hand edge of the chart.

As an example of the use of the chart, assume a post load of 3,000 lb. at a height of 20 ft. The soil is a clay type with an allowable average soil stress of 2,500 lb per sq ft. The upright is assumed to be 14 in. in diameter.

The required embedment for these conditions is determined by entering the nomograph when $S_1 = 2{,}500$ lb per sq ft and drawing a line from this point through the post load point of 3,000 lb to intersect a value at C. From the intersected value of C a line is drawn through a point corresponding to the width, or diameter of the upright to determine the depth coefficient L. From L (equal to 1.0 in this example), a line is drawn horizontally across the chart to meet the load-height curve of 20 ft. A line is extended vertically down to intersect the value of the required depth of embedment at D which is 8.5 ft. For these conditions a groundline motion of not more than one-half inch will exist under full post load of 3,000 lb. In actual practice and with the experience which comes with repeated use of the chart, soil types and the corresponding allowable soil stress values can be estimated quite closely without repeating the indicator auger test at each site.

The experience of association members who have made use of the Rutledge Chart has been most favorable. Previously unusable locations have lent themselves to the use of cantilever upright construction where an extensive trussed structure would have occupied too much space. Advertising structures are generally being placed higher for visibility and clearance over cars and other traffic. Cleaner and much less complicated advertising structures result when fabricated according to the cantilever principles and the foundation is designed according to the Rutledge formula. Less maintenance is involved in maintaining a cantilever foundation than a foundation involving anchors which may "pump" under load and usually require frequent attention after windstorms and rainy periods. Safety is a consideration as well, for while a broken or loose anchor can be overlooked, cantilever uprights are of such size and simplicity as to make apparent for immediate repair any unreasonable movement due to poor ground drainage, wind, or other damage.

In addition to the Rutledge nomographic chart for solving the depth of embedment of a cantilever upright, two previously unpublished graphs are also reproduced here.

The first is a chart for computing the average soil stress under a given percentage of the maximum cantilever pole load.

The second chart shows the location of the upper soil reaction resultant, the point of zero soil reaction, and the location of the lower soil reaction, shown as a percentage of the maximum load corresponding to the overturning load on the upright. These curves are based upon field test measurements. From these charts an average soil stress can be computed for a given set of height, post width, and load conditions for an assumed percentage of maximum load. The point of rotation of the upright for these conditions can also be determined.

Modifications of the cantilever method have also been found useful in extending the range of loads which can be supported within limited upright deflection. In some cases soil cement or concrete has been used to extend the effective diameter of the embedded portion of the upright. Walers 6 to 8 ft long, placed horizontally across the face and back of the upright just below groundline will substantially raise the point of rotation of the cantilever in the ground. This results in a lower angular movement and groundline deflection of the upright for a given load. These methods are particularly effective where weak soils would otherwise require excessive initial embedments or continued maintenance to correct for small motions of the upright.

International Conference of Building Officials Approved Procedures

Design Criteria: The following formula is used in determining required embedment depth where no constraint is provided at the ground surface, such as rigid floor or ground surface pavement:

$$d = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right) \qquad (1)$$

Where $A = \dfrac{2.34P}{S_1 \, b_1}$

P = Applied horizontal force, in lb.

S_1 = Allowable lateral soil-bearing pressure as set forth in Table II based on one-third the depth of embedment.

b_1 = Diameter of round post or diagonal dimension of square post, in ft.

h = Distance, in ft from ground surface to point of application of P.

d = Depth of embedment of post, in ft.

Minimum embedment shall be 4 ft into natural soils or compacted fill.

TABLE II—ALLOWABLE LATERAL SOIL PRESSURE

Class of Material	Allowable Values per Foot of Depth Below Natural Grade[1] (Pounds per Square Foot)	Maximum Allowable Values (Pounds per Square Foot)
Good—compact well-graded sand and gravel Hard Clay Well-graded fine and coarse sand (All drained so water will not stand)	400	8000
Average—Compact Fine Sand Medium Clay Compact sandy loam Loose Coarse sand and gravel (All drained so water will not stand)	200	2500
Poor—Soft Clay Clay Loam Poorly compacted sand Clays containing large amounts of silt (Water stands during wet season)	100	1500

[1] Isolated poles, such as flagpoles, or signs, may be designed using lateral bearing values equal to two times tabulated values.

