STRENGTH OF HOUSES

APPLICATION OF ENGINEERING PRINCIPLES TO STRUCTURAL DESIGN

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by

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Foreword

Strength of houses in the past has been made adequate by patterning them after those which have withstood the test of service conditions. Architects and builders of small structures have followed closely traditional methods handed down from the craftsmen of medieval England. From these traditions, cities have crystallized building codes now enforced under the police power of the community.

The trend for the immediate future seems to indicate houses so constructed as to contribute in greater measure to the welfare of the occupants by bringing more of the out-of-doors into the house. Wider windows to give more sunlight and allow stimulating vistas of garden, trees, and flowing water; larger rooms and movable partitions; and walls, floors, and roofs fabricated from plastics and from aluminum and magnesium alloys are some of the improvements anticipated.

Library research failed to disclose rational methods for determining the strength of present-day houses and little in that respect that could be applied to house design for the future. This report is an attempt to apply engineering methods to the design of houses for strength. Fundamental data for wind, snow, and floor loads have been reviewed and convenient methods developed for computing applied loads.

The engineering approach to strength of houses described in this report, it is thought, will open the way for designers to introduce unconventional materials and unusual methods of fabrication by determining in the laboratory whether constructions have the necessary strengths, thus greatly shortening the time required to develop and obtain acceptance of new constructions for houses.

Some approach along rational lines is necessary if houses are to benefit from the fund of technical information now available on materials and methods of manufacture being utilized for other commodities.

It is time that the strength of houses be given careful engineering scrutiny — not because houses need be stronger, for few fail — but to judge how much material is superfluous. Material is costly as is the labor required to shape and fit it into place.

E. U. Condon, *Director.*

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ABSTRACT

Methods for designing small houses to have adequate strength without waste of material are described and illustrated in this report. For each element of a house, compressive, transverse, and racking loads were computed by the accepted principles of mechanics for typical one- and two-story frame houses in several locations representative of extreme wind and snow loads in the United States.

Allowable loads for the 100 wall, partition, floor, and roof constructions included in the "structural properties" Building Materials and Structures Reports are compared with design loads for two houses in three locations. The comparison shows that some had insufficient strength, while others were much stronger than is necessary.

The application of engineering principles to the design of houses presents a complete and logical method for determining allowable loads for walls, floors, and roofs, and makes it practicable to develop house constructions that have sufficient strength and require the least amount of material.

I. INTRODUCTION

Substantial shelter has been a contributing factor in the advancement of civilization down through the ages, particularly where it has furnished an environment conducive to creative work in the realm of invention and culture. Although intensive study has resulted in unmistakable improvements in many lines, the evolution of structural design for houses has not kept pace with the advance in engineering requirements for other types of construction, with the progress attained in the development of safer and more rapid methods of transportation, or with the achievements in speedier and more efficient means of communication.

In an effort to improve the structural design of houses and impelled by the results of studies of substandard dwellings and their demoralizing effect on the community, the National Bureau of Standards, by quantitative evaluation of technical construction aspects, has investigated by scientific method the structural characteristics of a house which are measurable and to which engineering principles can be applied. Social and economical aspects of housing are not treated in this discussion. Some of the properties of a house which can be measured are strength, heat transmission, moisture penetration, sound insulation, and fire resistance. Although methods for measuring these properties have been known for years, their application to house constructions has been limited.

Among the engineering requirements necessary for a satisfactory house, undoubtedly, the most important is strength. Low heat transmission or high sound insulation would be of no value if the structure were not sufficiently strong to withstand severe winds, heavy rains or snows, and other service loads.

With the strength of a house made adequate for the conditions to which it is to be subjected over a period of years, other desirable features may be considered and minor modifications added to make the construction satisfactory in all essential respects.

From a cost standpoint, it is desirable to use material more efficiently than is done at present. There is a well-defined trend in other engineering fields to decrease weight and increase strength by using less but stronger material. For example, the weight of a structure may be decreased by employing improved material such as heat-treated steel, by the careful design of each member for the actual load, and by more efficient methods of fabrication, such as welding. Although economies resulting from advantageous utilization of material in stationary structures may not be as great as in the manufacture of movable equipment, such as automobiles and airplanes, where this principle is strictly adhered to, still, its application to the construction of houses will effect noticeable savings in their cost. More efficient fabrication should result in benefits similar to those that have made possible the general use of motor cars and other products formerly not within reach of the average citizen.

In the past, engineering methods have been applied to the manufacture of new materials, but not to the house as a complete unit. This is evident by the lack of structural-test results. Loads under service conditions can be determined as accurately for houses as for bridges or office buildings.

Allowable loads are the loads which can safely be applied to a particular construction in a house under service conditions. Methods for computing allowable compressive, transverse, racking, concentrated, and impact loads are described in this paper, and by these methods the allowable loads were determined for each of the 100 house constructions described in individual BMS reports of the "structural properties" series.

Although it is customary for engineers to compute the allowable load which can safely be applied to a construction, it is believed that for house construction, loading tests in a laboratory are much more satisfactory. Laboratory tests provide a quick and economical means for developing a floor, wall, or roof construction to have the necessary strength at the lowest practicable construction cost. Both the materials and the design can be changed repeatedly until the most satisfactory construction is obtained.

Materials and mode of fabrication favorable to high strength and other factors are discussed for each group of building material: wood, steel, and masonry. To cite just one example, there is an indication that for wood-framed walls the allowable load is greater when the facing material (plywood or fiberboard) is glued to the studs than when it is nailed. Whether to glue or nail the faces is an economic problem which the builder can solve only after making tests and studying costs.

Some of the wall and floor specimens covered by the "structural properties" BMS reports have been considered stronger than the same constructions in houses. Judging from the meager information available, it seems probable that there is no significant difference in strength between such a specimen and the same construction in a house. However, this conclusion should receive further study and consideration to definitely settle the matter.

When designing a house for strength, it is essential to know the greatest loads which may be applied to elements of the house during its service life. These are the design loads for compression, transverse, and racking on walls and partitions, and the design transverse loads on roof and floors. Architects compute design transverse loads on floors and sometimes other design loads, but do not compute all the design loads on each wall, partition, roof, and floor. In fact, there has been no accepted method for computing these loads.

In addition to the weight of the construction, design loads for the house depend upon the plans for the house and the wind, snow, and floor loads. The methods described for computing the design compressive, transverse, and racking loads on walls and partitions, also the transverse loads on roofs and floors, should present no difficulties to an engineer because they are an application of the fundamental principles of engineering mechanics, especially the principle of static equilibrium, parallel and concurrent coplanar forces.

Design loads for a typical one-story and a typical two-story house were computed for three locations, Los Angeles, Calif.; Miami, Fla.; and Portland, Maine, through the use of a wind map based on Weather Bureau data giving the velocity pressure of the wind. Upon this velocity pressure the wind and snow loads on roof and walls depend. The wind and snow map indicates that the strength of house constructions should be different for different parts of the country. Therefore, the three locations were selected as representative of extremes in wind and snow load.

The ratio of the allowable load to the design load for each of the 100 constructions is given in this report for one- and two-story houses in the three locations. In general, the picture presented by the ratios is that constructions which have given satisfactory service over a period of years usually have enough strength for severe conditions. However, many constructions are too strong for some locations, indicating economic waste, and others are not strong enough, indicating structural weakness.

One important feature of the proposed method for obtaining design loads is that design loads for the fastenings, such as roof to wall, are easily obtained. Consideration of a number of usual fastenings shows that they have only a fraction of the strength necessary for some locations.

Inadequate fastenings probably account for much damage resulting from severe storms. If the strength of all fastenings were ample for any load to be expected, the increase in cost of the house would be small and many failures of houses would be prevented. Tests of fastenings to determine strength about which there is doubt seems desirable and the cost of testing justified.

It is believed that for the first time, this application of engineering principles to structural design presents a complete and logical method for designing houses and, more important, ways for determining the allowable load of walls, floors, and roofs. This approach makes it practicable to develop house constructions that have sufficient strength with the least amount of material at the lowest cost of fabrication.

II. OUTLINE OF METHOD FOR DETERMINING STRENGTH OF HOUSES

1. Load

Each of the factors affecting the strength of a house was studied and the most reliable fundamental data selected. The applied loads on a house include the weight of furniture and occupants applied to floors, the pressure of wind on roof and walls, and the weight of snow on roof. These loads with the weight of the construction are carried by supporting elements and eventually by the earth under the foundation. Although for some of the loads the magnitude changes from time to time from an engineering viewpoint, all, except impact loads, are static loads.

The floor loads are those recommended by competent authorities after surveys to determine actual loads for different occupancy.

The greatest velocity pressure of wind and greatest snow load for each station were computed from data furnished by the Weather Bureau and are presented herein for ready use by house designers. For a given house, the wind load on roof depends upon the slope and on both walls and roof; upon the air pressure inside the house; which, in turn, depends upon the area of openings and their location with respect to direction of wind. For a given house, values may be taken directly from graphs based on these relationships.

The weight of construction may be computed or, more accurately, determined by weighing large specimens of the wall or roof.

2. Design Load

From the dimensions given on the plans for the house, also the applied loads and weights of constructions, it should be practicable to compute the design load for each floor, wall, and roof. It is convenient to consider each roof or story separately, beginning at the top of the house. The work is greatly simplified if each element and also the entire house is considered statically determinate, i.e., the loads and reactions on an element do not depend upon the deformation under load of either the loaded element or adjacent elements.

Actually, some houses are statically indeterminate, but, if the design loads are based on assumptions that they are statically determinate, there is reason to believe the the design is safe although it may be somewhat uneconomical.

Typical assumptions are: each floor is a rigid diaphragm insofar as loads in the plane of the floor are concerned, portions of wall in each story between doors and windows are simple vertical beams under horizontal wind loads, and they are supported laterally only at the floor above and below. Wind load on portions of wall having openings is transmitted laterally half to each of the adjacent walls; floors and roofs are simple beams having only two supports.

3. Racking Load

It was found that this approach could be applied satisfactorily except for racking loads on walls and load-bearing partitions. If the floor is a rigid diaphragm and the house has more than one room, the house is statically indeterminate under the racking load because the load on an intersecting wall or partition depends upon the load on the other elements and, in turn, on their racking moduli.

The conventional engineering solution involves simultaneous equations and for this case the solution is very tedious. This method does not appear suitable for designing houses.

After some study, it was found that if the story above was assumed to move to leeward, parallel to its original position, until the total racking resistance equalled the wind load, in most cases the floor was not in equilibrium under these forces. It was evident that the story above must rotate about the center of rotation of the racked story until equilbrium is established.

For translation, only walls and partitions parallel to the wind are loaded but, for rotation, all the walls and partitions are loaded except those intersecting the axis of rotation.

Racking loads, determined by considering translation as an approximate solution only, may be either very much less or very much greater than the correct values. In the latter case, the design is not safe.

The computation of racking loads by steps is simple and direct and should present no difficulties to an engineer. Heretofore, little or no attention has been paid to racking loads; certainly no method of making sure that a house would not collapse under wind loads has been available, although this subject has received much attention for steel-frame buildings.

It is quite evident that engineering training is essential for the application of these methods for obtaining the design loads on a particular house. These methods apply particularly if many houses are to be built from the same plans — as for prefabricated houses.

4. Allowable Load

Perhaps the reason adequate methods have not been developed heretofore for finding all the design loads for each element of a house is because there have been no very satisfactory methods for determining the allowable (working) loads on a particular construction. Of course, allowable transverse loads on wood-frame floors and roofs are computed frequently, but these methods cannot be applied with confidence to prefabricated sheet-steel floors nor to other unusual constructions. In particular, there are no accepted methods for computing allowable impact or racking loads on any construction.

To determine the strength of any house construction under all the loads to which it is subjected in a house, a systematized program of laboratory tests on large portions of a wall, floor, or roof was worked out. Previous reports in this series give the structural properties for 100 house constructions of wood, steel, and masonry. Some are conventional constructions and others are newer developments, as yet not extensively used.

The Building Materials and Structures (BMS) reports give the results of the National Bureau of Standards investigations of the properties and suitability of unusual building materials and methods of construction.

A list of BMS reports, with prices and method of purchasing, may be obtained without cost from the National Bureau of Standards, Washington 25, D. C. The structural reports are those having "structural properties of" in the title.

This report discusses the basic considerations for selecting an allowable load from the laboratory data, based on strength and safety considerations only. It should be a simple matter for anyone to modify the criteria to comply with other essential requirements but, in general, the cost of the house will be increased.

Allowable loads for each of the constructions for which the structural properties have been determined are given in this report. Because the values are not based on experience with actual houses, they should be considered as engineering estimates to be confirmed by experience.

If the design loads for each element of a given house are known, suitable constructions can be selected, or can be developed, that will provide adequate strength without the use of unnecessary material or workmanship. It seems evident that only in some such way as this can houses be designed with the assurance that they will not fail under loads for which they have been designed and also that materials will be used efficiently.

III. DEAD LOAD

The force exerted by gravity on an element of a house may be considered a dead load. It is, therefore, the weight of the construction, which may be expressed in pounds per square foot of face area. For a particular construction, the weight may be taken as the nominal value which has found acceptance in the building industry, or, it may be computed from the description and dimensions of the materials given in the specifications; or, it may be obtained much more accurately by weighing the completed construction. The BMS reports on the structural properties of house constructions give the weight of each construction which was obtained by dividing the weight of the specimens by the actual face area.

IV. FLOOR LOAD

The force exerted by gravity on objects or persons on the floor of a house may be considered the floor load, usually given in pounds per square foot of the face area of the floor.

When a house is being built, and also when it is being repaired or altered, floor loads are exerted by staging, equipment, piles of material, and workmen. For conventional constructions erected and repaired by the usual methods, these loads are not considered by the architect when designing the house because experience indicates that they do not exceed safe values. For any building, whether of conventional or of unusual construction, the contractor should prevent overloading by shoring and bracing, erecting falsework, and judiciously scheduling the sequence of the operations. The contractor alone is responsible for the satisfactory completion of the building.

When a house is occupied as a dwelling, furniture and occupants exert loads on the floors. When designing a house, it is impracticable to determine for each room either the amount of furniture or the number of occupants because they vary with the occupancy. Likewise, it is not practicable to determine the weight and location of each piece of furniture. Therefore, it is customary to consider the load on a floor as uniformly distributed. Experience indicates that floors designed in this way are safe, i.e., they do not collapse under the load for which they were designed, perhaps because heavy furniture, such as pianos and bookcases, is usually placed near a wall and covers only a small portion of the floor area.

Floor loads are discussed in Minimum Live Loads Allowable for Use in Design of Buildings [1].* A survey showed that the heaviest furniture loads for residential occupancy were pianos weighing up to 55 lb/ft^2 and bookcases weighing up to 170 lb/ft^2. The equivalent uniform load was much less than 30 lb/ft^2.

After careful study, several competent archi-

* Figures in brackets indicate the literature references at the end of this paper.

tects and builders stated that for residential occupancy the furniture loads seldom exceed 15 lb/ft^2 uniformly distributed, but that the load caused by a crowd of people averaged 40 lb/ft^2 and might occur in any room at any time. It is evident, therefore, that the floor of a house should be designed for a uniformly distributed load of 40 lb/ft^2.

For persons moving in unison, as when dancing, the load should be considered an impact load. The stresses in a floor under an impact load are much greater than those under static load. However, movement in unison is practically impossible in a crowd that loads the floor to 40 lb/ft^2.

A load of 20 lb/ft^2 on an attic floor used for light storage only is recommended in Light Frame House Construction [2].

V. WIND LOAD

1. General

Wind loads are the forces exerted upon surfaces in the path of the wind. These loads depend upon velocity of wind, size and shape of surface, and angle between surface and direction of wind. Wind loads on walls and roof of a building are normal to the surface on which they act because air is an almost perfect fluid.

Experience indicates that the direction of the wind is horizontal, i.e., parallel to the ground in flat country. If the building is on a steep slope or in rough, broken country, this assumption may not be justified. Experience also indicates that the actual wind loads on a building depend considerably upon the surroundings, such as hills, trees, and adjacent buildings.

Many houses in New England were damaged or destroyed during the hurricane in 1938. The reports of this disaster indicate that high water, storm waves along the coast, and uprooted and broken trees caused most of the damage.

If the roof was lifted bodily from the building or the building moved from the foundation, probably there were no anchors. Too often these anchors are omitted.

It does not appear feasible to design houses to withstand the impact of large trees nor storm waves along the coast.

2. Velocity Pressure

The pressure exerted by the wind is

$$q = \tfrac{1}{2} m v^2, \qquad (1)$$

in which

q = velocity pressure
m = mass of unit volume of air, i.e., weight per unit volume divided by acceleration due to gravity
v = velocity of wind.

Kent [3] gives the weight of dry air as

$$W = 1.3253 B/T, \qquad (2)$$

in which

W = weight of air, pounds per cubic foot
B = height of barometer, inches of mercury
T = absolute temperature, $459.6+t$, t being temperature, degrees Fahrenheit.

The value 1.3253 is the weight, in pounds, of 4,596 ft^3 of air at 0° F and 1-in. barometric pressure.

The United States Weather Bureau furnished values up to December 31, 1939, of the "maximum wind velocity" observed at 188 stations in this country. Each value is the greatest velocity, for a time interval of 5 minutes, ever recorded at the station. The records at individual stations range from 5 to 70 years; the average is 52 years.

The data include the highest barometer reading and lowest temperature ever recorded at each station, also, the height of the anemometer cups above the ground.

To obtain the greatest velocity pressure that might occur at each station, the weight of air was computed for the greatest pressure (highest barometer) and lowest temperature, although these conditions might never occur simultaneously with the "maximum wind velocity."

The ratios of B/T range from 0.0577 for 21 stations to 0.0730 for Canton, N. Y. and average 0.0661. For standard conditions — temperature 59° F and barometric pressure 29.92 in. of mercury — the ratio is 0.0577. This average ratio of B/T is 14.6 percent greater than for standard conditions. The ratios of B/T range from 12.7 percent less than the average to 10.4 percent more (spread 23.1 percent). For the same velocity of wind, the velocity pressures have the same range as the ratios B/T.

The effect of water vapor in air upon the weight is very small and is neglected because the greater the amount of water vapor the less the weight.

Humphreys [4] says:

> From careful theoretical considerations, in which the individual eddies or parcels are supposed continuously to exchange momentum with the surrounding air, O. G. Sutton, following Ertel, derives the simple expression,
>
> $$\frac{v}{v_1} = \left(\frac{h}{h_1}\right)^{\frac{n}{2-n}},$$
>
> for moderate heights, a few feet to 200, say, in which v and v_1 are the mean wind velocities at the heights h and h_1 respectively, and n is a number, 0 to 1, though commonly about 0.25, that varies with the vigor of the

vertical interchange that, in turn, depends on the intensity of the insulation, mainly, and the condition and nature of the surface.

Taking $n=0.25$, this expression becomes

$$\frac{v}{v_1}=\left(\frac{h}{h_1}\right)^{1/7}$$

Knowing the height of anemometer, the "maximum wind velocity" at the height of 30 ft may be computed. This height was selected as about the height of the roof for three-story houses. Both the velocity and the velocity pressure decreases with a decrease in height, therefore, the observed velocity, v, was multiplied by $(30/H)^{1/7}$, in which H is the height of anemometer in feet. The relation of velocity of wind and velocity pressure to height above the ground are shown in figure 1, as ratios of the values for 30 ft.

Mattice found [5] — using a Dines pressure-tube anemometer — that during gusts, the wind velocity is considerably greater than the "extreme wind velocity" (fastest single mile) recorded by a Robinson anemometer. Mattice draws no conclusions but, after considering his data, the greatest velocity of wind for this report was taken as 50 percent greater than the "maximum wind velocity." Therefore, the velocity, v, was multiplied by 1.50/1.00.

The greatest velocity pressure, at 30 ft above ground, for each Weather Bureau station was computed by using the equation

$$q_{30}=\tfrac{1}{2}\times\frac{1.3253\ B/T}{32.2}\times\left[\left(\frac{30}{H}\right)^{1/7}\times\frac{1.50}{1.00}\times\left(\frac{5280}{60\times60}\right)v\right]^2$$

$$q_{30}=0.0996\ B/T\left(\frac{30}{H}\right)^{2/7}v^2,$$

in which

q_{30}= velocity pressure at 30 ft above ground, pounds per square foot
B= barometric pressure, inches of mercury
T= absolute temperature, $459.6+t$, t being temperature, degrees Fahrenheit
H= height of anemometer, feet
v= wind velocity, miles per hour.