Construction Requirements: The backfill in the annular space around posts shall be by one of the following methods:

1. Backfill shall be of concrete with an ultimate strength of 2,000 lb per sq in at 28 days. The hole shall be not less than 4 in larger than the diameter of the round post at its bottom, or 4 in larger than the diagonal dimension of a square or rectangular post. The dimensions b_1 and b_2 in Equations 1 and 2 respectively, shall be the outside diameter of the concrete casing.
2. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 in thick.

The following formula is used to determine embedment depth where constraint is provided at the ground surface, such as a rigid floor, or a rigid ground surface pavement:

$$d^2 = 4.25 \frac{Ph}{S_3 \, b_2} \qquad (2)$$

Where p = Applied horizontal force, in lb.

S_3 = Allowable lateral soil-bearing pressure as set forth in Table II based on a depth equal to the total depth of embedment.

b_2 = Diameter of round post, or diagonal dimension of square post, in ft.

h = Distance, in ft, from ground surface to point of application of P.

d = Depth of embedment of post, in ft.

Approximating Allowable Unit Soil Values

Table III

Shown here is a list of fourteen soils, taken from the United States Steel Sheet Piling Handbook and based on the Coulomb-Rankine theories. Allowable unit pressures of these soils, when plotted in comparison with values given on the Rutledge chart for embedment, and making a reasonable allowance for the ultra-conservative passive earth pressures obtained from the Coulomb theory, will give a rough estimate of "s" to be used in the chart for each. In the following table they are given with their approximate position, lower third, middle third or upper third, of the respective portion of the soil classification given in the Rutledge chart.

Clay, in lumps, dry	Poor soil	Upper third
Clay, damp, plastic	Poor soil	Lower third
Clay and gravel, dry	Average soil	Upper third
Clay, gravel and sand, dry	Average soil	Upper third
Earth,loose, perfectly dry	Average soil	Lower third
Earth, packed, perfectly dry	Average soil	Upper third
Earth, loose, slightly moist	Average soil	Middle third
Earth, packed, more moist	Very hard soil	Middle third
Earth, soft flowing mud	Very soft soil	Upper third
Earth, soft mud, packed	Poor soil	Lower third
Gravel, one inch and under, dry	Good soil	Lower third
Gravel, two and one-half inches and under, dry	Average soil	Upper third
Sand, clean and dry	Average soil	Lower third
Sand, river, dry	Average soil	Middle third

Classifying the soil from a visual inspection and entering the Rutledge chart from the approximate position indicated in the above table may furnish fairly reliable results for depth of embedment, lacking more positive information on soil bearing values.

Properties of Species Approved for Poles by USA

Table IV

The species listed in USA Table 6 (Table VI of Appendix) supply slightly more than 90 percent of the poles produced and treated in the United States. The USA Standards Institute, however, has approved 8 other species for poles and published dimension tables and ultimate fiber stresses for them.

The moduli of rupture, strengths in extreme fiber in bending, which currently are assigned by the USA to nine of the species approved for poles after treatment are as follows (in pounds per square inch.):

Western larch	8,400 psi
Southern yellow pine	8,000 psi
Pacific Coast Douglas fir	8,000 psi
Lodgepole pine	6,600 psi
Jack pine	6,600 psi
Red, or Norway, pine	6,600 psi
Ponderosa pine	6,000 psi
Western red cedar	6,000 psi
Northern white cedar	4,000 psi

According to the U. S. Dept. of Agriculture Wood Handbook of 1955, page 156, the recognized values for moduli of elasticity for the usual pole species are as follows:

Values of Moduli of Elasticity

Southern yellow pine	1,600,000
Douglas fir	1,600,000
Western larch	1,600,000
Red or Norway, pine	1,200,000
Jack pine	1,100,000
Lodgepole pine	1,000,000
Ponderosa pine	1,000,000
Western red cedar	1,000,000
Northern white cedar	800,000

The American Wood-Preservers' Association has prepared pressure treating specifications for 9 of the more important species for use as poles.