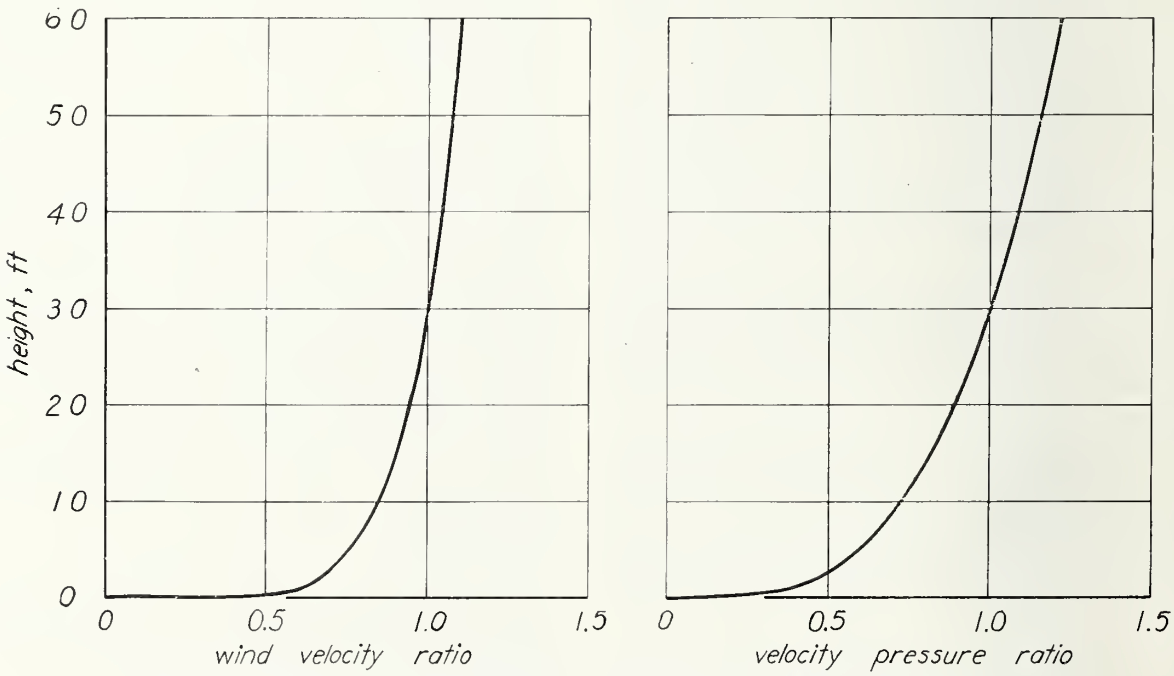


FIGURE 1.—*Relation of wind velocity and velocity pressure to height above the ground.*

[The values are ratios of value at 30 ft.]

From the velocity pressures for each station, figure 2 was plotted.

The isograms connect points having the same velocity pressure, and the value at any location is termed the "basic velocity pressure."

Assuming the velocity pressure varies linearly between isograms, the value for any location may be found by interpolation along the shortest line through the location between isograms or an isogram and a point having

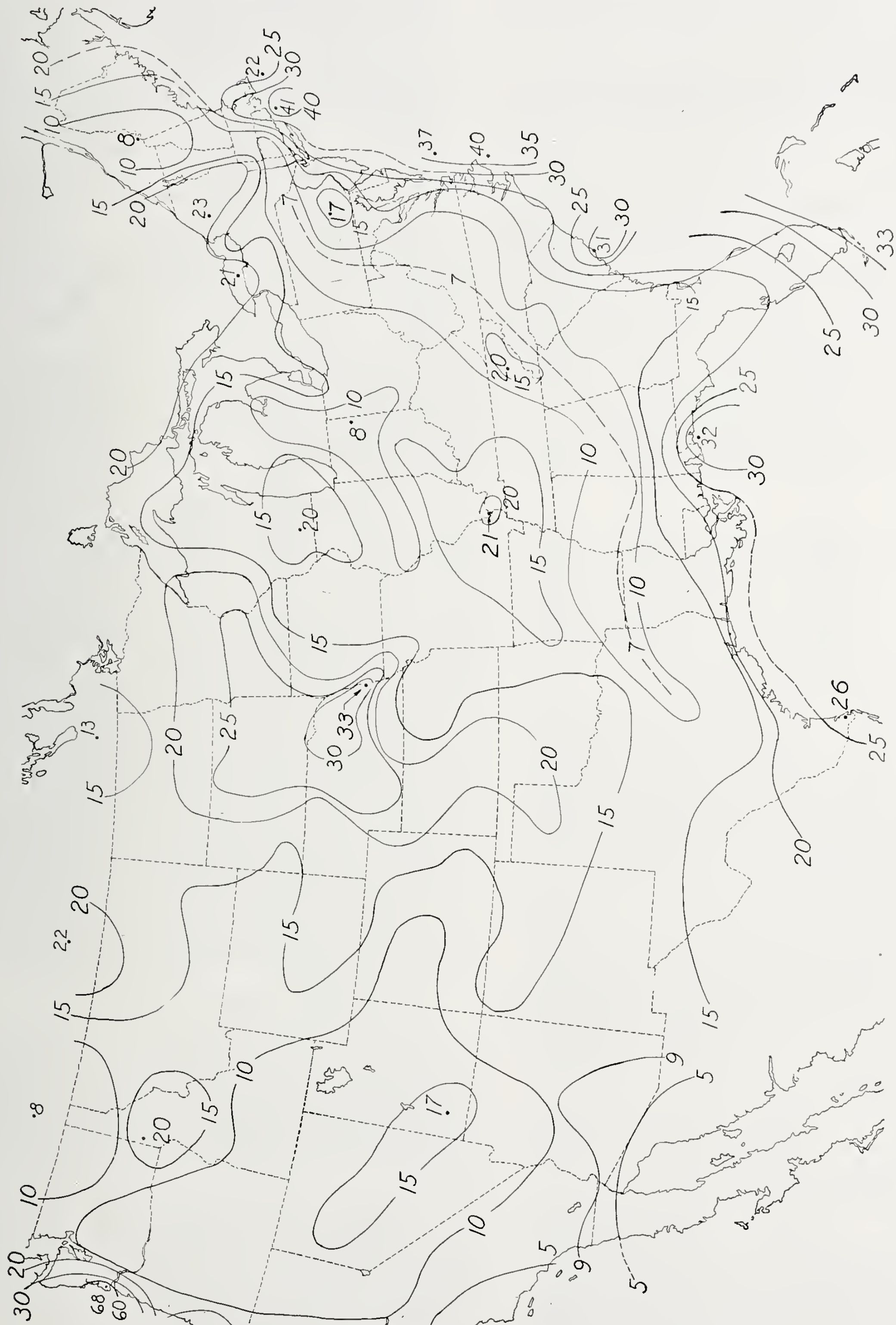


FIGURE 2.—*Wind map.*

[Isograms are maximum velocity pressures, pounds per square foot, at height of 30 ft above ground.]

different values. The line may have any direction and may cross intermediate regions within closed isograms, as around Reading, Pa., and Knoxville, Tenn.

Erie, Pa., is 6/11 the shortest distance between the 10 and 15 lb/ft^2 isograms or 12.7 lb/ft^2. Milwaukee, Wis., is half the distance from the 15 lb/ft^2 isogram to the point 20 lb/ft^2 (Madison) or 17.5 lb/ft^2.

The errors in velocity pressures taken from the map, were estimated by comparing the map values for Weather Bureau stations with the computed values. For 52 percent of the stations, the map values are too great, average difference 1.6 lb/ft^2, and for 48 percent they are too small, average difference 0.8 lb/ft^2. Of those too great, the differences do not exceed 5 lb/ft^2, except Charles City, Iowa, 5.8; Milwaukee, Wis., 7.5; Minneapolis, Minn., 8.6; Philadelphia, Pa., 5.9; and Topeka, Kan., 6.4 lb/ft^2. Of those too small, the differences do not exceed 2 lb/ft^2, except Parkersburg, W. Va., 3.0 lb/ft^2.

It is a remarkable coincidence that in regions where there are the most houses, there also are the most stations, and there probably the errors in the velocity pressure are small. This reflects credit on the judgment of the Weather Bureau when selecting sites for stations, although the use of the wind data for determining wind loads on houses was not contemplated.

The wind load on a house is a function of the velocity pressure which increases continuously from zero at the ground to the value at the top of the house as shown by velocity pressure curve in figure 1. Therefore, it does not appear practicable to compute the theoretical wind load on a given house nor the height of the resultant above ground.

To facilitate the computation of wind loads on houses, consideration was given to the simplifying assumption that the wind load on each story is a function of the velocity pressure at midheight of the story and that this load is uniformly distributed vertically. This velocity pressure is termed the "nominal velocity pressure" for the story.

Actually, the wind load is greater than the nominal value above midheight and less below midheight and the resultant of the wind load is above midheight of the story.

The nominal velocity pressure is safe for designing houses, provided the racking load and the overturning moment on story and transverse load on wall are not less than the corresponding theoretical values. At infinite height, the nominal and theoretical racking, overturning, and transverse loads are the same and the difference increases the nearer the ground.

The racking load is the resultant horizontal force on the story and the overturning moment is the racking load times the vertical distance from the bottom of story to the action line of the resultant racking load. For design, the effect of different transverse loads on walls may be judged by comparing the maximum bending moments (where shear is zero) due to the transverse wind loads, considering the wall a simple (vertical) beam supported along the bottom and top.

To determine whether nominal windloads gave safe values for design, both nominal and theoretical loads were computed and compared for stories that should give the greatest differences.

For a story 9 ft high, between 2 and 11 ft above ground, having a wall 8 ft high (platform construction), the nominal racking load is 3.2 percent greater than the theoretical load and the transverse moment on wall 7.8 percent greater.

It is believed that for any story not over 25 ft high, the nominal racking load and nominal transverse moment on a wall are greater than the theoretical values. Thus, for a story 17 ft high, between 2 and 19 ft above ground and a wall 16 ft high, the nominal racking load is 5.0 percent greater and the transverse moment 4.4 percent greater than the theoretical values. For racking and transverse loads, therefore, nominal wind loads are safe values for design.

However, the nominal overturning moment of a story, about the bottom of the story, is a little *less* than the theoretical moment. Thus, for the 9-ft story, the nominal overturning moment is 4.2 percent *less* than the theoretical value and for the 17-foot story, 4.4 percent *less*. If the overturning moment is taken either at the ground level or the level of the footings for the foundation, the nominal again is *less* than the theoretical, but the difference in percent is smaller.

Ferrington [6] says:

> The resistance of the surface of the ground, fences, hedges, shrubs, etc., all help to reduce the energy of the wind in the lower levels. . . . From the comparison of a long series of geostrophic and observed winds we conclude that over the open sea, or on an exposed spit of flat sand like Spurn Head, the wind loses one-third of its velocity from "friction" and at other well-exposed stations the loss is, on the average, as much as 60 percent. . . .

It should be remembered that theoretical values apply to houses on bare, smooth, horizontal surfaces. Actually, when a house is built, this condition may be approximated for a few locations only because the house, as soon as occupied, is inevitably surrounded by windbreaks unless erected on an exposed beach. If it is a farmhouse, barns, haystacks, and fences immediately become necessities, and soon trees and shrubs add more shelter.

If the house is in a settled community, the surrounding buildings and trees decrease the wind load considerably. These considerations justify taking the nominal wind load for all design wind loads.

To find the nominal wind load value, take the velocity pressure for the building site from the wind map, multiply it by the height factor, *h*, in table 1 for midheight of each story or roof.

Table 1.—*Height factor for velocity pressure*

[The height factor, *h*, is the ratio of velocity pressure at given height to velocity pressure at 30 ft above ground]

Factors for height from 0 to 10 ft		Factors for height from 10 to 20 ft		Factors for height from 20 to 30 ft		Factors for height from 30 to 40 ft		Factors for height from 40 to 50 ft		Factors for height from 50 to 60 ft	
Height	Height factor	Height	Height factor	Height	Height factor	Height	Height factor	Height	Height factor	Height	Height factor
ft	*h*	*ft*	*h*	*ft*	*h*	*ft*	*h*	*ft*	*h*	*ft*	*h*
0.0	0	10.5	0.7408	20.5	0.8969	30.5	1.0047	40.5	1.0895	50.5	1.1604
1.0	0.3784	11.0	.7508	21.0	.9031	31.0	1.0094	41.0	1.0934	51.0	1.1637
1.5	.4249	11.5	.7604	21.5	.9092	31.5	1.0140	41.5	1.0971	51.5	1.1670
2.0	.4614	12.0	.7697	22.0	.9152	32.0	1.0186	42.0	1.1009	52.0	1.1702
2.5	.4917	12.5	.7787	22.5	.9211	32.5	1.0231	42.5	1.1046	52.5	1.1734
3.0	.5179	13.0	.7875	23.0	.9269	33.0	1.0276	43.0	1.1083	53.0	1.1766
3.5	.5413	13.5	.7960	23.5	.9326	33.5	1.0320	43.5	1.1120	53.5	1.1797
4.0	.5623	14.0	.8043	24.0	.9382	34.0	1.0364	44.0	1.1156	54.0	1.1829
4.5	.5816	14.5	.8124	24.5	.9438	34.5	1.0407	44.5	1.1192	54.5	1.1860
5.0	.5993	15.0	.8203	25.0	.9492	35.0	1.0450	45.0	1.1228	55.0	1.1891
5.5	.6159	15.5	.8281	25.5	.9546	35.5	1.0493	45.5	1.1264	55.5	1.1922
6.0	.6314	16.0	.8356	26.0	.9599	36.0	1.0535	46.0	1.1299	56.0	1.1952
6.5	.6460	16.5	.8430	26.5	.9652	36.5	1.0576	46.5	1.1334	56.5	1.1983
7.0	.6598	17.0	.8502	27.0	.9704	37.0	1.0618	47.0	1.1369	57.0	1.2013
7.5	.6730	17.5	.8573	27.5	.9754	37.5	1.0658	47.5	1.1403	57.5	1.2043
8.0	.6855	18.0	.8642	28.0	.9805	38.0	1.0699	48.0	1.1437	58.0	1.2073
8.5	.6974	18.5	.8710	28.5	.9855	38.5	1.0739	48.5	1.1471	58.5	1.2102
9.0	.7089	19.0	.8776	29.0	.9904	39.0	1.0778	49.0	1.1505	59.0	1.2132
9.5	.7200	19.5	.8842	29.5	.9952	39.5	1.0818	49.5	1.1538	59.5	1.2161
10.0	.7306	20.0	.8906	30.0	1.0000	40.0	1.0857	50.0	1.1571	60.0	1.2190

In general, houses having unnecessary strength cost more than those having little more than sufficient strength to withstand service conditions. The economic importance of designing houses for wind loads based on the velocity pressure at midheight of each story for the building site is obvious because most of the houses in this country are one-story.

3. Relation Between Velocity Pressure and Wind Load

(a) *Subcommittee No. 31, American Society of Civil Engineers*

All available information on wind loads, including wind-tunnel data, was studied carefully over a period of ten years by Subcommittee No. 31, and is covered in the report Wind Bracing in Steel Buildings issued by the American Society of Civil Engineers. The Final Report (1940) [7] recommends a minimum standard wind load, in pounds per square foot, for the design of buildings, including tall buildings, anywhere in the United States and Canada, with the qualification that

> "Special windforce specifications should be formulated locally for areas that are definitely known to be subject to hurricanes and tornadoes."

The Subcommittee makes many definite recommendations for wind load depending on position with respect to direction of wind, slope of roof, and size and position of openings with respect to direction of wind. Sufficient allowance has been made in the prescribed wind load for the effect of air currents striking vertical faces obliquely, either in a lateral or vertical sense.

All these values are in pounds per square foot for a velocity pressure of 15.5 lb/ft^2, the velocity pressure resulting from the adoption of 20 lb/ft^2 by the Subcommittee as the minimum standard wind load.

For some locations in this country the greatest velocity pressure is less and for other locations more than 15.5 lb/ft^2.

For this report, since the wind load is directly proportional to the velocity pressure, the recommendations of the Subcommittee are given in terms of velocity pressure.

The following recommendations are those of the Subcommittee with reasonable deductions therefrom. Much of the wording is that of the Subcommittee report. Only the recommendations applicable to houses are given here.

To obtain the coefficient of velocity pressure given here, divide the wind load recommended by the Subcommittee by 15.5.

(*b*) *External Loads on Buildings With Plane Surfaces Normal to Wind*

For buildings having vertical plane faces normal to the wind, the load to be considered is a pressure of 20 lb/ft^2 corresponding to a wind velocity of only 77.8 mi/hr and a velocity pressure of 15.5 lb/ft^2. Consequently, a substantial amount of shielding is tacitly assumed to exist.

This load is the combined wind force of an external pressure (inward) of 0.8 q on the windward wall of buildings having an average ratio of height to width, with an external suction (outward) of 0.5 q on the leeward wall. Therefore, the total combined load on the outside of the windward and leeward vertical faces of an average building may be 1.3 q.

(*c*) *External Loads on Plane Surfaces Inclined to Wind*

For both symmetrical and unsymmetrical gable roofs, where α is the slope of the roof to the horizontal, in degrees, the recommended wind loads are as follows:

(*1*) *Windward Slope*

For α not greater than 20°, a suction (12 lb/ft^2) of 0.774 q out.

For α between 20° and 30°, a load of $f=(0.0774\ \alpha-2.322)\ q$ out, the negative value indicating suction.

For α equals 30°, the external load is zero.

For α between 30° and 60°, a load of $f=(0.01937\ \alpha-0.581)\ q$ in, the positive value indicating pressure.

For α greater than 60°, a pressure (9 lb/ft^2) of 0.581 q in.

(*2*) *Leeward Slope*

For all values of α in excess of zero, a suction (9 lb/ft^2) of 0.581 q out should be considered.

The external wind loads are indicated graphically in figure 3.

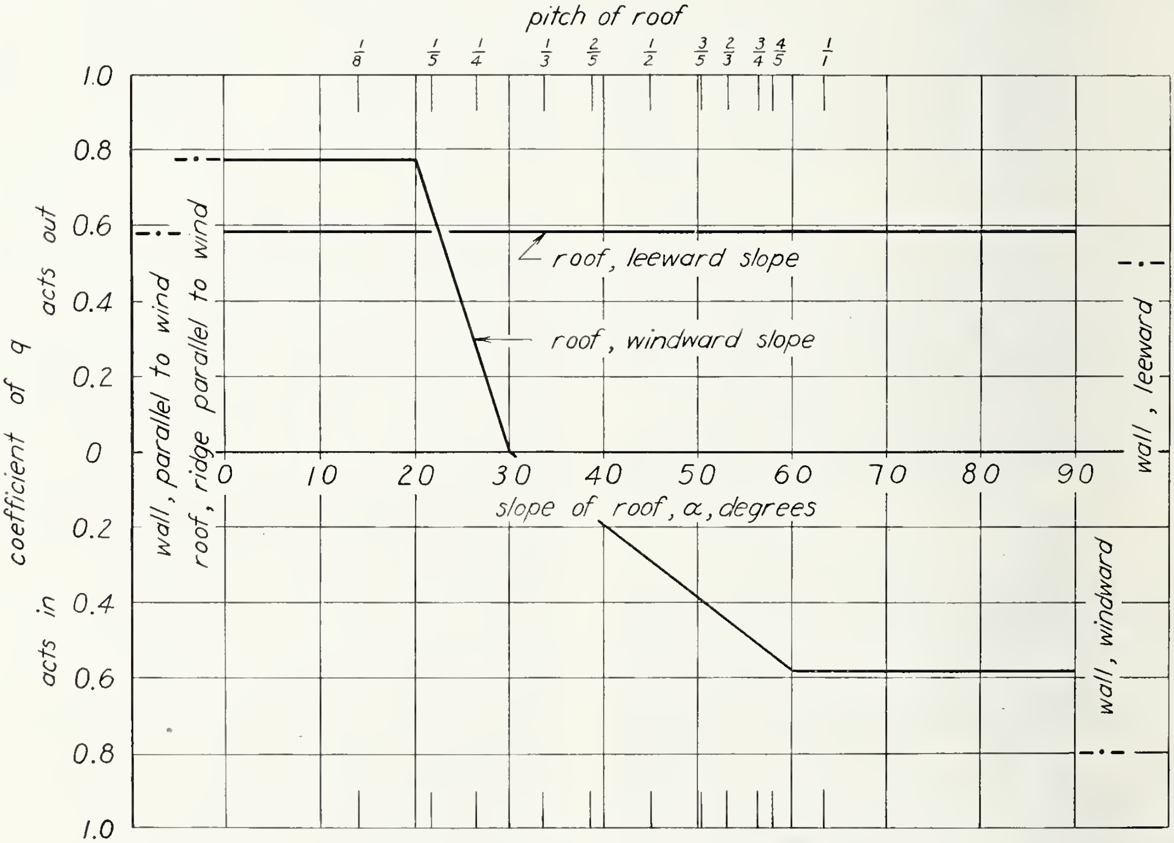


FIGURE 3.—*External wind loads on house.*

(*d*) *External Loads on Rounded Roof Surfaces*

The Subcommittee report includes values for the external wind loads on rounded roofs but they are not given here because very few houses have rounded roofs.

(*e*) *External Loads on a Flat Roof*

For a flat roof, a normal suction of not less than (12 lb/ft^2) 0.774 q out should be considered as applied to the entire roof surface.

(*f*) *External Loads on Walls Parallel to Wind*

On walls parallel to wind, an external suction (9 lb/ft^2) of 0.581 q out should be considered.

(*g*) *Angle of Wind to Walls*

These values of wind load are for rectangular houses and for the direction of the wind normal to any one of the outside walls. Using the wind loads for roofs of any slope, the greatest wind loads on the walls and the racking loads on the intersecting walls and partitions were estimated for other wind directions. It was evident that the wind loads and racking loads were greater for wind directions normal to the faces of the house than for any intermediate direction.