These include Southern yellow pine, which is the dominant tree of the South; Douglas fir, Western larch, Western red cedar, and Ponderosa pine in the Northwest; Lodgepole pine in the Rocky Mountain area; and Jack and Red pine in the North. All of these can be pressure treated by plants located within the regions of their growth or can be processed in transit by other plants between their points of origin and their markets.

If any of these species not listed in Table VI are used, their strengths may be approximated by assuming that their working stresses are in the same ratio as the ultimate fiber stresses assigned by the USA.

Table V Properties of Timber Beams

Nominal Size Inches	Actual Size Inches	Area of Section Square Inches	Moment of Inertia $\frac{bd^3}{12}$ In.^{4}	Section Modulus $\frac{bd^2}{6}$ In.^{3}
2 × 4	1⅝ × 3⅝	5.89	6.45	3.56
2 × 6	1⅝ × 5⅝	9.14	24.10	8.57
2 × 8	1⅝ × 7½	12.19	57.13	15.23
2 × 10	1⅝ × 9½	15.44	116.10	24.44
2 × 12	1⅝ × 11½	18.69	205.95	35.82
3 × 6	2⅝ × 5⅝	14.77	38.93	13.84
3 × 8	2⅝ × 7½	19.69	92.29	24.61
3 × 10	2⅝ × 9½	24.94	187.55	39.48
3 × 12	2⅝ × 11½	30.19	332.69	57.86
4 × 6	3⅝ × 5⅝	20.39	53.76	19.12
4 × 8	3⅝ × 7½	27.19	127.44	33.98
4 × 10	3⅝ × 9½	34.44	259.00	54.53
4 × 12	3⅝ × 11½	41.69	459.43	79.90
4 × 2	3⅝ × 1⅝	5.89	1.30	1.60

(For 2" × 4" purlins, laid flat)

Table VI — Dimensions of Douglas Fir (Both Types) and Southern Pine Poles
(Based on a Fiber Stress of 8,000 psi)

Class		1	2	3	4	5	6	7	9	10
Minimum Circumference at Top (Inches)		27	25	23	21	19	17	15	15	12
Length of Pole (Feet)	*Groundline Distance from Butt (Feet)	Minimum Circumference at 6 Feet from Butt (Inches)								
20	4	31.0	29.0	27.0	25.0	23.0	**21.0**	**19.5**	**17.5**	**14.0**
25	5	33.5	31.5	29.5	27.5	25.5	**23.0**	**21.5**	**19.5**	**15.0**
30	5½	36.5	**34.0**	**32.0**	**29.5**	**27.5**	**25.0**	**23.5**	**20.5**	
35	6	39.0	**36.5**	**34.0**	**31.5**	**29.0**	**27.0**	**25.0**		
40	6	**41.0**	**38.5**	**36.0**	**33.5**	**31.0**	**28.5**	26.5		
45	6½	**43.0**	**40.5**	**37.5**	**35.0**	**32.5**	30.0	28.0		
50	7	**45.0**	**42.0**	**39.0**	36.5	34.0	31.5	29.0		
55	7½	**46.5**	**43.5**	**40.5**	38.0	35.0	32.5			
60	8	**48.0**	**45.0**	**42.0**	39.0	36.0	33.5			
65	8½	**49.5**	**46.5**	**43.5**	40.5	37.5				
70	9	**51.0**	**48.0**	**45.0**	41.5	38.5				
75	9½	**52.5**	**49.0**	**46.0**	43.0					
80	10	**54.0**	**50.5**	47.0	44.0					
85	10½	**55.0**	**51.5**	48.0						
90	11	**56.0**	**53.0**	49.0						
95	11	**57.0**	**54.0**	50.0						
100	11	**58.5**	**55.0**	51.0						
105	12	**59.5**	**56.0**	52.0						
110	12	**60.5**	**57.0**	53.0						
115	12	**61.5**	**58.0**							
120	12	**62.5**	**59.0**							
125	12	**63.5**	**59.5**							

* The figures in this column are intended for use only when a definition of groundline is necessary in order to apply requirements relating to scars, straightness, etc.

NOTE: Classes and lengths for which circumferences at 6 feet from the butt are listed in boldface type are the preferred standard sizes. Those shown in light type are included for engineering purposes only.

When lengths shorter than 20 feet are required the class specified will designate the minimum top circumference. The circumference at six feet from the butt may be estimated by adding to the top circumference 0.25 inches for each foot of length. As an illustration, Class 6 Douglas fir or Southern pine poles 16 feet in length should have a minimum circumference of 19.5 inches at 6 feet from the butt.

Strength of Bolted Wood Joints

Bolt-bearing stresses and other factors for calculating the strength of bolted timber joints, for the more common species of wood, are given in Tables VII, VIII and IX.

The allowable stresses, bolt-diameter factors and percentages of stresses tabulated are in accordance with values recommended by the U. S. Forest Products Laboratory in Wood Handbook.*

Allowable Bolt Loads Parallel to Grain

(Common Bolts)

To calculate the allowable load on a bolted timber joint when the load is acting parallel to the grain of the wood, and is applied at both ends of the bolt:

1. Select from Col. 1, Table VII, the stress S, in compression parallel to the grain for the species of wood to be used.

2. Calculate the ratio, $\frac{L}{D}$ of the length of bolt in main member to its diameter and, for this ratio, select the applicable stress percentage r from Table IX.

3. Multiply the stress S by percentage r. Their product is the safe working stress S_2, which is assumed to be uniformly distributed.

4. P, the safe working load for one bolt, is obtained by multiplying the safe working stress S_2 by the projected area of the bolt in the main member.

* Handbook No. 72, U. S. Department of Agriculture, Forest Products Laboratory, Forest Service.

Allowable Bolt Loads Perpendicular To Grain

(Common Bolts)

To calculate the allowable load on a bolted timber joint when the load is acting perpendicular to the grain and is applied at both ends of bolt:

1. Select from Col. 2, Table VII, the basic stress S_1 in compression perpendicular to the grain for the proper species of wood.

2. Calculate the ratio $\frac{L}{D}$ of the length of bolt in the main member to the bolt diameter. Select from Table IX the appropriate percentage r of basic stress for the $\frac{L}{D}$ value.

3. Select from Table VIII the diameter factor v for the bolt size to be used.

4. S_2, the allowable average unit stress $= S_1 \times r \times v$.

5. The allowable two end load P for a single bolt is the product of S_2 and the projected area of that portion of the bolt through the main member.

The horizontal plates that support the rafters in pole-type buildings are bolted to the pole columns. These plates, usually 2 in plank, bear on the bolts and are stressed in compression perpendicular to the grain. For computing the bolt strength in these joints, the plate and not the column should be considered the main member, since the plate develops the critical bolt load. The $\frac{L}{D}$ ratio, and projected area of the bolt, therefore, should be determined for the portion through the plate, and the allowable bearing load computed on this basis.

Example. Compute the allowable load on one ⅝ in common bolt through two 1⅝ in thick plates and a separating upright. The lumber and pole upright are both Southern yellow pine.

The basic stress for Southern pine in compression perpendicular to the grain, from Col. 2, Table VII, is 320 psi, and the diameter factor for a ⅝ in. bolt (Table VIII) is 1.52. The $\frac{L}{D}$ ratio of the length of bolt in one plate is 1⅝ ÷ ⅝ or 2.6. At this value of $\frac{L}{D}$ (Table IX), there is no reduction of stress perpendicular to the grain, and the allowable bolt bearing stress is 1.52 × 320 or 486 psi. The area of bearing on the bolt is 1⅝ in × ⅝ in or 1.02 sq in. The allowable two end load on one bolt is, therefore, 1.02 × 486 or 496 lb. Where load is applied to only one end of the bolt, allowable load would