(*h*) *Internal Wind Loads*

The wind load to be assumed in design, is to be taken as the applicable external wind load given in paragraphs (*b*) to (*f*) only in case the building is airtight. Such a condition will rarely arise. Normally, air leakage due to the usual small openings around windows, doors, skylights, and eaves will give rise to an internal pressure or suction of from 0.25 q to 0.35 q, depending on whether the openings are chiefly in the windward or in the leeward surfaces. In the rapid building up of gust velocities, the air transfer may be so slow that the internal load is less than this. In Standard N-790 of the Netherlands, the external wind loads all have been adjusted to take account of an internal suction of 0.2 q.

Where openings are of substantial size, the internal wind load may be of considerable magnitude. It may be pressure or it may be suction, depending on whether the openings are in windward surfaces, in leeward surfaces, or in surfaces parallel to the wind. Very large internal pressures have been and may be built up because of the breaking of windows on the windward side of buildings by reason of flying gravel from the roof or other objects carried by the wind. The Subcommittee is of the opinion that the only buildings for which the internal wind load should be restricted to the amounts mentioned in the preceding paragraph are those for which the construction is such that the doors and windows cannot be broken in, or that have very small areas taken up by windows and doors.

Particularly large internal loads may arise when the windward or leeward side of a building is completely open. The German regulations, effective since June 18, 1938, provide that, for structures which are entirely open on one or more sides, or which can be opened, or which by reason of one or more openings in one or more sides are at least one-third open, or can be opened to this extent, a normal internal load of 1.2 q acting out is to be assumed as applied to the under side of the roof. This is an addition to the external suction of 0.4 q on the leeward side of plane roofs, and is specified with a view to insuring proper anchorage of the roof as well as of the attachment of roofing sheets.

It appears that reasonable allowances for internal wind load would be as follows:

(1) For buildings that, although nominally airtight with closed doors and windows and unbroken glass, are nevertheless, more or less "air leaky" by reason of numerous distributed small openings, an internal pressure or suction (4.5 lb/ft^2) or 0.2903 q out or in, acting normal to walls and roof;

(2) For buildings with 30 percent or more of the wall surface open, or subject to being opened or broken open, an internal pressure (12 lb/ft^2) of 0.774 q out or an internal suction (9 lb/ft^2) of 0.581 q in;

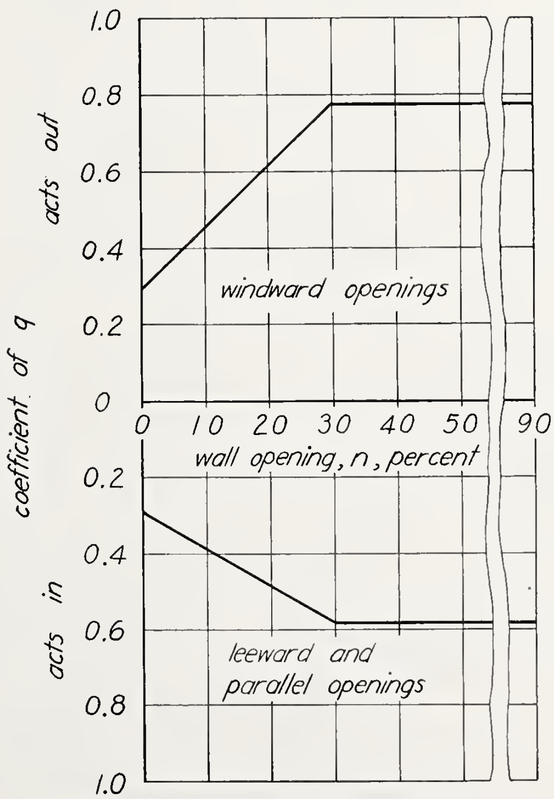
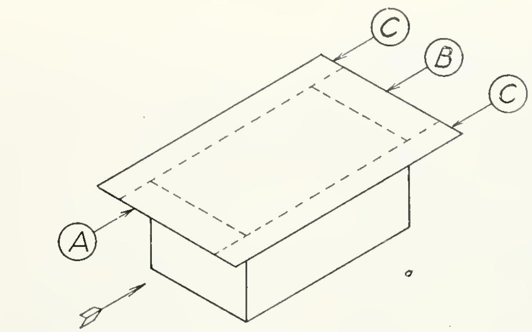
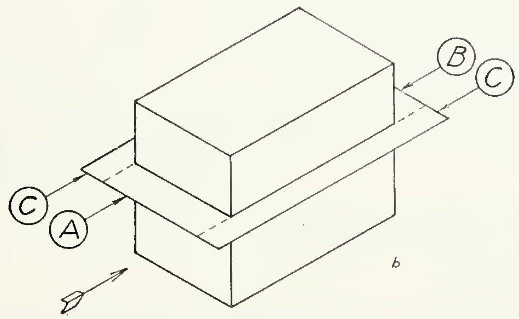


FIGURE 4.—*Internal wind loads on house.*





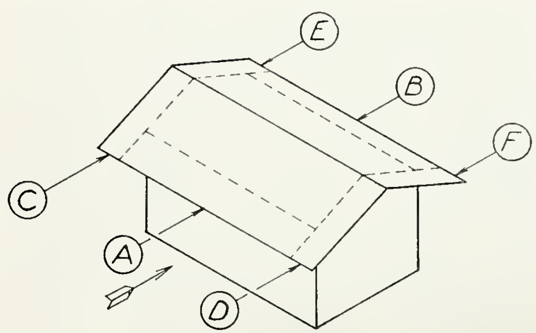


TABLE 2.—*Design wind loads for projections on house*

HORIZONTAL PROJECTION—AT TOP OF HOUSE

Face of house	Slope, α	Wind load: Upper face *coef*	Upper face *Velocity pressure*	Upper face *Acts*	Lower face *coef*	Lower face *Velocity pressure*	Lower face *Acts*	Combined or design *coef*	Combined *Velocity pressure*	Combined *Acts*
	Degrees									
Windward (A)	0 to 20	0.774	q	Up	0.8	q	Up	1.574	q	Up
Leeward (B)	0 to 20	.774	q	Do	.5	q	Down	0.274	q	Do
Parallel to wind (C)	0 to 20	.774	q	Do	.581	q	Do	.193	q	Do

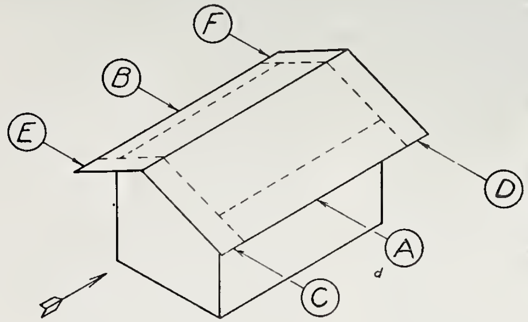
HORIZONTAL PROJECTION—BETWEEN GROUND AND TOP OF HOUSE

Face of house	Slope, α	Wind load: Upper face *coef*	Upper face *Velocity pressure*	Upper face *Acts*	Lower face *coef*	Lower face *Velocity pressure*	Lower face *Acts*	Combined or design *coef*	Combined *Velocity pressure*	Combined *Acts*
	Degrees									
Windward (A)	0 to 20	0.8	q	Down	0.774	q	Up	0.026	q	Down
Leeward (B)	0 to 20	.5	q	Up	.581	q	Down	.081	q	Do
Parallel to wind (C)	0 to 20	.774	q	Do	.581	q	Do	.193	q	Up

SLOPING PROJECTION—AT TOP OF HOUSE

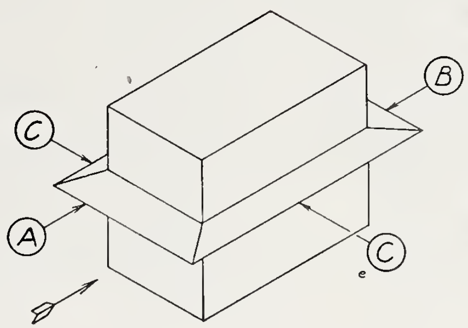
RIDGE NORMAL TO WIND

Face of house	Slope, α	Wind load: Upper face *coef*	Upper face *Velocity pressure*	Upper face *Acts*	Lower face *coef*	Lower face *Velocity pressure*	Lower face *Acts*	Combined or design *coef*	Combined *Velocity pressure*	Combined *Acts*
	Degrees									
Windward (A)	0 to 20	0.774	q	Up	0.8	q	Up	1.574	q	Up
Do	20 to 30	$(.0774\,\alpha-2.322)$	q	Do	.8	q	Do	----	----	----
Do	30	.0	----	----	.8	q	Do	0.8	q	Do
Do	30 to 60	$(.01937\,\alpha-0.581)$	q	Down	.8	q	Do	----	----	----
Do	60	.581	q	Do	.8	q	Do	.219	q	Do
Do	60 to 90	.581	q	Do	.8	q	Do	.219	q	Do
Leeward (B)	0 to 90	.581	q	Up	.5	q	Down	.081	q	Do
Parallel to wind										
Windward slope (C and D)	0 to 20	.774	q	Do	.581	q	Do	.193	q	Do
Do	20 to 30	$(.0774\,\alpha-2.322)$	q	Do	.581	q	Do	----	----	----
Do	30	.0	----	----	.581	q	Do	.581	q	Down
Do	30 to 60	$(.01937\,\alpha-0.581)$	q	Down	.581	q	Do	----	----	----
Do	60	.581	q	Do	.581	q	Do	1.162	q	Do
Do	60 to 90	.581	q	Do	.581	q	Do	1.162	q	Do
Leeward slope (E and F)	0 to 20	.581	q	Up	.774	q	Do	0.193	q	Do
Do	20 to 30	.581	q	Do	$(.0774\,\alpha-2.322)$	q	Do	----	----	----
Do	30	.581	q	Do	.0	----	----	.581	q	Up
Do	30 to 60	.581	q	Do	$(.01937\,\alpha-0.581)$	q	Up	----	----	----
Do	60	.581	q	Do	.581	q	Do	1.162	q	Do
Do	60 to 90	.581	q	Do	.581	q	Do	1.162	q	Do



SLOPING PROJECTION—AT TOP OF HOUSE — Continued

Face of house	Slope, α	RIDGE PARALLEL TO WIND								
		Wind load								
		Upper face			Lower face			Combined or design		
	Degrees	*coef*	*Velocity pressure*	*Acts*	*coef*	*Velocity pressure*	*Acts*	*coef*	*Velocity pressure*	*Acts*
Windward (C and E)	0 to 90	0.774	*q*	Up	0.8	*q*	Up	1.574	*q*	Up
Leeward (D and F)	0 to 90	.774	*q*	Do	.5	*q*	Down	0.274	*q*	Do
Parallel to wind (A and B)	0 to 90	.774	*q*	Do	.581	*q*	Do	.193	*q*	Do



SLOPING PROJECTION—BETWEEN GROUND AND TOP OF HOUSE

Face of house	Slope, α	Wind load								
		Upper face			Lower face			Combined or design		
	Degrees	*coef*	*Velocity pressure*	*Acts*	*coef*	*Velocity pressure*	*Acts*	*coef*	*Velocity pressure*	*Acts*
Windward (A)	0 to 90	0.8	*q*	Down	0.774	*q*	Up	0.026	*q*	Down
Leeward (B)	0 to 90	.5	*q*	Up	.581	*q*	Down	.081	*q*	Do
Parallel to wind (C)	0 to 90	.774	*q*	Do	.581	*q*	Do	.193	*q*	Up

(3) For buildings that may have percentages n of openings varying from 0 to 30 percent of the wall space, an internal pressure of

$$f=(0.2903+0.01612\ n)\ q \text{ out,}$$

or an internal suction of

$$f=(-0.2903-0.00969\ n)\ q \text{ in.}$$

These proposals are indicated graphically in figure 4.

(*i*) *Design Wind Load for Building Surfaces*

On the basis of the foregoing, the Subcommittee would recommend that the design wind load applied to any surface of a building be the combination of:

(1) The appropriate external wind load indicated in paragraphs (*b*) to (*f*) and

(2) The appropriate internal wind load from paragraph (*h*).

(*j*) *Elements Which are Subject to External Wind Loads on Both Faces*

The Subcommittee discusses only wind loads on walls and roofs which have external wind loads on the outside face and internal wind loads on the inside face, but many houses have elements such as overhanging eaves and porch roofs which are subject to external wind loads on both faces.

The design wind load on plane elements was estimated by considering the external wind load on both faces giving the greatest design load. With the element attached along one edge to one of the walls of a house, the wind load was taken as that on the adjacent wall *unless for some other assumption the combined load was greater*. It seems probable that the wind load on the wall extends some distance outward presumably to the outer edge of an intersecting element on a house.

The design wind loads on projections are given in table 2 and shown in figure 5.

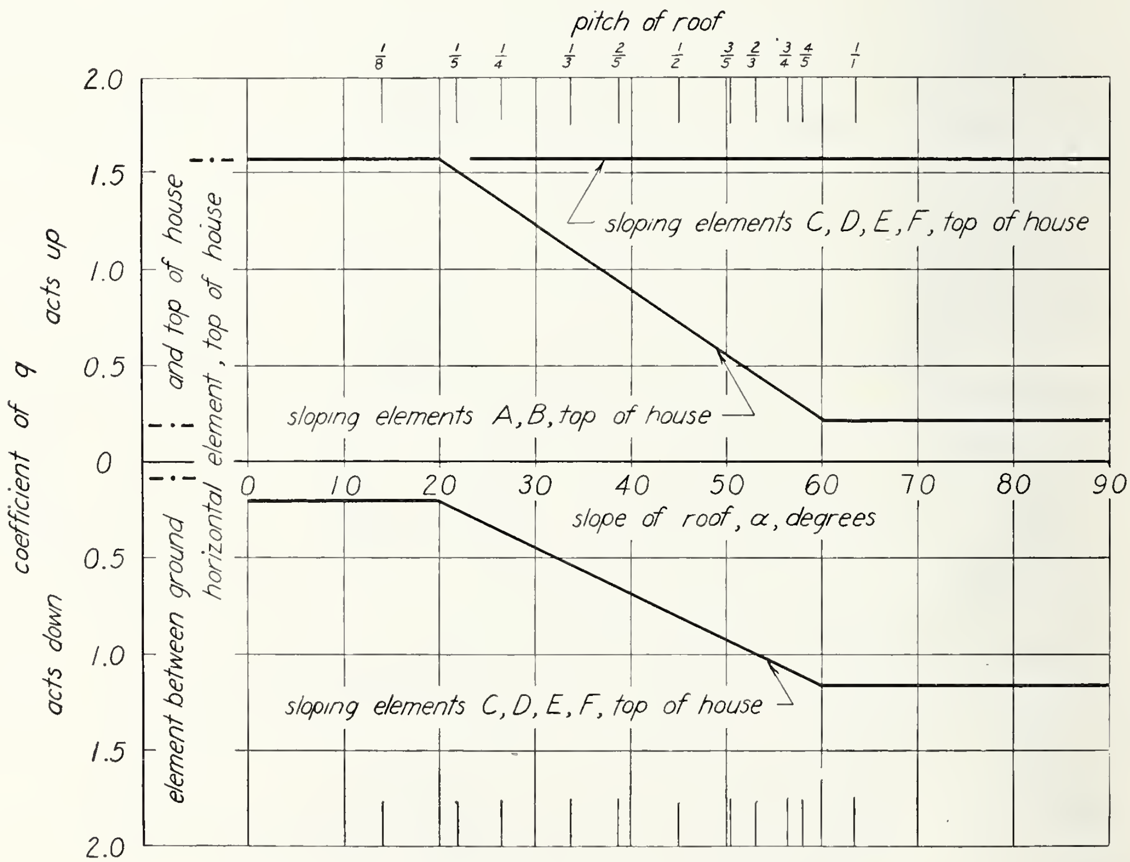


FIGURE 5.—*Design wind loads for projecting elements on house.*

4. Discussion

Wind-tunnel tests on models of buildings show that the external wind loads are far from uniform over a plane wall or roof. Therefore, the values given above are not the actual loads. Evidently the Subcommittee believes that if the building is designed for these uniformly distributed loads, it will withstand the actual wind loads.

Wind loads are given as accurately as justified by the experimental data if the coefficient of q is carried to the nearest tenth, i.e., to one decimal place. The Subcommittee gives wind loads in pounds per square foot in round numbers; therefore, the coefficients of q derived from them are carried to several decimal places, so that there will be no apparent discrepancies, particularly in values computed from the equations for coefficients depending on the slope, α, or percentage of openings, n. Insofar as accuracy is concerned, all these coefficients might well be rounded off to the nearest tenth.

For external wind loads, no value is given by the Subcommittee for wind load on gable roof if wind is parallel to ridge. For this report, each slope is considered a flat roof, slope zero degrees, and wind load taken as 0.774 q out.

5. Precise Wind Load

The computation of the external wind loads on each story of a house presents no difficulties, each face in turn being considered the windward face, but the combined load on each element depends upon the internal wind load. Because of open stairways in houses and the fact that inside doors are open much of the time and do not fit closely when closed, it must be assumed that the internal load is the same on all walls and roof. Apparently this load can be determined by considering the percentage of openings, n, in each face of each story and for each direction of wind. For a given direction of wind only two internal wind loads need be considered; the greatest for windward openings in windward face and the greatest for leeward openings in either side face or in leeward face.

The external wind load on windward face always acts inward. If there are windward openings, the internal load acts outward and the combined wind load is less than the external. However, if the openings are in either side face or in leeward face, the internal load acts inward and the combined load is the sum of the external and internal loads.

Therefore, for each direction of wind, the combined wind load on windward face is the external wind load for the story (in) plus the greatest internal wind load (in) for any leeward face in any story.

For each of the three leeward faces, the external wind load acts outward and the greatest combined wind load occurs only when there are windward openings and the internal load also acts outward.

Therefore, for each direction of wind, the combined wind load on each of the three leeward faces is the external wind load for the story (out) plus the greatest internal wind load (out) for openings in the windward face in any story.

The precise method is best explained by an example. The Federal Housing Administration cooperated in this work by supplying the plans for house E, a typical two-story frame dwelling. The location is Madison, Wis., where the velocity pressure at 30 ft is 20 lb/ft^2.

The precise external wind loads are given in table 3 and precise internal wind loads in table 4.

For computing the percentage of openings, it is assumed that doors and window sash are removed, leaving the greatest possible opening in the wall.

The precise combined wind loads and precise design wind loads are given in table 5.

The greatest combined loads, acting in and acting out, for each face of house and for each story (in bold-faced type) are the only ones considered when selecting the design wind loads.

Athough for this demonstration of the precise method each value is given, it is obvious that only those giving the greater values need be computed. They can be selected by inspection.

6. Approximate Wind Load

Architects, when determining the design wind loads on a house, may feel that the recommended method is too cumbersome and may compute the loads more quickly and easily by an approximate method in general accordance with Subcommitte recommendations. The approximate method might be somewhat as follows:

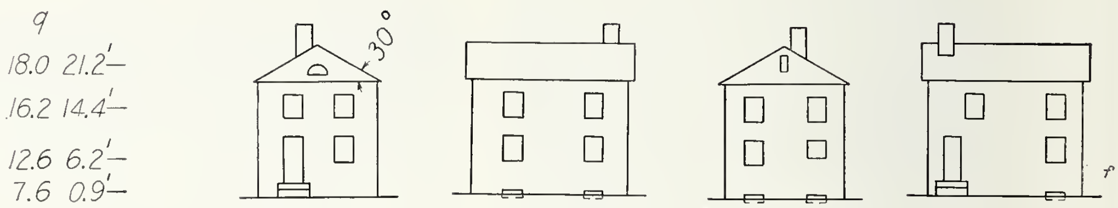
(1) The external wind load for the entire wall or face of the house and the roof is found by multiplying the velocity pressure (height 30 ft) for the location, from the map, by the coefficient recommended by the Subcommittee;

(2) The internal wind load is found by computing the percentage of openings, n, in each face and by determining the greatest outward and greatest inward load for that face;

(3) The combined wind load is found by adding the external and internal wind loads algebraically. The design wind loads for each wall and roof are the greatest value inward and the greatest value outward.