Table VII—Factors for Calculating Allowable Strength of Bolts in Seasoned Wood

Species	Stresses for Determining Allowable Bolt-Bearing Stresses of Joints with Timber Side Plates: Parallel to Grain S	Perpendicular to Grain S_1
Douglas Fir: Coast Type	1160	320
Western Larch	1160	320
Southern Yellow Pine	1160	320
Western Hemlock	960	300
Lodgepole Pine	760	220
Ponderosa Pine	800	250
Red Pine	840	220

Table VIII—Diameter Factor for Bolts

Diameter of Bolt (Inches)	Diameter Factor
¼	2.50
⅜	1.95
½	1.68
⅝	1.52
¾	1.41
⅞	1.33
1	1.27

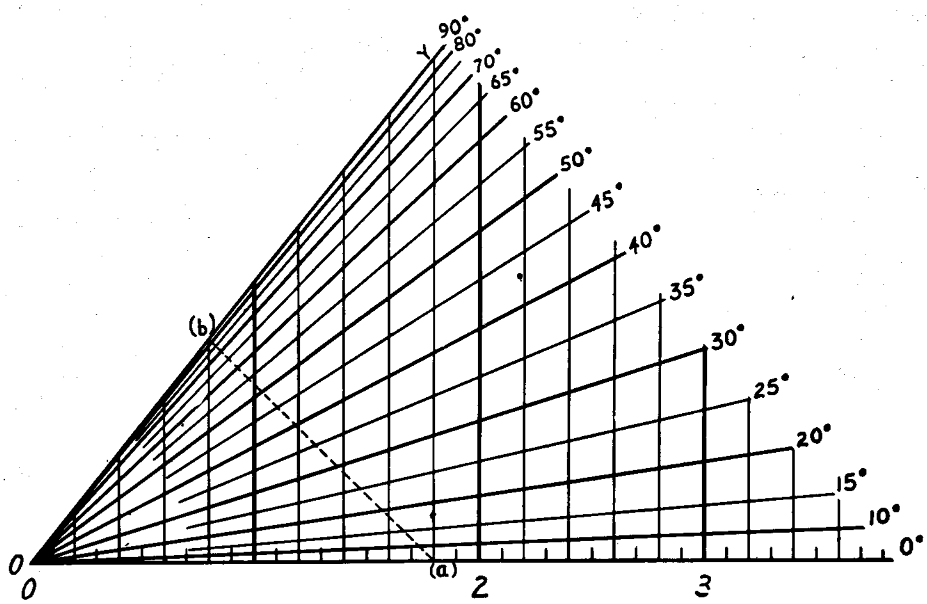
Table IX—Percentages (r) of Basic Stress Used in Calculating Allowable Bearing Stresses For Common Bolts When Load Is Applied

Length of Bolt in Member Divided By Diameter	Bolts Bearing Parallel to Grain When Basic Stress Is:		Bolts Bearing Perpendicular to Grain When Basic Stress Is:	
	750 to 950 psi	1000 to 1200 psi	200 to 280 psi	300 to 350 psi
1	100	100	100	100
2	100	100	100	100
3	100	100	100	100
4	99.5	97.4	100	100
5	95.4	88.3	100	100
6	85.6	75.8	100	100
7	73.4	65.0	100	97.3
8	64.2	56.9	96.1	88.1
9	57.1	50.6	86.3	76.7
10	51.4	45.5	76.2	67.2
11	46.7	41.4	67.6	59.3
12	42.8	37.6	61.0	52.0
13	39.5	35.0	55.3	45.9

be one half this or 248 lb. When bolt holes are properly centered, the safe load on a number of bolts is the sum of their individual capacities.

These figures are for long time loading under dry or interior conditions, and may be increased for loads of short duration. They also include a limit on distortion, and again could be increased where minor distortion is not objectionable.

Loads At Any Angle With The Grain



Allowable bolt loads when the loads act at any angle to the grain of the wood may be estimated from the Scholten nomograph* shown above.