TABLE 3.—*Precise external wind loads on House E in Madison, Wis.*

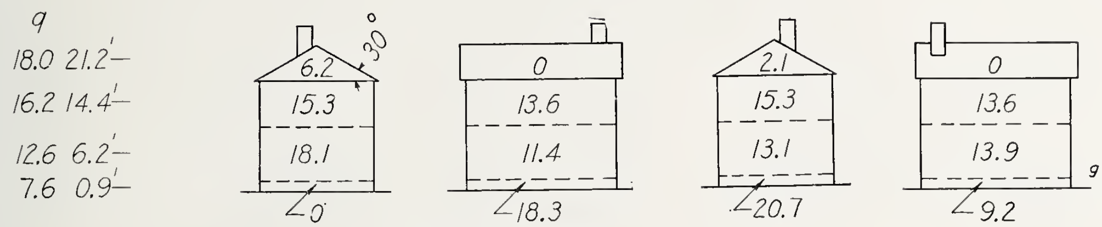
[Velocity pressure 20 lb/ft² at 30 ft]



Element	Velocity pressure, q	SOUTH FACE			EAST FACE			NORTH FACE			WEST FACE		
SOUTH WIND													
		Windward			Parallel to wind			Leeward			Parallel to wind		
	lb/ft²	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*
Gable or roof	18.0	0.8	14.4	In	0.774	13.9	Out	0.5	9.0	Out	0.774	13.9	Out
Second story	16.2	.8	13.0	do	.581	9.4	do	.5	8.1	do	.581	9.4	do
First story	12.6	.8	10.1	do	.581	7.3	do	.5	6.3	do	.581	7.3	do
Cellar	7.6	.8	6.1	do	.581	4.4	do	.5	3.8	do	.581	4.4	do
EAST WIND													
		Parallel to wind			Windward			Parallel to wind			Leeward		
Gable or roof	18.0	0.581	10.4	Out	0.0	0.0	----	0.581	10.4	Out	0.581	10.4	Out
Second story	16.2	.581	9.4	do	.8	13.0	In	.581	9.4	do	.5	8.1	do
First story	12.6	.581	7.3	do	.8	10.1	do	.581	7.3	do	.5	6.3	do
Cellar	7.6	.581	4.4	do	.8	6.1	do	.581	4.4	do	.5	3.8	do
NORTH WIND													
		Leeward			Parallel to wind			Windward			Parallel to wind		
Gable or roof	18.0	0.5	9.0	Out	0.774	13.9	Out	0.8	14.4	In	0.774	13.9	Out
Second story	16.2	.5	8.1	do	.581	9.4	do	.8	13.0	do	.581	9.4	do
First story	12.6	.5	6.3	do	.581	7.3	do	.8	10.1	do	.581	7.3	do
Cellar	7.6	.5	3.8	do	.581	4.4	do	.8	6.1	do	.581	4.4	do
WEST WIND													
		Parallel to wind			Leeward			Parallel to wind			Windward		
Gable or roof	18.0	0.581	10.4	Out	0.581	10.4	Out	0.581	10.4	Out	0.0	0.0	----
Second story	16.2	.581	9.4	do	.5	8.1	do	.581	9.4	do	.8	13.0	In
First story	12.6	.581	7.3	do	.5	6.3	do	.581	7.3	do	.8	10.1	do
Cellar	7.6	.581	4.4	do	.5	3.8	do	.581	4.4	do	.8	6.1	do

TABLE 4.—*Precise internal wind loads on House E in Madison, Wis.*

[Velocity pressure 20 lb/ft² at 30 ft. Numbers on house are wall openings in percent. Values in bold-faced type are the greatest loads acting in and acting out for each face of house]



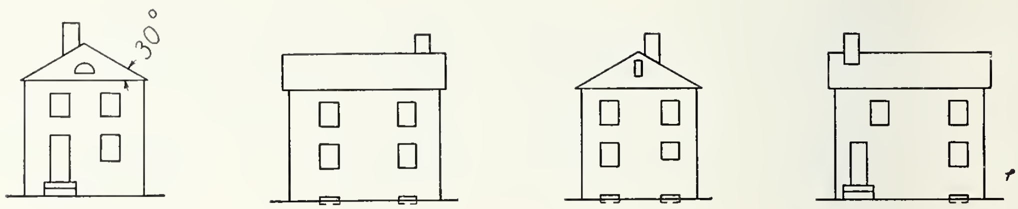
Wall opening, n, percent

		SOUTH FACE			EAST FACE			NORTH FACE			WEST FACE		
							SOUTH WIND						
Element	Velocity pressure, q	Windward			Parallel to wind			Leeward			Parallel to wind		
	lb/ft²	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*
Gable or roof	18.0	0.390	7.0	Out	0.290	5.2	In	0.310	5.6	In	0.290	5.2	In
Second story	16.2	.537	**8.7**	Do	.422	**6.8**	Do	.439	**7.1**	Do	.422	**6.8**	Do
First story	12.6	.581	7.3	Do	.401	5.0	Do	.417	5.3	Do	.425	5.4	Do
Cellar	7.6	.290	2.2	Do	.468	3.6	Do	.490	3.7	Do	.379	2.9	Do
							EAST WIND						
		Parallel to wind			Windward			Parallel to wind			Leeward		
Gable or roof	18.0	0.350	6.3	In	0.290	5.2	Out	0.310	5.6	In	0.290	5.2	In
Second story	16.2	.439	**7.1**	Do	.509	**8.2**	Do	.439	7.1	Do	.422	6.8	Do
First story	12.6	.465	5.9	Do	.474	6.0	Do	.417	5.3	Do	.425	5.4	Do
Cellar	7.6	.290	2.2	Do	.585	4.4	Do	.490	3.7	Do	.379	2.9	Do
							NORTH WIND						
		Leeward			Parallel to wind			Windward			Parallel to wind		
Gable or roof	18.0	0.350	6.3	In	0.290	5.2	In	0.324	5.8	Out	0.290	5.2	In
Second story	16.2	.439	7.1	Do	.422	6.8	Do	.537	**8.7**	Do	.422	6.8	Do
First story	12.6	.465	5.9	Do	.401	5.0	Do	.500	6.3	Do	.425	5.4	Do
Cellar	7.6	.290	2.2	Do	.468	3.6	Do	.623	4.7	Do	.379	2.9	Do
							WEST WIND						
		Parallel to wind			Leeward			Parallel to wind			Windward		
Gable or roof	18.0	0.350	6.3	In	0.290	5.2	In	0.310	5.6	In	0.290	5.2	Out
Second story	16.2	.439	7.1	Do	.422	6.8	Do	.439	7.1	Do	.509	**8.2**	Do
First story	12.6	.465	5.9	Do	.401	5.0	Do	.417	5.3	Do	.514	6.5	Do
Cellar	7.6	.290	2.2	Do	.468	3.6	Do	.490	3.7	Do	.438	3.3	Do

Table 5.—*Precise combined wind loads and precise design wind loads for House E in Madison, Wis.*

[Velocity pressure 20 lb/ft² at 30 ft. Values in bold-faced type are the greatest loads acting in and acting out for each face of house and for each story].

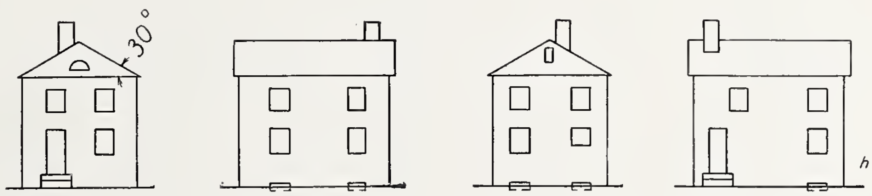
q
18.0 21.2'—
16.2 14.4'—
12.6 6.2'—
7.6 0.9'—



Element	SOUTH FACE				EAST FACE				NORTH FACE				WEST FACE			
	PRECISE COMBINED WIND LOAD—SOUTH WIND															
	Windward				Parallel to wind				Leeward				Parallel to wind			
	lb/ft²	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*
Gable or roof	**21.5**	In	0.0	Out	0.0	In	**22.6**	Out	0.0	In	**17.7**	Out	0.0	In	**22.6**	Out
Second story	**20.1**	Do	.0	Do	.0	Do	**18.1**	Do	.0	Do	16.8	Do	.0	Do	**18.1**	Do
First story	**17.2**	Do	.0	Do	.0	Do	**16.0**	Do	.8	Do	15.0	Do	.0	Do	**16.0**	Do
Cellar	**13.2**	Do	2.6	Do	2.7	Do	**13.1**	Do	3.3	Do	12.5	Do	2.7	Do	**13.1**	Do
	PRECISE COMBINED WIND LOAD—EAST WIND															
	Parallel to wind				Windward				Parallel to wind				Leeward			
Gable or roof	0.0	In	**18.6**	Out	**7.1**	In	8.2	Out	0.0	In	**18.6**	Out	0.0	In	18.6	Out
Second story	.0	Do	**17.6**	Do	**20.1**	Do	0.0	Do	.0	Do	**17.6**	Do	.0	Do	16.3	Do
First story	.0	Do	**15.5**	Do	**17.2**	Do	.0	Do	.0	Do	**15.5**	Do	.8	Do	14.5	Do
Cellar	2.7	Do	**12.6**	Do	**13.2**	Do	2.1	Do	2.7	Do	**12.6**	Do	3.3	Do	12.0	Do
	PRECISE COMBINED WIND LOADS—NORTH WIND															
	Leeward				Parallel to wind				Windward				Parallel to wind			
Gable or roof	0.0	In	17.7	Out	0.0	In	22.6	Out	**21.5**	In	0.0	Out	0.0	In	22.6	Out
Second story	.0	Do	16.8	Do	.0	Do	18.1	Do	**20.1**	Do	.0	Do	.0	Do	18.1	Do
First story	.8	Do	15.0	Do	.0	Do	16.0	Do	**17.2**	Do	.0	Do	.0	Do	16.0	Do
Cellar	3.3	Do	12.5	Do	2.7	Do	13.1	Do	**13.2**	Do	2.6	Do	2.7	Do	13.1	Do
	PRECISE COMBINED WIND LOAD—WEST WIND															
	Parallel to wind				Leeward				Parallel to wind				Windward			
Gable or roof	0.0	In	18.6	Out	0.0	In	18.6	Out	0.0	In	18.6	Out	**7.1**	In	8.2	Out
Second story	.0	Do	17.6	Do	.0	Do	16.3	Do	.0	Do	17.6	Do	**20.1**	Do	0.0	Do
First story	.0	Do	15.5	Do	.8	Do	14.5	Do	.0	Do	15.5	Do	**17.2**	Do	.0	Do
Cellar	2.7	Do	12.6	Do	3.3	Do	12.0	Do	2.7	Do	12.6	Do	**13.2**	Do	2.1	Do
	PRECISE DESIGN WIND LOADS—ANY WIND DIRECTION															
Gable or roof	21.5	In	18.6	Out	7.1	In	22.6	Out	21.5	In	18.6	Out	7.1	In	22.6	Out
Second story	20.1	Do	17.6	Do	20.1	Do	18.1	Do	20.1	Do	17.6	Do	20.1	Do	18.1	Do
First story	17.2	Do	15.5	Do	17.2	Do	16.0	Do	17.2	Do	15.5	Do	17.2	Do	16.0	Do
Cellar	13.2	Do	12.6	Do	13.2	Do	13.1	Do	13.2	Do	12.6	Do	13.2	Do	13.1	Do

In order to obtain some idea of how loads determined in this way compare with loads computed by the precise method; the approximate external, internal, and combined wind loads; together with approximate design wind loads and ratio of approximate to precise design wind loads were computed. The values are given in tables 6, 7, and 8.

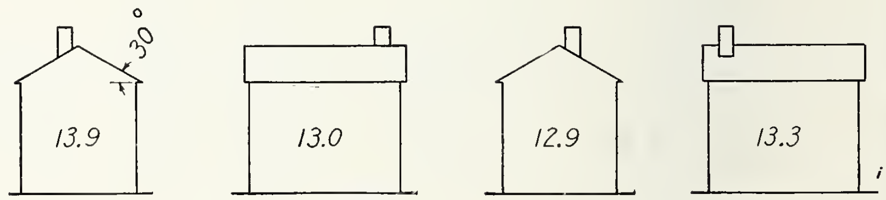
TABLE 6.—*Approximate external wind loads on House E in Madison, Wis.*



		SOUTH FACE			EAST FACE			NORTH FACE			WEST FACE		
SOUTH WIND													
Element	Velocity pressure, *q*	Windward			Parallel to wind			Leeward			Parallel to wind		
	lb/ft²	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Act*
Roof	20				0.774	15.5	Out				0.774	15.5	Out
Wall	20	0.8	16.0	In	.581	11.6	do	0.5	10.0	Out	.581	11.6	do
EAST WIND													
		Parallel to wind			Windward			Parallel to wind			Leeward		
Roof	20				0.0	0.0					0.581	11.6	Out
Wall	20	0.581	11.6	Out	.8	16.0	In	0.581	11.6	Out	.5	10.0	do
NORTH WIND													
		Leeward			Parallel to wind			Windward			Parallel to wind		
Roof	20				0.774	15.5	Out				0.774	15.5	Out
Wall	20	0.5	10.0	Out	.581	11.6	do	0.8	16.0	In	.581	11.6	do
WEST WIND													
		Parallel to wind			Leeward			Parallel to wind			Windward		
Roof	20				0.581	11.6	Out				0.0	0.0	
Wall	20	0.581	11.6	Out	.5	10.0	do	0.581	11.6	Out	.8	16.0	In

Table 7.—*Approximate internal wind loads on House E in Madison, Wis.*

[Numbers on house are wall openings in percent. Values in bold-faced type are the greatest loads acting in and acting out for each face of house].

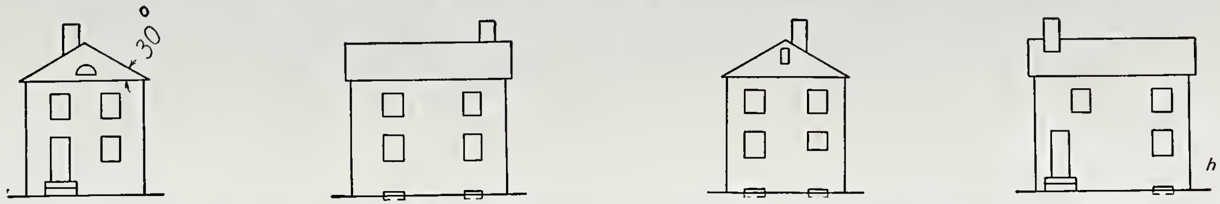


Wall opening, n, percent

		SOUTH FACE			EAST FACE			NORTH FACE			WEST FACE		
							SOUTH WIND						
Element	Velocity pressure, q	Windward			Parallel to wind			Leeward			Parallel to wind		
	lb/ft²	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*
Roof	20				0.290	5.8	In				0.290	5.8	In
Wall	20	0.514	**10.3**	Out	.416	8.3	Do	0.415	8.3	In	.419	**8.4**	Do
							EAST WIND						
		Parallel to wind			Windward			Parallel to wind			Leeward		
Roof	20				0.290	5.8	Out				0.290	5.8	In
Wall	20	0.425	**8.5**	In	.500	**10.0**	Do	0.415	**8.4**	In	.419	8.4	Do
							NORTH WIND						
		Leeward			Parallel to wind			Windward			Parallel to wind		
Roof	20				0.290	5.8	In				0.290	5.8	In
Wall	20	0.425	8.5	In	.416	**8.4**	Do	0.498	**10.0**	Out	.419	**8.4**	Do
							WEST WIND						
		Parallel to wind			Leeward			Parallel to wind			Windward		
Roof	20				0.290	5.8	In				0.290	5.8	Out
Wall	20	0.425	8.5	In	.416	8.3	Do	0.415	8.4	In	.504	**10.1**	Do

TABLE 8.—*Approximate combined wind loads, approximate design wind loads, and ratio of approximate to precise design wind loads for House E in Madison, Wis.*

[Values in bold-faced type are the greatest loads acting in and acting out for each face of house]



APPROXIMATE COMBINED WIND LOADS—SOUTH WIND

Element	SOUTH FACE: Windward				EAST FACE: Parallel to wind				NORTH FACE: Leeward				WEST FACE: Parallel to wind			
	lb/ft²	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*lb/ft²*	*Acts*
Roof					0.0	In	**25.8**	Out					0.0	In	**25.8**	Out
Wall	**24.4**	In	0.0	Out	.0	Do	**21.9**	Do	0.0	In	20.3	Out	.0	Do	**21.9**	Do

APPROXIMATE COMBINED WIND LOADS—EAST WIND

Element	SOUTH FACE: Parallel to wind				EAST FACE: Windward				NORTH FACE: Parallel to wind				WEST FACE: Leeward			
Roof					**8.5**	In	10.0	Out					**0.0**	In	21.6	Out
Wall	0.0	In	21.6	Out	**24.5**	Do	0.0	Do	0.0	In	21.6	Out	**.0**	Do	20.0	Do

APPROXIMATE COMBINED WIND LOADS—NORTH WIND

Element	SOUTH FACE: Leeward				EAST FACE: Parallel to wind				NORTH FACE: Windward				WEST FACE: Parallel to wind			
Roof					0.0	In	25.5	Out					0.0	In	25.5	Out
Wall	0.0	In	20.0	Out	.0	Do	21.6	Do	**24.5**	In	0.0	Out	.0	Do	21.6	Do

APPROXIMATE COMBINED WIND LOADS—WEST WIND

Element	SOUTH FACE: Parallel to wind				EAST FACE: Leeward				NORTH FACE: Parallel to wind				WEST FACE: Windward			
Roof					0.0	In	21.7	Out					**8.5**	In	10.1	Out
Wall	0.0	In	**21.7**	Out	.0	Do	20.1	Do	0.0	In	**21.7**	Out	**24.5**	Do	0.0	Do

APPROXIMATE DESIGN WIND LOADS—ANY WIND DIRECTION

Element	SOUTH FACE				EAST FACE				NORTH FACE				WEST FACE			
Roof					8.5	In	25.8	Out					8.5	In	25.8	Out
Wall	24.4	In	21.7	Out	24.5	Do	21.9	Do	24.5	In	21.7	Out	24.5	Do	21.9	Do

RATIO OF APPROXIMATE TO PRECISE[1] DESIGN LOADS

Element	SOUTH FACE: In			SOUTH FACE: Out			EAST FACE: In			EAST FACE: Out			NORTH FACE: In			NORTH FACE: Out			WEST FACE: In			WEST FACE: Out		
	Approx.	Precise	Ratio	Approx.	Precise	Ratio	Approx.	Precise	Ratio	Approx.	Precise	Ratio	Approx.	Precise	Ratio	Approx.	Precise	Ratio	Approx.	Precise	Ratio	Approx.	Precise	Ratio
	lb/ft²	*lb/ft²*		*lb/ft²*	*lb/ft²*		*lb/ft²*	*lb/ft²*		*lb/ft²*	*lb/ft²*		*lb/ft²*	*lb/ft²*		*lb/ft²*	*lb/ft²*		*lb/ft²*	*lb/ft²*		*lb/ft²*	*lb/ft²*	
Gable or roof	24.4	21.5	1.14	21.7	18.6	1.17	8.5	7.1	1.20	25.8	22.6	1.14	24.5	21.5	1.14	21.7	18.6	1.17	8.5	7.1	1.20	25.8	22.6	1.14
Second story	24.4	20.1	1.21	21.7	17.6	1.23	24.5	20.1	1.22	21.9	18.1	1.21	24.5	20.1	1.22	21.7	17.6	1.23	24.5	20.1	1.22	21.9	18.1	1.21
First story	24.4	17.2	1.42	21.7	15.5	1.40	24.5	17.2	1.42	21.9	16.0	1.37	24.5	17.2	1.42	21.7	15.5	1.40	24.5	17.2	1.42	21.9	16.0	1.37
Cellar	24.4	13.2	1.85	21.7	12.6	1.72	24.5	13.2	1.86	21.9	13.1	1.67	24.5	13.2	1.86	21.7	12.6	1.72	24.5	13.2	1.86	21.9	13.1	1.67

[1] Precise design wind loads taken from table 5.

The approximate design wind loads on roof range from 14 to 20 percent more than the precise values; those on the second-story walls from 21 to 23 percent more; and on first-story walls from 37 to 42 percent more.

7. Comparison With Authorities

A review of literature on wind loads indicates that, until recently, the engineering basis for computing them was not as sound as for other phases of the theory of structures, perhaps because of the lack of experimental data, in particular, results of wind-tunnel tests.

Recommended wind loads on surfaces inclined to the wind are not reviewed here. For information on this subject, the six reports of Subcommittee No. 31 may be consulted.

It is interesting, however, to summarize the recommended wind pressures (pounds per square foot) for plane surfaces normal to the wind in chronological order.

1900, Bovey [8]. Wind pressure is the maximum force which has been estimated to vary from 40 to 50 lb/ft^2 of surface *perpendicular to the direction of blow*. Ordinary gales blow with a force of from 20 to 25 lb, which may sometimes rise to 34 to 35 lb, and even to upward of 50 lb during storms of great severity. Pressures much greater than 50 lb have been recorded but they are wholly untrustworthy.

Up to the present time, indeed, all wind-pressure data are most unreliable, and to this fact may be attributed the frequent wide divergence of opinion as to the necessary wind allowance in any particular case.

It would be practically absurd to base calculations upon the violence of a wind gust, a tornado, or other similar phenomena, as it is almost absolutely certain that a structure would not lie within its range. In fact, it may be assumed that a wind pressure of 40 lb/ft^2 upon a surface perpendicular to the direction of blow is an ample and perfectly safe allowance, especially when it is remembered that a greater pressure than this would cause the overthrow of nearly all the existing towers, chimneys, etc.

1908, Burr and Falk [9]. A design pressure of 30 lb/ft^2 is usual.

1910, Johnson, Bryan, and Turneaure [10]. A pressure exceeding 30 lb/ft^2 is very improbable over a space as wide as 150 or 200 ft and generally a greater pressure than this does not extend over a path wider than 60 ft. A maximum pressure of 30 lb/ft^2 would appear ample for areas of any considerable size; and in localities not subject to tornadoes or hurricanes, or in protected locations, a maximum pressure of 20 to 25 lb/ft^2 is sufficient. The usual assumption for roofs and exposed buildings is 30 lb.

1911, Marburg [11]. In good practice the usual allowance for pressures on surfaces normal to the direction of the wind is 30 lb/ft^2 which is about the strength of ordinary windows. This allowance may be regarded as amply sufficient for small roofs and buildings except in very exposed situations, for which 40 lb/ft^2 may be assumed. In built-up districts and for buildings which present very large surfaces to the wind the assumed pressure may be safely reduced to 25 lb or even less.

1912, Hool [12]. For all inclinations greater than 60° the normal pressure is practically 30 lb/ft^2. Some engineers, however, consider 40 lb/ft^2 a suitable figure to use in practice.

1914, Ketchum [13]. It would seem that 30 lb/ft^2 on the side and the normal component of a horizontal pressure of 30 lb/ft^2 on the roof would be sufficient for all except exposed locations. If the building is somewhat protected, a horizontal pressure of 20 lb/ft^2 on the sides is certainly ample for heights less than 30 ft.

1914, Morris [14]. Thirty pounds per square foot has come to be recognized pretty generally as a safe value for the wind pressure on vertical walls of ordinary height.

1914, Schneider's Specifications for wind in Ketchum's Structural Engineers' Handbook [13]. The wind pressure shall be assumed as acting in any direction horizontally. First—At 20 lb/ft^2 on the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs. Second—At 30 lb/ft^2 on the totally exposed surfaces of all parts composing the metal frame work. The frame work shall be considered an independent structure without walls, partitions, or floors.

1914, Thayer [15]. For office buildings 30 lb/ft^2 in any direction.

1921, Ketchum [16]. A pressure of 30 lb/ft^2 on vertical face of buildings seems to be sufficient for all except the most exposed positions.

1924, Hool and Kinne [17]. A maximum pressure of 30 lb/ft^2 is ample for structures in exposed positions. For structures in a protected position, 20 to 25 lb/ft^2 is ample.

1927, Swain [18]. For buildings in cities, partially sheltered, 30 lb/ft^2 is probably ample or excessive, while in exposed situations it may be desirable to assume as high as 50 lb/ft^2.

1930, Merriman and Wiggin [19]. For buildings not more than 25 ft to the eave line, a horizontal wind pressure should be assumed not less than 15 lb/ft^2 on the sides and the corresponding normal component on the roof according to the Duchemin formula for wind pressure on inclined surfaces. For buildings more than 25 ft to the eave line, the horizontal pressure should be taken at not less than 15 lb/ft^2 for the lower 25 ft, and 20 lb/ft^2 for the side surface above 25 ft and the normal component on the roof.

1932, Urquhart and O'Rourke [20]. The wind is assumed to blow horizontally with a velocity sufficient to cause a pressure of 30 to 40 lb/ft^2 on a vertical surface.

1939, Spofford [21]. Pressure on vertical surfaces may be taken normal to surface and equal to the assumed wind pressure; for bridges 30 lb/ft^2 in any direction.

1940, Subcommittee No. 31, ASCE [7], recommends a wind force of 20 lb/ft^2 for buildings not exceeding 300 ft in height.

1945, ASA American Standard [22].

The following are the requirements for buildings less than 50 ft high:

Design wind pressure 20 lb/ft^2 in any direction. Every exterior wall shall be capable of withstanding this load acting either inward or outward.

The roofs of all buildings shall be capable of withstanding a load equal to 25 lb/ft^2 applied over the entire roof and acting outward normal to the surface.

The windward slope only of roofs or sections of roofs with slopes greater than 30° shall be capable of withstanding a load of 20 lb/ft^2 acting inward normal to the surface.

Overhanging eaves and cornices shall be capable of withstanding outward loads equal to 40 lb/ft^2.

Many texts, very properly, refer to the building code for the location of the building for the design wind load; some give values for the larger cities.

None of the authorities call attention to the fact that wind loads have different values in some regions from those in others; therefore, the values they give apply to a building anywhere in the United States. It is a reasonable assumption that many buildings have been designed for the values given by each authority and that damage or failure due to wind has been infrequent. Otherwise, the values would have been discredited and greater values substituted. On the other hand, if the values were much too great there would not be the same incentive to change them and, as years rolled by, they would tend to be above criticism.

Presumably, all of the recommended wind pressures given in the literature are a combination of the force inward on the windward wall ($0.8\,q$) and that outward on the leeward wall ($0.5\,q$) or $1.3\,q$, in accordance with the viewpoint of the Subcommittee, although the authors do not discuss the actual load distribution.

It is evident that the velocity pressure corresponding to a given design wind pressure may be obtained by dividing the pressure by 1.3 and the result can be compared with the velocity pressures shown on the wind map. The velocity pressures corresponding to the recommended wind pressures are

Wind pressure	Velocity pressure
lb/ft^2	*lb/ft^2*
15	11.5
20	15.5
25	19.2
30	23.1
34	26.2
35	26.9
40	30.8
50	38.5

Of the 16 authorities, 12 give pressures ranging from 20 to 30 lb/ft^2 for ordinary conditions.

Considering first the ASA American Standard, the design wind pressure in any direction is equivalent to a velocity pressure of 15.4 lb/ft^2.

For racking and overturning, the ASA value is correct.

For transverse wind load on wall, there are two conditions to be investigated, an airtight building and a building with openings.

If the building is airtight, the wind load for windward wall is ($0.8\,q$) 12.4 lb/ft^2 acting in; on leeward wall ($0.5\,q$) 7.75 lb/ft^2 acting out; on wall parallel to wind ($0.581\,q$) 9 lb/ft^2 acting out. All these loads are much less than 20 lb/ft^2. But, as usually constructed, houses are not airtight.

If the building has 30 percent or more of openings in each wall, the internal wind load for windward openings is ($0.7739\,q$) 12 lb/ft^2 acting out and for leeward openings ($0.581\,q$) 9 lb/ft^2 acting in.

The design loads for each wall are 21.4 lb/ft^2 acting in and 21.0 lb/ft^2 acting out, only slightly greater than the ASA value.

Taking House E as a typical dwelling, the approximate loads for a velocity pressure of 15.5 lb/ft^2 are 0.775 of those for Madison. The greatest transverse wind load on walls inward is 19.0 and outward, 17.0 lb/ft^2, a little less than the ASA value of 20 lb/ft^2. The greatest wind load inward on roof is 6.6 and outward 20 lb/ft^2, the latter the ASA value.

The velocity pressures corresponding to the design wind pressures recommended by authorities were compared with the wind map.

The value of 20 lb/ft^2 (velocity pressure of 15.5 lb/ft^2) is not exceeded except near the Atlantic and Gulf Coasts, western Vermont, northern New York, eastern Pennsylvania, Florida, eastern and western Tennessee, western Kentucky, southern and northwestern Wisconsin, northern and southern Illinois, eastern and northwestern Missouri, northern Arkansas, Minnesota, western Iowa, southern North Dakota, South Dakota, Nebraska, western Kansas, Oklahoma, northern Texas, northern and eastern Montana, southeastern Wyoming, eastern Colorado, northeastern New Mexico, southwestern Utah, northern Idaho, southern and western Nevada, southeastern and western Washington, and northwestern Oregon.

The value of 25 lb/ft^2 (velocity pressure of 19.2 lb/ft^2) is not exceeded except close to the Atlantic and Gulf Coasts and in Washington and Oregon on the Pacific Coast. Other regions having greater values are northern New York, southern Florida, northwestern Wisconsin, southern Illinois, southwestern Minnesota, western Iowa, southeastern North Dakota, eastern South Dakota, eastern Nebraska, western Kansas, western Oklahoma, and northern Montana.

The value of 30 lb/ft^2 (velocity pressure of 23.1 lb/ft^2) is not exceeded except intermittently along the Atlantic and Gulf Coasts and in Washington and Oregon along the Pacific Coast. A few other regions having greater values are southern Florida, southwestern Minnesota, eastern South Dakota, and eastern Nebraska.

The value of 40 lb/ft^2 (velocity pressure of 30.8 lb/ft^2) is exceeded only in Rhode Island, eastern Massachusetts, Charleston, S. C., southern tip of Florida, around Pensacola, Fla., northeastern Nebraska, western Washington, and northwestern Oregon.

The value of 50 lb/ft^2 (velocity pressure of 38.5 lb/ft^2) is exceeded only in southern Rhode Island, Cape Hatteras, N. C., and very near the Pacific Coast in Washington and Oregon.

It should be remembered, that when recommending design wind pressures most of the authorities, perhaps unconsciously, were thinking only of regions with which they were familiar and of the larger cities.

Most of the regions for which 20 lb/ft^2 appear to be inadequate are not thickly populated, therefore the authorities and ASA American Standard are justified provided these regions prepare local codes.

For 30 lb/ft^2, the same comment applies except that the regions are much smaller, which confirms the judgment and experience of the authorities giving this value.

It may be pointed out that the wind map gives velocity pressures for very exposed building sites although a somewhat smaller value may be justified if the building is sheltered, but necessarily any decrease in the value must be left to the designer. For this report, velocity pressures are taken from the map.

8. Combined and Design Wind Load

It is recommended that houses be designed for all loads—dead load, floor load, wind load, snow load, and water load, if any—applied simultaneously although actually the occurrence of any one of them is unlikely and the occurrence of all of them at the same time highly improbable. It is very difficult to believe that when the wind is blowing a gale there can be the greatest depth of snow on the roof or that during a record-breaking blizzard there will be standing room only in each room.

Experience may show that a house is safe if designed for combinations of loads somewhat less than the greatest load.

Combined loading conditions at all likely to occur may be floor load 10 lb/ft^2 with: first, the greatest wind load and half the greatest snow load; or second, half the greatest wind load and the greatest snow load.

Combined wind and snow loads on roofs have been discussed at length by structural engineers, apparently without general agreement.

Designing houses for the greatest value of each load is suggested tentatively and is the method followed in this report.

VI. SNOW LOAD

1. Basic Snow Load

The weight of snow accumulated on the roof constitutes the snow load on a house. It varies with the altitude of the building site, the geographical location of the building, and the slope of the roof.

The United States Weather Bureau (1939) furnished data on the greatest depth of snow in sheltered areas (such as clearings in a forest) at 166 stations. The data cover periods of 15 to 67 years; the average was 44 years. That Bureau gave the average density of snow soon after it had fallen as 0.10 that of water. Two inches of snow weigh about one (1.04) lb/ft^2. Later, the depth decreases and the density increases. The snow load for each station was computed and figure 6 drawn. The isograms connect points having the same snow load, and the value at any location is termed the "basic snow load."

Assuming the snow load to vary linearly between isograms, the value for any location may be found by interpolation along the shortest line through the location between isograms or an isogram and a point having *different values*. It may cross intermediate regions within closed (or nearly closed) isograms. For locations south of the Great Lakes and north of the regions of 10 lb/ft^2 or more, the line may cross these regions from the 5 lb/ft^2 isogram along the Gulf to the 10 lb/ft^2 isogram along the Great Lakes. Columbus, Ohio, is 4/5 this distance, 4 lb/ft^2 greater than 5 lb/ft^2 or 9 lb/ft^2.

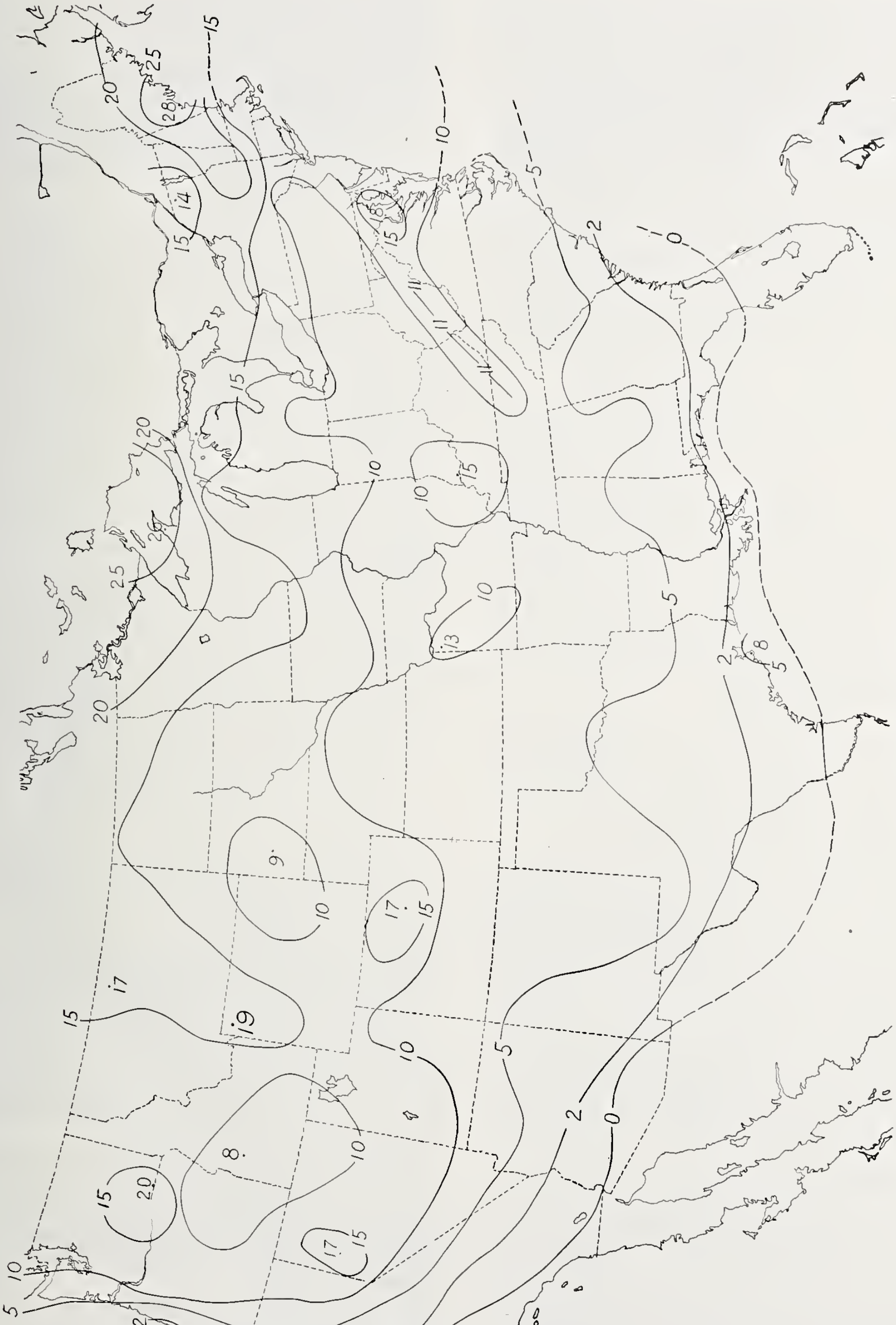


FIGURE 6.—*Snow map.*

[Isograms are basic snow load, pounds per square foot.]

The errors in snow loads taken from the map were estimated by comparing the map values for Weather Bureau Stations with the value computed from the observed depth of snow.

For 63 percent of the stations, the map values are too great, average 1.1 lb/ft^2, and for 37 percent they are too small, average difference 1 lb/ft^2. Of those too great, the differences do not exceed 3 lb/ft^2, except for Sandusky, Ohio, Providence, R.I., Sheridan, Wyo., and Trenton, N.J., for which the differences range from 4 to 5.1 lb/ft^2. Of those too small, the differences do not exceed 2.0 lb/ft^2, except Albany, N.Y., for which the value is 3.0 lb/ft^2.

2. Comparison with Authorities

Although the snow load on a flat roof undoubtedly is less than on the ground in sheltered areas, the only way to compare the values recommended by authorities with those from the map is to consider the values recommended for flat roofs. The snow loads recommended by Ketchum [23] for latitude 35° and Kidder-Parker [24] for the Southern and Pacific states agree closely with those on the map. For more northern locations their values are about twice those on the map, implying an average snow density of 0.2 that of water.

The values recommended by Hool [12] agree closely with those on the map but Merriman-Wiggin [19] are slightly greater, except those for Baltimore, Cincinnati, and St. Louis which are about the same as the map values.

For houses, it is believed that the values on the map are justified. The snow load on a roof is somewhat less than on the ground around the house because wind blows snow from the roof. The depth of snow on the roof of an occupied house decreases with time after a snow storm more rapidly than on a storage building because the house is heated. If there is thermal insulation in the roof, snow will remain on the roof longer than if there is no insulation, but, even in northern states, snow disappears from the roofs of houses in a short time.

3. Snow Load on Sloping Roof

Apparently structural engineers agree that the snow load on the horizontal projection of a sloping roof is less than on a flat roof because wind blows more snow from the sloping roof and snow slides off; the greater the slope the less the snow load. The ration of snow load on horizontal projection of roof to basic snow load is the "stay-on" factor k.

After consideration of the recommendations in the literature, for this report, values of k were taken as follows.

For slopes of 20° or less......1.00.

For slopes of 20° to 60°.... $1.50-0.025\,\alpha$.

For slopes greater than 60°...0.

For any slope, the factor k may be taken from figure 7.

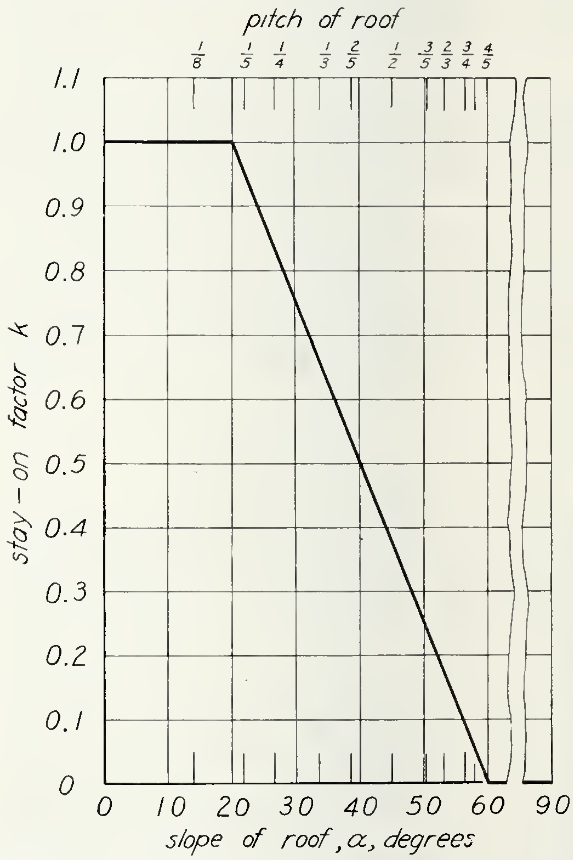


Figure 7.—*Relation between slope of roof and stay-on factor k.*

For slopes from 20° to 60°, these values of k are somewhat greater than those computed from the snow loads for sloping roofs recommended by Ketchum [23], Kidder-Parker [24], Marburg [11], and Schneider in Ketchum's Structural Engineers' Handbook [23].

All authorities agree that all snow slides off a roof if the slope is 45° to 60°, provided there are no snow guards. The slope causing the snow to slide depends upon the roughness of the roof covering; the rougher the surface the greater the slope.

Because sleet may freeze to the roof as it falls, Ketchum [23] and Kidder-Parker [24] recommend a snow load not less than 5 to 10 lb/ft^2 (horizontal projection) for all slopes except in Southern and Pacific States.

Hool [12] says on roofs inclined 60°, snow load is often neglected if there are no snow guards as it is naturally expected that snow will slide off. Merriman-Wiggin [19] say there is no snow on 45° roofs if there are no snow guards and Schneider's formula [23] gives no snow load for slopes 45° and greater.

If the slope is 60° or more, it is very improbable that there is any sleet load on the roof because houses are heated.