Example: When P, the allowable load parallel to the grain, is 1800 lb and Q, the allowable load perpendicular to the grain is 800 lb, find the allowable load at an angle of 40° to grain. Connect with a straight edge 1800 lb, (a) on the O-X axis with a point (b) on line O-Y, directly above 800 on the O-X axis. Intersection of line ab and the line representing 40° is vertically above 1200 lb on O-X, the permissible load.

* Nomograph devised by John A Scholten, Engineer, U. S. Forest Products Laboratory.

Bibliography

Lateral Earth Pressure, Jacob Feld, with Discussion, Transactions, A. S. C. E., Vol. 88, 1923.

Earth Pressure, Earth Resistance and Bearing Capacity of Foundations, Dr. Ing. H. Krey, Berlin, 1926.

Effect of Depth of Embedment on Pole Stability, J. P. Seiler, Wood Preserving News, November 1932.

Lateral Pile-Loading Tests, L. B. Feagin, with Discussion, Transactions, A. S. C. E., Vol. 102, 1937.

Steel H-Beams and Steel Sheet Piling Used as Cantilevers in Soil, (from Herman Blum, in Einspannungsverhaltnisse bei Bohlwerken), United States Steel Corporation, 1938.

Lateral Earth and Concrete Pressures, Lazarus White and George Paaswell, with Discussion, Transactions, A. S. C. E., Vol. 104, 1939.

Foundation Depths for Self-Supporting Poles Subjected to Transverse Loads, Lieut. Comdr. James R. Griffith, U. S. Navy, 1939.

Symposium on Load Tests of Bearing Capacity of Soils, A. S. T. M., Special Technical Publication No. 79, June 1947.

Foundations to Resist Tilting Moments, W. C. Anderson, The Union Metal Manufacturing Company, 1947.

Specifications and Dimensions for Wood Poles, American Standards Association, April 1948.

A Report of Field and Laboratory Tests on the Stability of Posts Against Lateral Loads, Walter L. Shilts, Leroy D. Graves and George G. Driscoll, Notre Dame University, 1948.

Symposium on Lateral Load Tests on Piles, A. S. T. M., Special Technical Publication No. 154, July, 1953, and No. 154-A, June 1954.

Engineering Design Manual, Outdoor Advertising Association of America, 1955.

How to Prevent Wind Damage to Farm Buildings, by Ray W. Carpenter and Albert V. Krewatch, Fact Sheet 86, University of Maryland, College Park, Md.

Houses Can Resist Hurricanes, Forest Products Laboratory, Madison, Wis., No. FPL33, August, 1965.

Thrust Loading on Piles, James F. McNulty, A. S. C. E., Journal, Soils Mechanics and Foundations Division (SM2), April 1956.

Rigid Post Construction—Posts Embedded in Earth, West Coast Lumberman's Association, 1956.

Soil Modulus for Laterally Loaded Piles, Bramlette McClelland and John A. Focht, Jr., A. S. C. E., Journal, Soils Mechanics and Foundations Division (SM4), October 1956.

Performance of Structural Pole Frames Under Test Load, D. V. Doyle, Forest Products Laboratory, Madison, Wis., No. 2111, June, 1958.

Wood Preserving News, Articles on Pole-Type Construction in issues of September and November, 1949; January, February, March, June, August, September, November and December, 1950; February, March, June, September, October and December, 1951; January, February, April, May, June, September, November and December, 1952; January, February, April, May, June, August, September, November and December, 1953; January, February, March, April, August, September, October and December, 1954; March, April, June, August, September, November and December, 1955; and February, March, April, May, July, August, October, November and December, 1956. February, March, April, May, June, July, August, September, October, November and December, 1957; January, July, August, September and October, 1958. Photographs on pages 1-6 from issues of May 1957; April, October, December, 1959; March, 1960; April, November, 1961; March, April, May, July, August, 1962; January, March, May, June, July, August, November, 1963; May, August, September, November, 1964; February, March, November, December, 1965; February, March, November, December, 1966; August, October, 1967; January 1968.

Index