Knowing the basic snow load and the factor k, the snow load on the surface of the roof may be computed. In figure 8, let A be snow on the ground; B, snow on the horizontal projection of

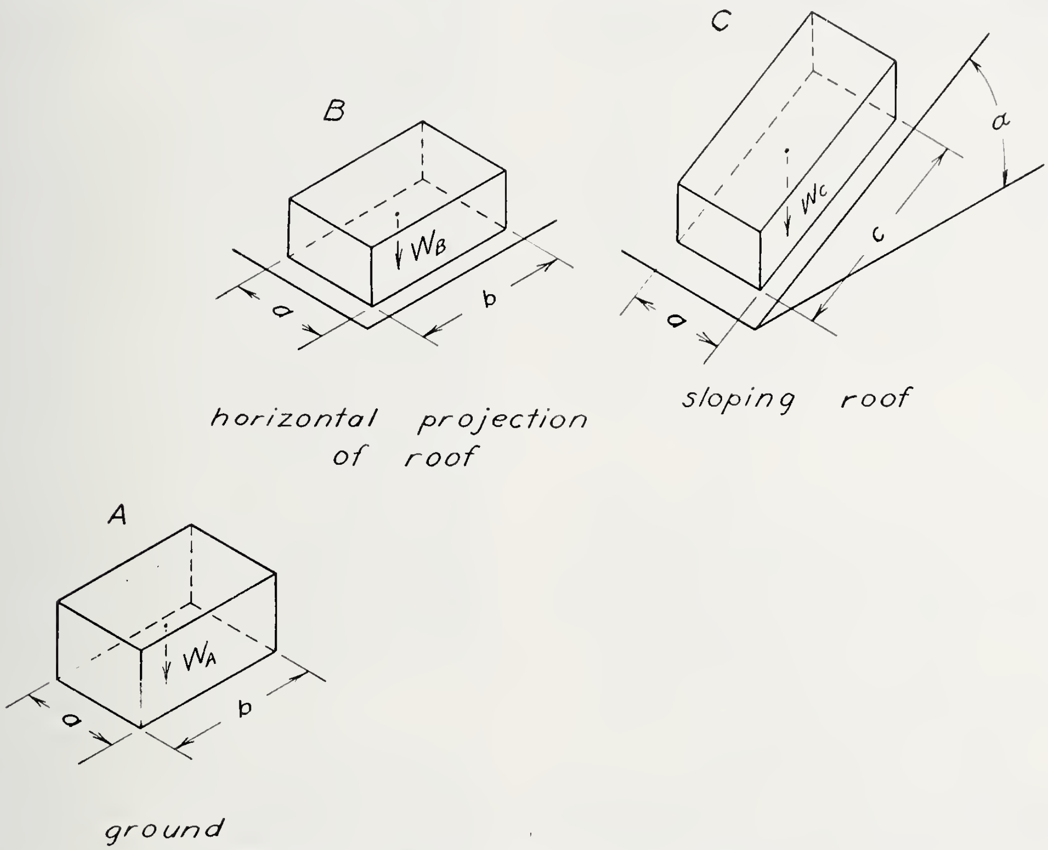


FIGURE 8.—*Snow load on ground and on sloping roof.*

the roof; and C, snow on the roof having the slope $\propto$. The dimensions of the bases are a, b, and c, and the Weights W_A, W_B and W_C.

The snow load on ground is

$$S_B = W_A / (a \times b);$$

that on horizontal projection,

$$S_H = W_B / (a \times b);$$

and on roof,

$$S_R = W_C / (a \times c),$$

but $W_C = W_B = kW_A$, and $c = b / \cos \propto$ because b is the horizontal projection of c.

Then

$$S_R = kW_A \cos \propto / (a \times b),$$

but $W_A / (a \times b)$ is the basic snow load. Therefore, the snow load on a sloping roof is $S_R = kS_B \cos \propto$. If K, the snow-load factor is k cos $\propto$ then $S_R = KS_B$. The value of K for any roof may be taken from figure 9.

VII. WATER LOAD

Water accumulated on a roof constitutes a water load. Obviously, water can accumulate only on a conventional flat roof if it is entirely surrounded, as by a parapet wall and only when the drains, if any, are clogged. The maximum water load depends upon the depth of water. As water weighs 62.4 lb/ft^3, the water load, w, lb/ft^2, at any place on the roof is

$$w = 62.4d \text{ lb/ft}^2,$$

in which d is the depth of water, feet.

Unconventional houses may be designed for maintaining water on the roof for aquatic gardens, swimming pools, thermal insulation, or other purposes.

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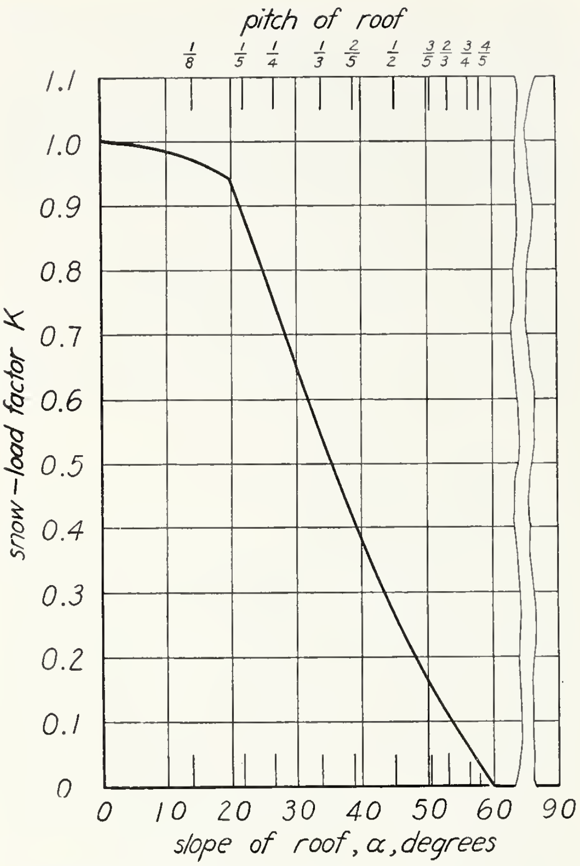

FIGURE 9.—*Relation between slope of roof and snow-load factor K.*

VIII. ASSUMPTIONS

The structural analysis of a house is greatly simplified if, except under racking load, it is considered a statically determinate structure; then four assumptions are justified.

Floor and roof rigid diaphragm.—Each floor and roof is considered a rigid diaphragm under forces acting in its plane. These forces cause neither failure nor appreciable deformation of the floor or roof.

It is reasonable to assume that if a floor or roof is satisfactory under design transverse loads it, also, is satisfactory under the coplanar loads such as tensile, compressive, and racking loads applied under service conditions. If, for a particular floor or roof construction, the validity of this assumption is doubted, then the construction should be tested and the structural properties under these loads determined.

Wall connected to floor.—Each wall and load-bearing partition is so connected to adjoining floor or roof at bottom and top that all horizontal reactions are transmitted. These reactions may be either normal to wall (wind load) or parallel to wall (racking load).

If floor and roofs are to be supported, obviously, in addition at least some of the walls must be so connected to floors and roofs that vertical reactions are transmitted.

Floor and roof simple beam.—Each floor and roof is a simple beam between adjacent supports such as walls and load-bearing partitions.

In most cases, no bending moment is transmitted past a support but moment may be transmitted, as past a wall, by a balcony or by overhanging eaves.

It is believed that there is very little economic justification for a continuous floor or roof in a house. The safety of the design depends greatly upon the actual elevation of supports being the design elevation and any change in elevation with time may cause failure, particularly changes resulting from unequal settling of foundations. If for economical erection of the house, joists and rafters are continous over more than two supports, they should be designed as simple beams between adjacent supports; then, in all probability, the house will be safe although somewhat uneconomical in use of material.

Wall simple beam.—Each wall and load-bearing partition is a simple beam supported laterally at bottom and top to floor or roof.

In most cases no bending moment is transmitted past a floor but moment may be transmitted as past attic floor by a gable end which is not supported laterally by roof.

IX. CONVENTIONS

When designing structures it is customary for engineers to follow well-established conventions for drawings and computations that simplify and expedite the work, especially if it is to be verified by other engineers. The conventions in general use for wood, steel, and concrete structures are not very helpful for designing houses because few are applicable.

The following conventions are suggested and followed in this report with the expectation that, in the future, they will be replaced by more useful suggestions when available.

1. DEFINITIONS

Anchor.—A fastening for elements or members resisting forces acting to cause separation of the contacting surfaces.

Bearing, Floor and Roof.—Horizontal line parallel to supporting wall through action lines of supporting reactions.

Closure.—Part of house closing an opening in wall or partition, i.e., door or window.

Construction.—Materials, dimensions, and methods of fabricating an element of a house. Elements are of different constructions if there is a significant difference in materials, dimensions, or method of fabrication.

Element.—Portion of completed house ready for occupancy, having one primary function, for example, a floor or wall.

Fastening.—Any device resisting relative displacement of two elements or members.

Height Factor, h.—Ratio of velocity pressure at given height to velocity pressure at 30 ft above ground.

Height, Story.—Vertical distance from the surface of a floor to surface of the next floor above.

Height, Story-Wall.—Wall not continuous past floor (platform construction), vertical distance from wall bearing at bottom to wall bearing at top of wall.

Wall continuous past floor (solid masonry), vertical distance from floor bearing at bottom of wall to floor or roof bearing next above.

Key.—A fastening for elements or members in contact resisting forces acting to cause displacement parallel to the contacting surfaces.

Load, Allowable.—Greatest applied load for which a construction functions satisfactorily in a house.

Load, Applied.—Load on an element in addition to the weight of the construction.

Load, Design.—For a particular house, greatest load on an element that may occur under service conditions.

Opening.—An aperture in wall or roof that may be closed by a door or window.

Portion.—An undivided section of an element having uniform width.

Reaction.—Load exerted by a support.

Stay-on Factor, k.—Ratio of snow load on horizontal projection of roof to basic snow load.

Snow Load, Basic.—Greatest snow load on ground ever recorded at a given location.

Snow-load Factor, K.—Ratio of snow load on roof surface to basic snow load.

Support.—Member of a house in contact with and exerting a load on another member or element.

Velocity Pressure, Basic.—Greatest velocity pressure at height 30 ft above ground ever recorded at a given location.

Velocity Pressure, Nominal.—Value at given location at midheight of given story or roof.

Weight.—Of a construction, the weight in pounds per square foot based on the face area.

2. General

(*a*) *Dimensions*

The dimensions for a house may be obtained from the plans. If they are drawn to scale, a dimension not given numerically may be measured.

For walls and roofs the dimensions are those measured on the outside of the house. An error of one percent (1 in. in 10 ft) is negligible when computing design loads. The difference between the longer dimensions of a completed house and the dimensions shown on the plans may exceed one percent because of differences between actual and nominal dimensions of materials and building units and, also, because of unavoidable errors in workmanship.

(*b*) *Weight*

The weight of elements of the house may be taken as the value accepted by the building industry, may be computed from information in specifications, or may be determined more accurately by weighing the element. Each BMS report on structural properties gives the weight based on the face area. Because of variations in the density, moisture content, and dimensions of commercial materials, the weight may vary as much as 10 percent.

(*c*) *Structural Properties*

The properties of a construction, including allowable load, may be computed by the usual engineering methods or they may be determined, much more accurately, by testing specimens of the construction representative of portions of a completed house.

The properties of over 100 constructions, determined by testing large specimens, are given in the BMS reports on structural properties. For some of these constructions, the properties also were computed by engineering methods but they did not agree, even approximately, with the test values. This indicates, quite definitely, that the structural properties of house constructions cannot be satisfactorily computed from average properties of the materials because the strength and other properties of many kinds of fastening are unknown and, for frame constructions, there is no accepted method for computing the effect of the faces such as sheathing and plaster on the strength of the construction.

3. Elements

(*a*) *Wall and Partition*

(*1*) *Height*

For any house suitable for human occupancy, the ceiling height (vertical distance from surface of floor to ceiling above) must allow the occupants to stand upright, a minimum of about 6 ft 6 in. It also should comply with health requirements, in some building codes at least 7 ft 6 in. From an economic viewpoint, if the ceiling height exceeds 9 ft, the cost of the house and of heating may be considered excessive. If the wall is not continuous past the floor (platform construction) the story-wall height is the ceiling height. If the wall is continous past the floor (solid masonry) the story-wall height is the ceiling height plus the thickness of floor. Therefore, the least wall height for a house should be 7 ft 6 in. The nominal height of walls and partitions in BMS structural reports is 8 ft.

(2) *Width*

The width of each portion of wall may be obtained from the plans. It is the distance between adjacent openings or edges of the wall. Likewise, the width of each portion of a partition may be obtained, being the distance between a door and the opposite face of intersecting wall or partition.

(3) *Thickness*

For design, the thickness of wall or partition is the distance between the inside and outside surfaces of the structural members. For frame walls, the studs are the structural members.

For masonry walls, stucco, furring, lath and plaster, if any, are disregarded. For cavity walls, if floor and roof bear only on the backing, the wall thickness is the thickness of the backing; if they bear on both backing and facing, the wall thickness is the distance from inside surface of backing to outside surface of facing. For brick veneer walls, if floor and roof bear only on the wood frame, the wall thickness is the distance from inside surface of studs to outside surface; if they bear on both, the thickness is the distance from surface of stud nearer inside of house to outside surface of brick veneer.

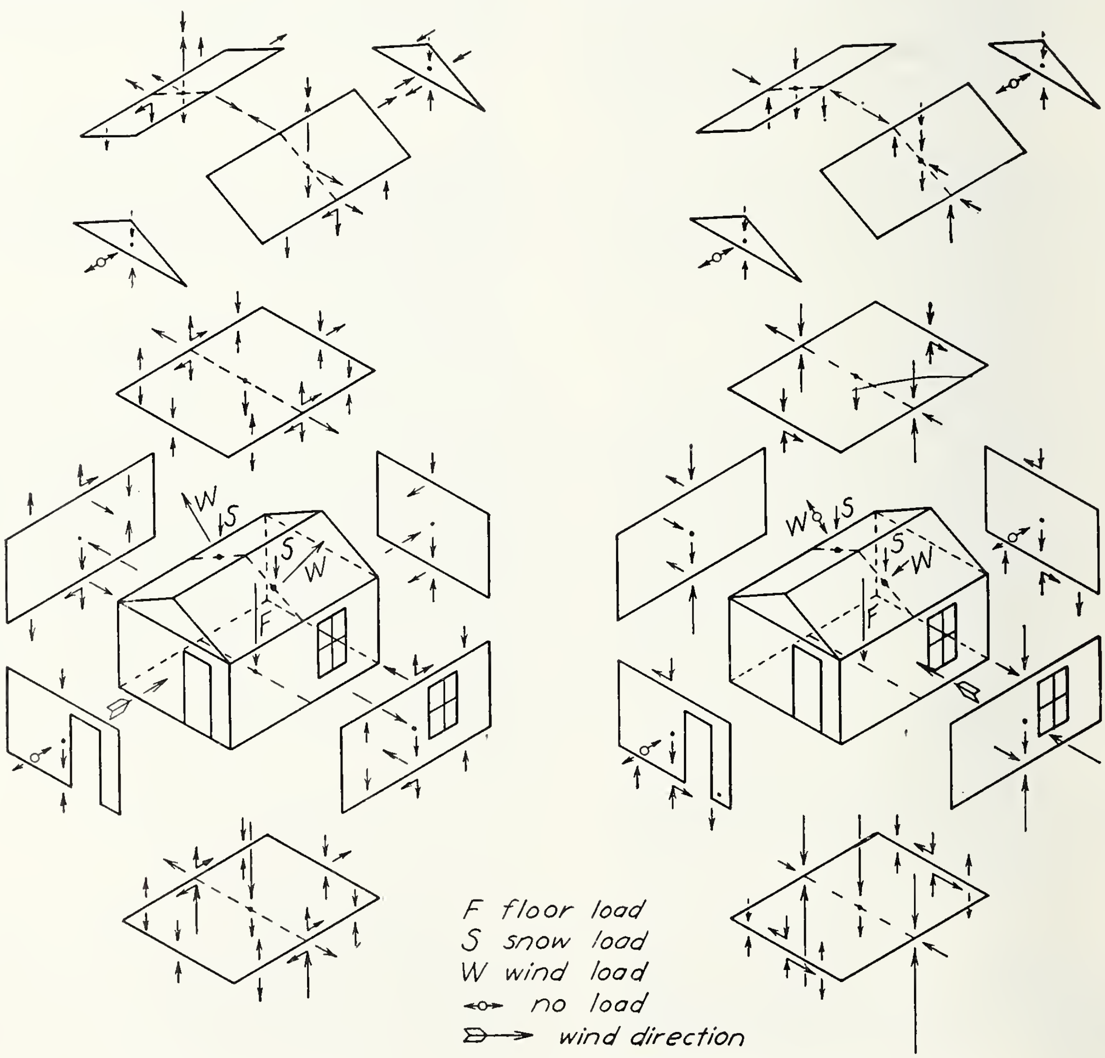

FIGURE 10.—*Reactions on each element of a simple house (south and east wind).*

(4) *Weight*

The weight of a wall is the outside face area times the weight of construction.

Although weight computed in this way closely approximates the actual weight of walls between openings, it is somewhat too great for walls intersecting at a corner projecting outward because the material in the corner is considered twice.

For frame walls, it is customary to place additional studs in corners, therefore, the computed weight approximates the actual weight.

For masonry walls, the error is greater the greater the wall thickness. For an 8-in. solid wall, the computed weight of a wall 3 ft wide having a corner along one edge is 11.1 percent too great, provided the error is equally divided between the intersecting walls. If the wall is 6 in. thick, the error is 8.3 percent, and if 3 in., only 4.2 percent.

If a corner projects inward, the computed weight of an intersecting wall is too small because none of the material in the corner is considered. The number of corners projecting outward is four more than the number projecting inward, and, most houses have few corners projecting inward.

If the weight of an 8-ft specimen is used for computing the weight of a wall having greater height, the value is a little too small, provided there are horizontal structural members in the construction such as sills, plates, or girts but the error is negligible for most constructions. On the other hand, for masonry constructions, the weight of the wall is proportional to the face area and the height; thus, the computed weight is correct.

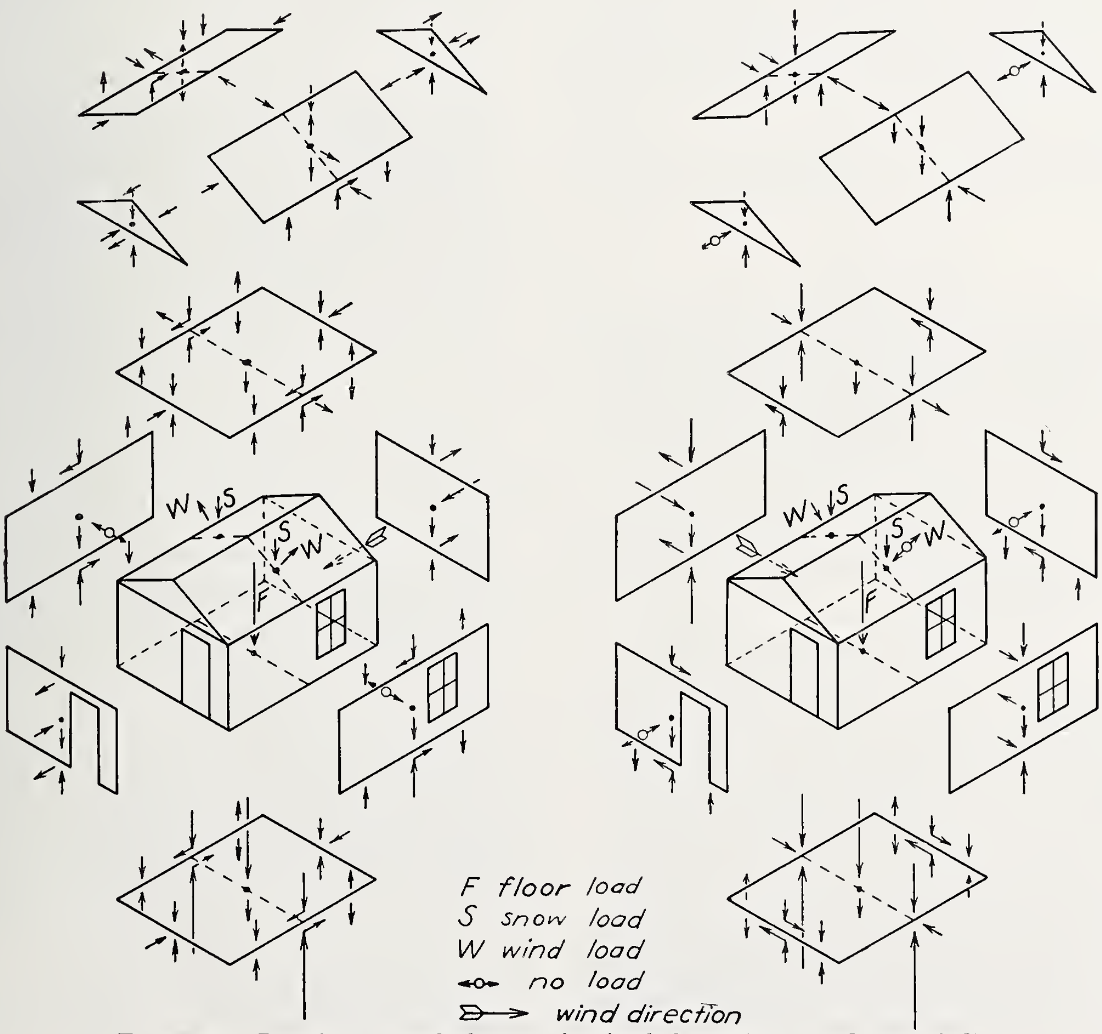

FIGURE 11.—*Reactions on each element of a simple house (north and west wind).*

In most houses, the structural members of the floor extend into the wall. In frame walls, they project into the spaces between studs, therefore, the weight of the wall is not affected appreciably. In solid masonry walls, there are spaces for the joists that decrease the weight of the wall at the floor but the effect on the weight of the wall is negligible, for an 8-in. wall only about 1.5 percent. This method of computing the weight of the wall probably is satisfactory, particularly as most of the errors are on the side of safety.

(5) Closures

The dimensions of doors and windows are the nominal height and width used commercially to designate the size. The weight may be obtained by placing the doors and sash on a platform scale or it may be taken from publications such as Building Material Data [25].

The frame for closure and additional members around the opening such as braces above the opening are disregarded because, to some extent, they replace wall materials and to estimate their effect on the weight of the wall presents complications that do not appear worth the effort.

For this report, the weight of all doors is taken as 4 lb/ft^2 and of all windows as 7 lb/ft^2.

(b) Floor

(1) Bearing

If the floor bearing is in a wall (or partition) it is taken as being one-third the thickness of the structural wall from the inside surface. Most floors do not extend through the wall and deflection of the floor under service loads causes the floor bearing to approach the inside surface of the wall.

If the floor is supported on a projection, as by a steel angle, the floor bearing is taken as being one-third the width of the projection from the inner edge.

(2) Dimensions and Weight

The span of a floor is the distance between floor bearings and the width is the distance between the structural walls on which the floor does not bear. The face area is the span times the width, and the weight of floor, the face area times the weight of construction.

The ceiling (if any), subfloor, and finish floor do not extend to the floor bearing in supporting walls and usually they do not extend to the other walls, so the computed weight is too great. However, the weight of the joists beyond the floor bearing is not included; therefore, it is probable that the computed weight approximates the actual weight.

(c) Roof

(1) Bearing

The roof bearing, as for a floor, is taken as being one-third the thickness of the structural wall from the inside surface.

(2) Dimensions and Weight

For a flat roof not extending beyond the walls, the dimensions and weight may be obtained as for a floor. If a flat roof extends beyond the walls, the length is the distance from end to end parallel to the structural members; the width is the distance from edge to edge normal to the members and the weight is the face area times the weight of construction.

For a sloping roof, the length is the distance from ridge to eaves parallel to the structural members, the width is the distance from edge to edge of the roof. The weight of each slope is the face area times the weight of construction.

X. REACTIONS ON ELEMENTS OF HOUSE

The reactions on each element of a simple house are shown in figures 10 and 11.

To simplify the presentation, the house is one-story 12 by 16 ft, walls 8 ft high, and the roof 30° slope. There is one door and one window. The dashed lines indicate the direction of the studs, joists, and rafters.

The weights of construction are taken as roof and attic floor 5 lb/ft^2; walls, gables, and floor 10 lb/ft^2. The velocity pressure is 30 lb/ft^2 on both wall and roof, basic snow load 25 lb/ft^2, and floor load 40 lb/ft^2.

The direction of wind is taken normal to each face in turn and, for each direction of wind, the window is closed and the door open; percentage of opening taken as 30 percent.

The assembled house with applied loads is shown for each direction of wind. They are the resultants of the uniformly distributed surface loads acting at the center of the surface.

Around the assembled house is each element with the weight, vertical and horizontal components of the wind load, if any, and all the reactions. The weight is the total weight of the element acting at the center of the face. The reactions at the midwidth of an edge are resultants of forces uniformly distributed along the edge. Horizontal racking forces along the top and bottom edges of walls cause a redistribution of the vertical forces along the bottom edge. After redistribution, they vary linearly from maximum at one vertical edge to minimum at the opposite vertical edge. In effect, the resultant at midwidth is combined with a couple of opposite sense to the racking couple and the resultant forces of this couple act at two-thirds the distance from midwidth to the vertical edge.

XI. ROOF

For both symmetrical and unsymmetrical sloping roofs of any slope, the design load and reactions at ridge and wall may be determined by the methods described here. The methods are best explained by computations for a particular house. Although analytical methods are described, it is obvious that graphical methods also are suitable and save time. Although it should be expected that errors in graphical results will be somewhat greater than those in analytical results, they are less than the errors in the assumed loads; therefore, entirely satisfactory for house design.

1. Sloping Roof With no Horizontal Reaction From Gable

If the gable is continuous with wall below, so that all wind load on gable is transmitted as reaction on attic floor and bending moment on wall below, the gable exerts no horizontal reaction on roof.

(*a*) *House With Unsymmetrical Sloping Roof*

For this explanation, house E in Madison, Wis., was selected, but the symmetrical roof is replaced by an unsymmetrical roof having slopes 25° and 50°, as shown in figure 12.

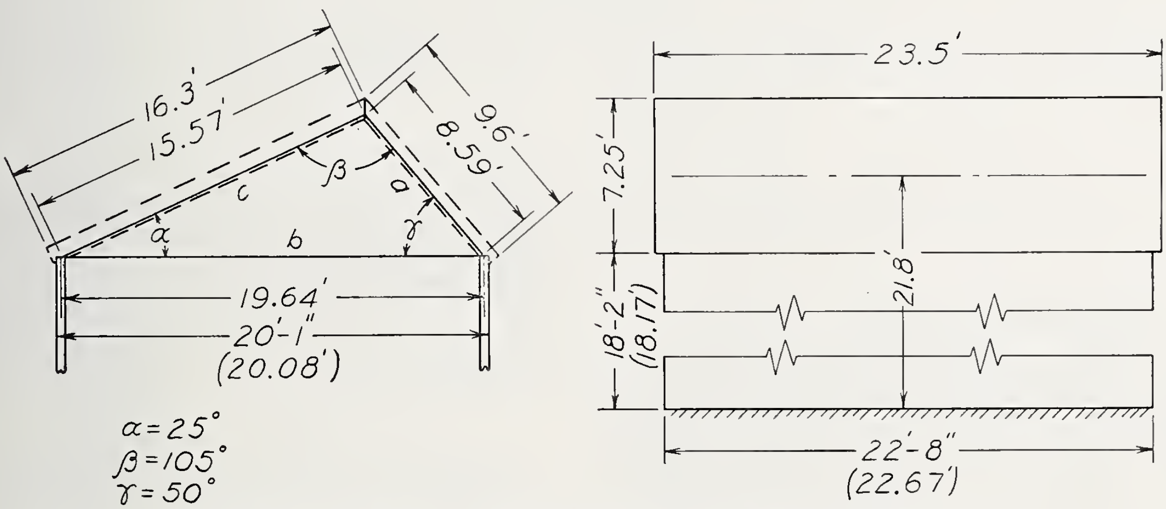

Figure 12.—*House E at Madison, Wis., with unsymmetrical sloping roof.*

It is assumed that walls are conventional wood-frame construction *QA* having 2- by 4-in. studs. The roof is construction *QC* having 2- by 6-in. rafters and weighs 4.6 lb/ft^2.

(*b*) *Dimensions*

For design, the dimensions of all structural members are taken as the *nominal* size.

The roof bearings on walls are one-third the distance from inside surface of stud to outside surface; therefore, the distance between bearings is 19.64 ft.

From the equation

$$\frac{a}{\sin \alpha} = \frac{b}{\sin \beta} = \frac{c}{\sin \gamma},$$

the slope spans are left slope 15.57 ft, right slope 8.59 ft. It is believed that spans computed in this way closely approximate the actual spans.

The dimensions measured on the plans or on a scale drawing are

		ft
Length of slope	left	16.3
	right	9.6
Width of slope		23.5
Height, roof bearing to ridge		7.25
Midheight of roof, above ground		21.8

(*c*) *Wind Load on Roof*

The height factor, *h*, is 0.9152 and velocity pressure on roof 18.3 lb/ft^2. The wind loads for each direction of wind and for windward and leeward openings are given in table 9.

Table 9.—*Wind loads on unsymmetrical sloping roof on House E in Madison, Wis.*

[Velocity pressure at midheight of roof 18.3 lb/ft². Leeward openings include openings in walls parallel to wind. Internal wind loads taken from table 4]

Roof	Left-hand slope, 25°								Right-hand slope, 50°								
	Velocity pressure, *q*	External wind load		Opening	Internal wind load		Combined wind load		Velocity pressure, *q*	External wind load		Opening	Internal wind load		Combined wind load	
	coef	*lb/ft²*	*Acts*		*lb/ft²*	*Acts*	*lb/ft²*	*Acts*	*coef*	*lb/ft²*	*Acts*		*lb/ft²*	*Acts*	*lb/ft²*	*Acts*
SOUTH WIND																	
Both slopes parallel to wind.	0.774	14.2	Out	Windward	8.7	Out	22.9	Out	0.774	14.2	Out	Windward	8.7	Out	22.9	Out
Do	.774	14.2	Do	Leeward	7.1	In	7.1	In	.774	14.2	Do	Leeward	7.1	In	7.1	In
EAST WIND																	
L H slope, leeward	0.581	10.6	Out	Windward	8.2	Out	18.8	Out	0.387	7.1	In	Windward	8.2	Out	1.1	Out
R H slope, windward	.581	10.6	Do	Leeward	7.1	In	3.5	Do	.387	7.1	Do	Leeward	7.1	In	14.2	In
NORTH WIND																	
Both slopes parallel to wind.	0.774	14.2	Out	Windward	8.7	Out	22.9	Out	0.774	14.2	Out	Windward	8.7	Out	22.9	Out
Do	.774	14.2	Do	Leeward	7.1	In	7.1	In	.774	14.2	Do	Leeward	7.1	In	7.1	In
WEST WIND																	
L H slope, windward	0.392	7.2	Out	Windward	8.2	Out	15.4	Out	0.581	10.6	Out	Windward	8.2	Out	18.8	Out
R H slope, leeward	.392	7.2	Do	Leeward	7.1	In	0.1	Do	.581	10.6	Do	Leeward	7.1	In	3.5	Do

(*d*) *Snow Load on Roof*

The snow loads are (basic snow load 13.0 lb/ft²)

Left slope .... ($K=0.79$) .....10.3 lb/ft².
Right slope ... ($K=0.16$) ..... 2.1 lb/ft².

For this explanation, if there is snow, it is assumed that both slopes are loaded. However, actually there may be snow on one slope but none on the other. The transverse load on a slope computed for snow on both slopes is correct if there is no snow on the other slope, but the longitudinal (tension or compression) load on loaded slope, and more important, the load on fastenings at ridge and wall may be much greater.

Obviously the condition for snow load on one slope and none on the other slope should be considered; otherwise the roof may fail under service conditions.

(*e*) *Loading Conditions*

All possible loading conditions are shown in the first columns of figures 13 and 14.

The factors are direction of wind, windward or leeward openings, and snow.

For any unsymmetrical sloping roof such as this, there are 12 conditions, including wind parallel to ridge and, from both directions, normal to ridge.

For any symmetrical sloping roof there are eight conditions, including wind parallel to ridge and from one direction normal to ridge. If there is no snow, there are only four conditions.

(*f*) *Components of Loads*

The vertical loads (weight and snow) are independent of the direction of wind. The sum of these loads for snow and no snow are resolved into components normal and parallel to slope. The values for this roof are shown on figure 15.

The normal component is combined with the wind load, also normal. The components for this roof under all conditions are shown in the second column, figures 13 and 14.

The greatest outward normal component is one of the design transverse loads for the slope and the greatest inward is the other design transverse load.

Each component is multiplied by length of slope, giving the total load per foot width of slope parallel to ridge. For the first condition these loads are:

	lb/ft
Left slope:	
Normal load	304.9
Parallel load	31.0
Right slope:	
Normal load	191.0
Parallel load	33.6

The normal reactions at ridge and wall are half the normal total load but the parallel reactions must be determined.

(*g*) *Reactions at Ridge*

The four concurrent reactions at ridge are shown on figure 16 for the first condition, wind parallel to ridge, windward openings, no snow.

It is convenient to choose axes normal to slopes.

$$F_x = 0 = b' \cos 15° - 89.55 \sin 15° - 152.45$$
$$b' = +181.9 \text{ lb/ft upward to left.}$$
$$F_y = 0 = 89.55 - 152.45 \sin 15° + b \cos 15°$$
$$b = +133.5 \text{ lb/ft upward to right.}$$

Negative values for these reactions would indicate that the assumed directions should be reversed.

(*h*) *Reactions at Wall*

The parallel reaction at wall is the combination of the parallel reaction at ridge and parallel load; in this case:

	lb/ft	
Left slope	108.9	downward to left
Right slope	149.9	downward to right

Obviously all parallel reactions are tensile or compressive loads on the slope and the value varies linearly along the slope from the value at one end to the value at the other end. If there is a tensile reaction at ridge less than the parallel load the longitudinal stress in the roof is zero somewhere between ridge and wall.

The greatest tensile reaction at either ridge or wall for any condition is the design tensile load and the greatest compressive reaction is the design compressive load for the slope of roof.

When designing bearing surfaces and fastenings, such as anchors or keys, it is convenient to know the vertical and horizontal reactions at ridge and wall. They may be computed from the normal and parallel reactions by the usual methods.

The values for this roof under all loading conditions are shown in the third column, figures 13 and 14.

(*i*) *Design Loads at Ridge and Wall*

At ridge, the greatest horizontal reaction acting outward is the design load in compression for the vertical bearing surface between slopes and the greatest value acting inward is the design load for anchors. The greatest vertical reaction, either upward or downward, is the design load for keys.

At wall, the greatest vertical reaction upward is the design load in compression for the wall and the greatest value downward is the design tensile load for wall and also the design load for anchors. The greatest horizontal reaction either inward or outward is the design load for keys.

(*j*) *Effect of Attic Floor on Reactions at Wall*

If there is no attic floor, the horizontal reactions on each slope of roof are exerted by the walls through keys.

If there is an attic floor and the horizontal reactions act in opposite directions, the floor is under a longitudinal load equal to the smaller of the reactions, tension if the reactions act outward, compression if they act inward. The difference (if any) between the reactions is transmitted by the floor as racking load to all the walls and load-bearing partitions normal to ridge.

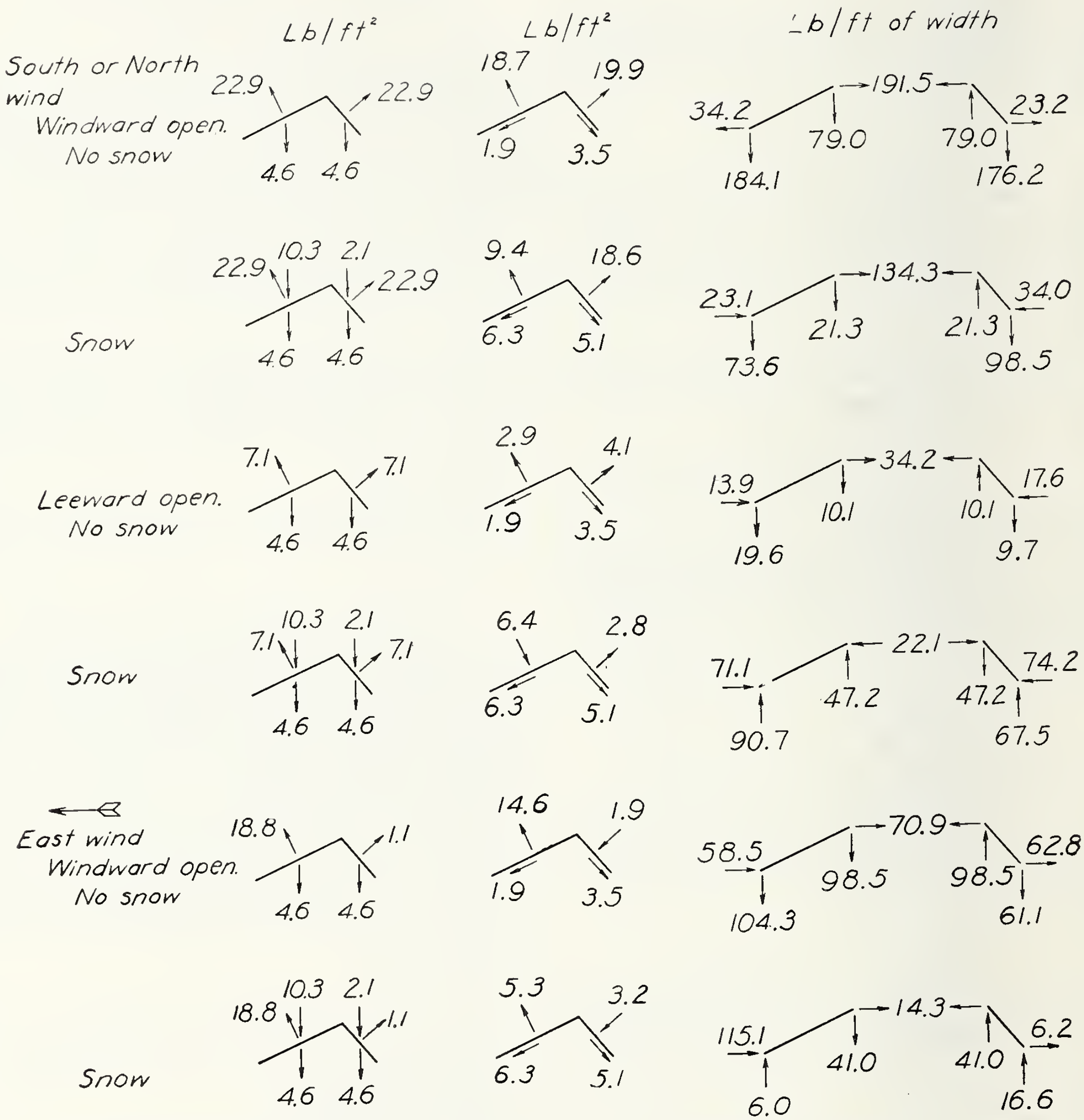

FIGURE 13.—*Loading conditions for unsymmetrical sloping roof on House E (north or south wind and part of east wind).*

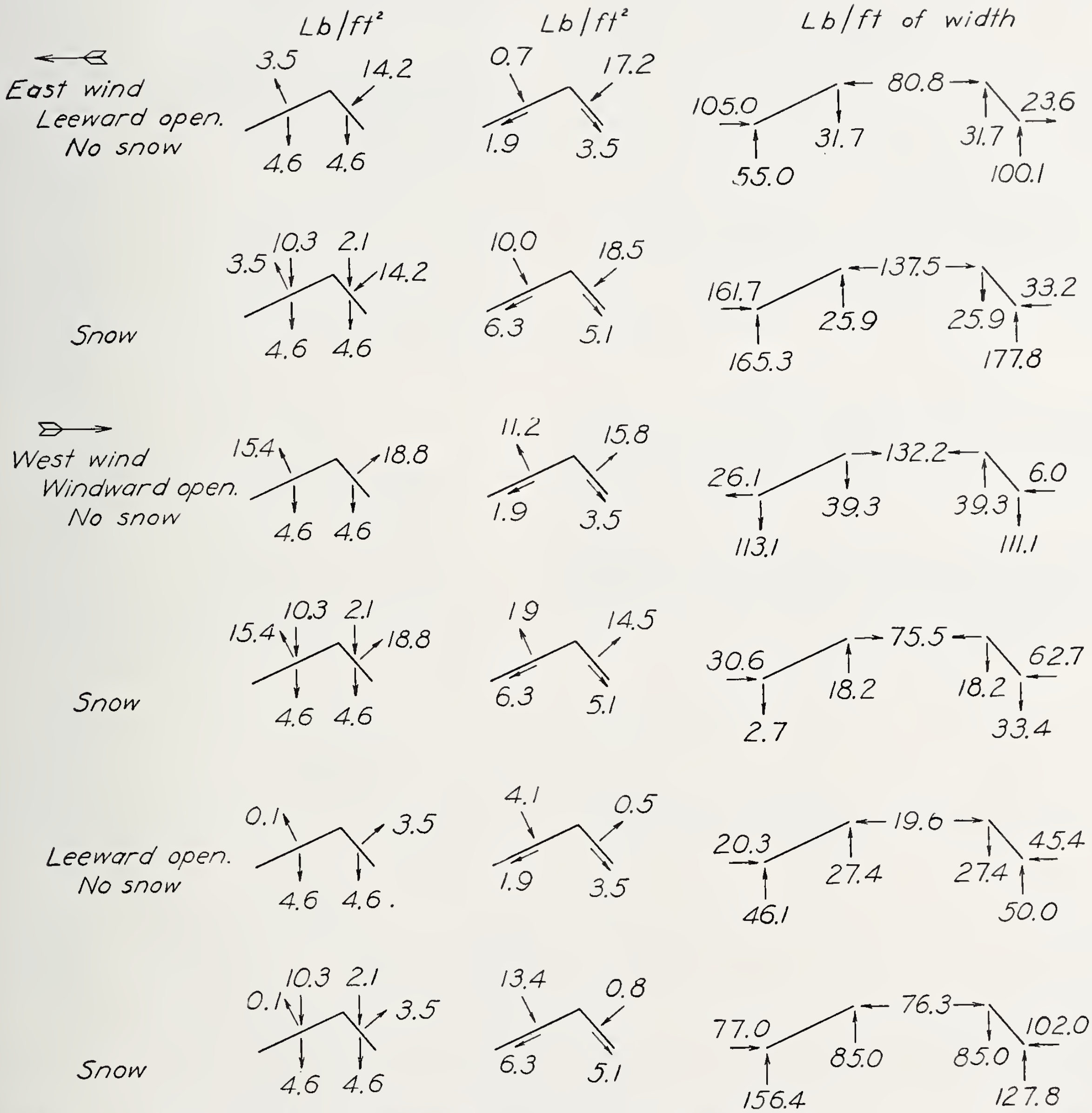

FIGURE 14.—*Loading conditions for unsymmetrical sloping roof on House E (part of east wind and west wind).*

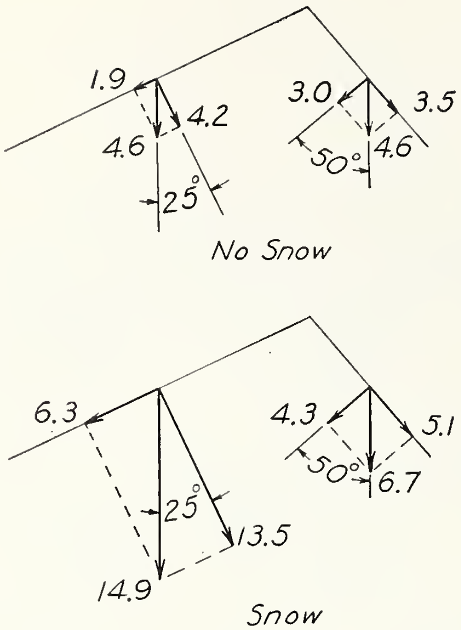

FIGURE 15.—*Normal and parallel components of weight and snow on unsymmetrical sloping roof.*

If the horizontal reactions act in the same direction, their sum is transmitted as racking load to walls and partitions.

2. Sloping Roof with Horizontal Reactions from Gable

If the gable is not continuous with wall below, the wind load on each gable is transmitted horizontally, half to the roof and half to attic floor, provided the structural members in the gable are vertical.

(a) *Wind Load on Gable*

The internal wind loads on gables are the same as those on roof given in table 4. All the wind loads on gables of this roof are given in table 10.

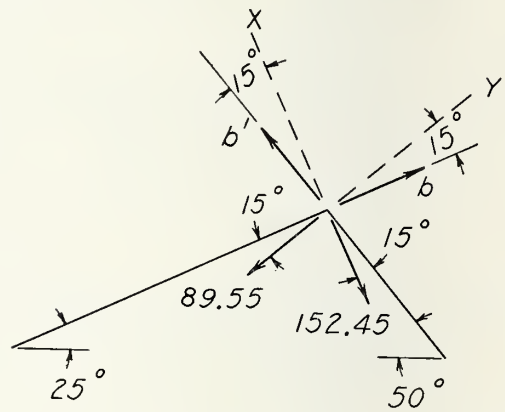

FIGURE 16.—*Reactions at ridge in unsymmetrical sloping roof.*

TABLE 10.—*Wind load on gables of House E in Madison, Wis.*

[Velocity pressure at midheight of roof 18.3 lb/ft². Leeward openings include openings in walls parallel to wind. Internal wind loads taken from table 4]

Gable	Velocity pressure, q	External wind load		Opening	Internal wind load		Combined wind load	
SOUTH WIND								
	coef	*lb/ft²*	*Acts*		*lb/ft²*	*Acts*	*lb/ft²*	*Acts*
Windward	0.8	14.64	In	Windward	8.7	Out	5.94	In
Do	.8	14.64	Do	Leeward	7.1	In	21.74	Do
Leeward	.5	9.15	Out	Windward	8.7	Out	17.85	Out
Do	.5	9.15	Do	Leeward	7.1	In	2.05	Do
EAST WIND								
Parallel to wind (both gables)	0.581	10.63	Out	Windward	8.2	Out	18.83	Out
Do	.581	10.63	Do	Leeward	7.1	In	3.53	Do
NORTH WIND								
Windward	0.8	14.64	In	Windward	8.7	Out	5.94	In
Do	.8	14.64	Do	Leeward	7.1	In	21.74	Do
Leeward	.5	9.15	Out	Windward	8.7	Out	17.85	Out
Do	.5	9.15	Do	Leeward	7.1	In	2.05	Do
WEST WIND								
Parallel to wind (both gables)	0.581	10.63	Out	Windward	8.2	Out	18.83	Out
Do	.581	10.63	Do	Leeward	7.1	In	3.53	Do

The greatest combined wind load acting out is one of the design transverse wind loads on gable and the greatest acting inward is the other design transverse load.

For wind parallel to ridge, both gables exert horizontal reactions on roof, windward gable inward and leeward gable outward, both to leeward, whether openings are in windward or leeward walls. The resultant of wind loads on both gables for any roof is independent of openings and always equal to 0.8 q+0.5 q, or 1.3 q. For this roof, it is 23.8 lb/ft², which is 1.3 q.

When designing a roof, the resultant of the gable reactions may be taken as applied at either the windward or leeward edge of the roof. If the roof is symmetrical and openings are in windward wall, probably there is tension in the roof along the ridge; if in leeward wall, there is compression. For this discussion, it will be assumed that half the resultant wind load on both gables is applied along windward edge of roof.

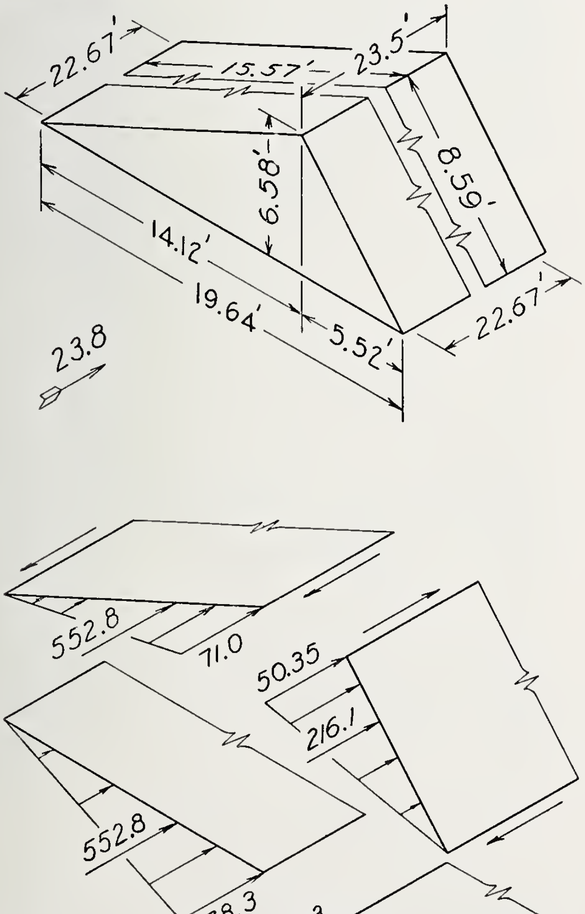

FIGURE 17.—*Distribution of racking load on roof.*

For wind normal to ridge (see table 9, east and west wind), it is evident that the wind loads on gables are always equal and both act outward; therefore, the only effect on roof is tension parallel to ridge, more for windward openings than for leeward.

(b) Distribution on Roof of Gable Reactions

For wind parallel to ridge, gable reactions on this roof are shown on figure 17.

Along the vertical line through ridge, wind load on gable is 23.8×6.58=156.6 lb/ft of width, on attic floor 78.3 lb/ft, on left slope 78.3×cos 25°=71 lb/ft, on right slope 78.3×cos 50°=50.35 lb/ft. The reactions on roof and floor decrease linearly from these values to zero at wall. The resultants for each slope and each portion of floor act at two-thirds the distance from wall. They are the average reaction times the span. The sum of these four resultants equals area of gable (64.6 ft²) times the wind load.

(c) Racking Load on Roof

The gable exerts racking loads on each slope of roof. In BMS structural reports are results for racking loads on walls but none on roofs. For determining design loads on roof, numerical values of deformation of roof under racking load are unnecessary if both slopes are the same construction, i.e., both slopes have the same racking modulus. The racking modulus for the construction must be known if the racking deformation is to be determined.

(1) Racking Modulus

Let *acde*, figure 18, be any house construction having length l and width b fixed along edge

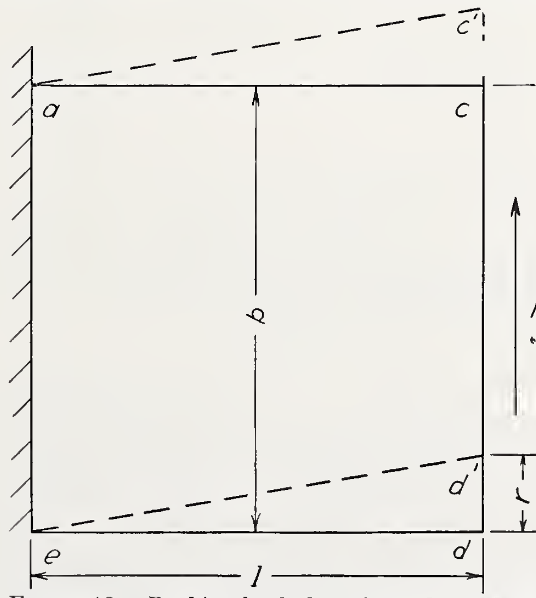

FIGURE 18.—*Racking load along free edge of house construction.*

ae and loaded along the opposite edge *cd* by the racking (shearing) force *F*.

The racking deformation *r* is directly proportional to force *F* and length *l* but inversely proportional to width *b*.

For *l* and *b* equal 1 foot the *racking modulus* *R* is the force, either in pounds or in kips, causing a deformation *r* of 1 foot computed from the inital rate of deformation.

For any values of *l* and *b*

$$r=\frac{F\,l}{b\,R} \text{ and } R=\frac{F\,l}{b\,r}.$$

The slope of ac' is $r/l=F/b\,R$.

If the racking force is applied anywhere between *ae* and *cd* as along *fg*, figure 19, the deformation of portion *afge* is

$$r_f=\frac{F\,af}{b\,R}.$$

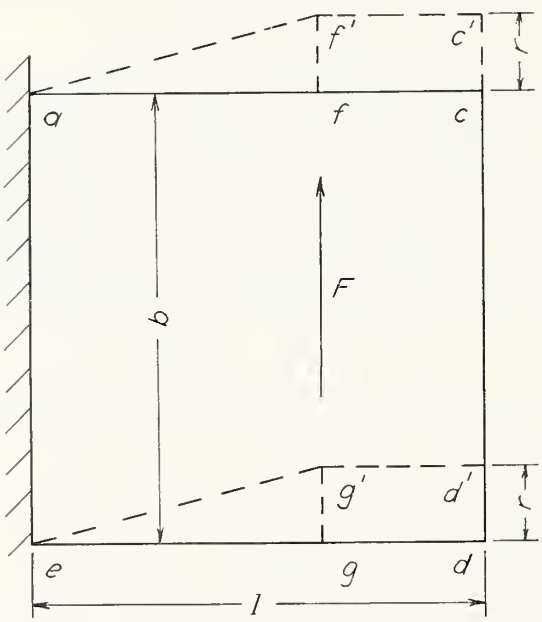

FIGURE 19.—*Racking load between fixed and free edge of house construction.*

There is no racking deformation of portion *fcdg* because there is no racking force on this portion. Therefore, the deformation at *c* is the same as the deformation at *f*.

(2) *Reactions Due to Racking Load*

If racking load is applied to a sloping roof there must be an equal and parallel reaction in the opposite direction. If the roof is symmetrical, this reaction cannot be applied at ridge by the other slope because the racking load on both slopes is the same and the racking deformations are the same. It is evident that this reaction must be exerted by the wall, the only other element in contact with the slope.

The racking load and wall reaction are a couple in the plane of the slope; therefore, there must be a couple having the same moment but opposite sign in the plane of the slope. If the angle between the slopes is 90°, this oppos-

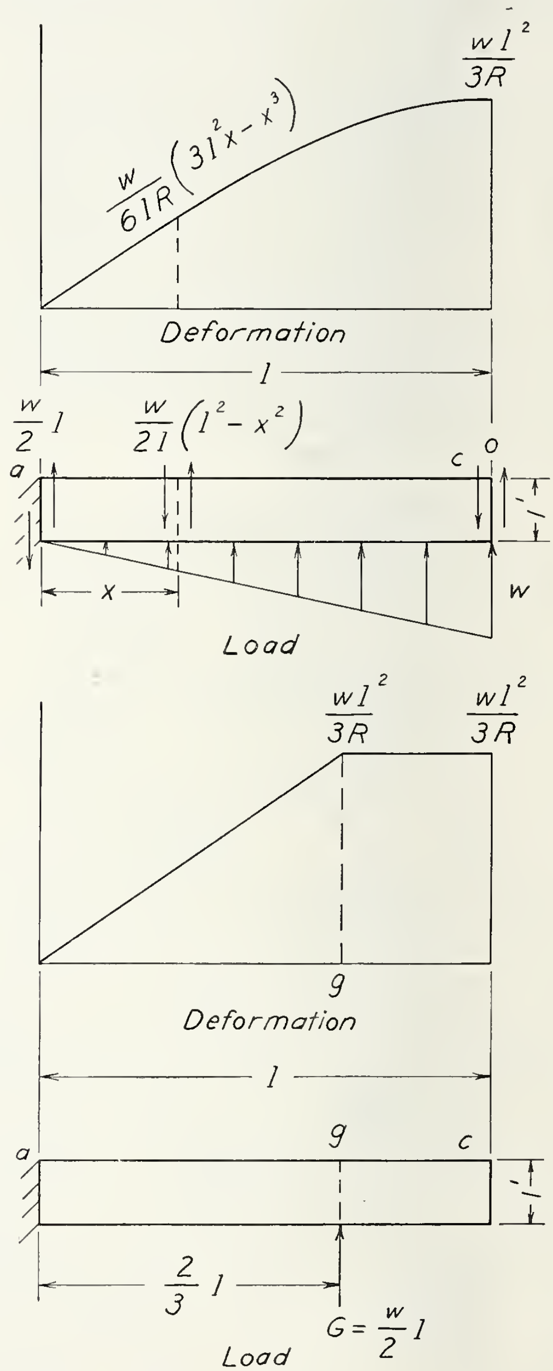

FIGURE 20.—*Deformation of house construction under racking load linearly distributed along edge normal to direction of load.*

ing couple could be exerted by the other slope only if it is warped, i.e., the ridge edge were forced out of parallel with the wall edge in plane normal to slope. But for roof constructions, the resistance to warping is negligible, even if the angle between slopes is not 90°; therefore, the opposing couple must be exerted by the wall.

For designing houses, it is assumed that the couple opposing the racking couple is in the plane of the slope and exerted by the wall.

For symmetrical sloping roofs having the same racking modulus for each slope, there is no longitudinal racking reaction between slopes at ridge, but for unsymmetrical sloping roofs, particularly if the slopes have different racking moduli, there may be a racking reaction at ridge because in a house the racking deformation of the slopes must be the same.

(*d*) *Deformation Under Distributed Racking Load*

Theorem: The racking deformation under a racking load distributed linearly along an edge normal to direction of load is the same as the deformation under the resultant of the distributed load.

Proof: Let *ac*, a construction 1 foot wide, fixed along the left edge, as shown in figure 20, have the racking modulus R.

The racking load at c is w pounds per foot of length and on any plane between a and c, such as x, is $\frac{x}{l}w$. The shearing forces on plane c are zero; on plane a, $\frac{w}{2}l$; on plane x, $\frac{w}{2l}(l^2-x^2)$.

The slope at x of the deformation curve is $\frac{F}{bR}=\frac{w}{2lR}(l^2-x^2)$ but this is the first derivative of the curve through the deformations. Integrating with respect to x, the deformation is $\frac{w}{6lR}(3l^2x-x^3)$, in feet. It follows that the deformation at a is zero, at c, $\frac{wl^2}{3R}$.

If the distributed racking load is replaced by the resultant $G=\frac{w}{2}l$, the deformation at g and also at c is

$$\frac{Fl}{bR}=\frac{G\times\frac{2}{3}\times l}{bR}=\frac{\frac{wl}{2}\times\frac{2l}{3}}{R}=\frac{wl^2}{3R}$$

Therefore, the racking deformation under any linearly distributed racking load may be computed by replacing the distributed load by the resultant racking load. *Q. E. D.*

If structural members in gable are horizontal, none of the wind load on gable is transmitted to attic floor (if any). The reaction on each slope is distributed linearly along the edge of roof from zero at ridge to maximum at wall. Half the wind load is transmitted to each slope whether the roof is symmetrical or unsymmetrical and the resultant reaction on each slope acts at two-thirds the span slope from ridge.

For symmetrical sloping roof, the racking deformation at resultant and at ridge is $wl^2/6R$, half the deformation for gables having vertical structural members.

(*e*) *Racking Load Transferred at Ridge in Unsymmetrical Sloping Roof*

For a symmetrical sloping roof, if the racking modulus is the same for both slopes, it is evident that the distributed racking loads and therefore deformations at ridge are the same for both slopes so there can be no longitudinal reaction at ridge. If the roof is unsymmetrical or the moduli different, there may be a longitudinal reaction necessary to make the deformations the same for both slopes.

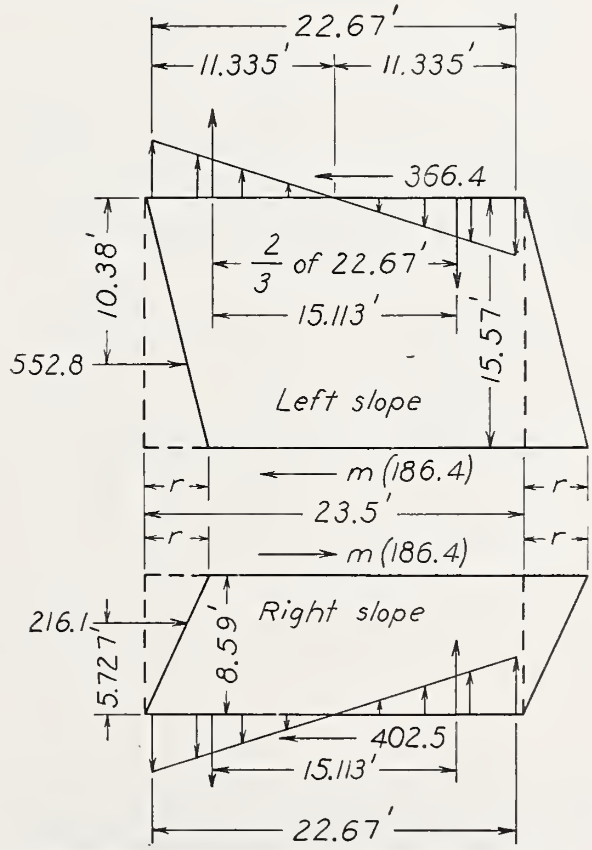

FIGURE 21.—*Transfer of racking load along ridge in unsymmetrical sloping roof.*

Both slopes of this roof are shown in the plane of the paper on figure 21.

Let m be the total longitudinal reaction at ridge.

The racking deformations at ridge are

$$\text{Left slope, } r_L = +\frac{552.8\times 10.38}{23.5R}-\frac{15.57m}{23.5R}$$

$$\text{Right slope, } r_R = +\frac{216.1\times 5.727}{23.5R}+\frac{8.59m}{23.5R}.$$

Because in a house r_L must equal r_R

$$552.8\times 10.38-15.57m=216.1\times 5.727+8.59m.$$
$m=186.4$ lb, or 7.93 lb/ft along ridge.

(*f*) *Reactions at Wall due to Racking Load*

The longitudinal reactions at walls are

Left wall...366.4 lb, or 16.18 lb/ft along wall
Right wall..402.5 lb, or 17.76 lb/ft along wall

Because the right slope is shorter, therefore stiffer under racking load, it transmits to right wall some of the racking load on the left slope.

The racking couples are

Left slope:
$+552.8\times 10.38-186.4\times 15.57=+2838.0$ ft-lb.

Right slope:
$-216.1\times 5.727-186.4\times 8.59=-2838.0$ ft-lb.

Assuming that the forces of these couples are normal to edge of slope along wall and are the resultants of the linearly distributed reactions shown on figure 21, the arm of both couples is 15.113 ft (two-thirds width of wall); therefore the resultants are

	lb
Left wall	187.8
Right wall	187.8

The average reactions are

	lb/ft
Left wall	16.57 along wall.
Right wall	16.57 along wall.

At the windward and leeward edges of wall the greatest reactions (twice the average) are

	lb/ft
Left wall	33.14 in plane of slope
Right wall	33.14 in plane of slope

Combining these reactions due to racking load with parallel reaction due to weight of roof and wind load on roof gives

Left wall:	*lb/ft*
Windward edge of wall	108.9+33.14=142.04.
Leeward edge of wall	108.9—33.14= 75.76.
Right wall:	
Windward edge of wall	149.9+33.14=183.04.
Leeward edge of wall	149.9—33.14=116.76.

The normal reactions have no components in plane of slope.

The vertical and horizontal reactions in plane normal to ridge are

Left wall:	Vertical *lb/ft*	Horizontal *lb/ft*
Windward edge	198.08 down	64.24 out.
Midwidth	184.1 down	34.2 out.
Leeward edge	170.17 down	4.16 out.
Right wall:		
Windward edge	201.56 down	44.5 out.
Midwidth	176.2 down	23.2 out.
Leeward edge	150.84 down	1.90 out.

For all conditions of loading with wind parallel to ridge (north or south wind), all the reactions at ridge and wall are shown on figure 22. All values are in pounds per foot along the edge of roof or wall.

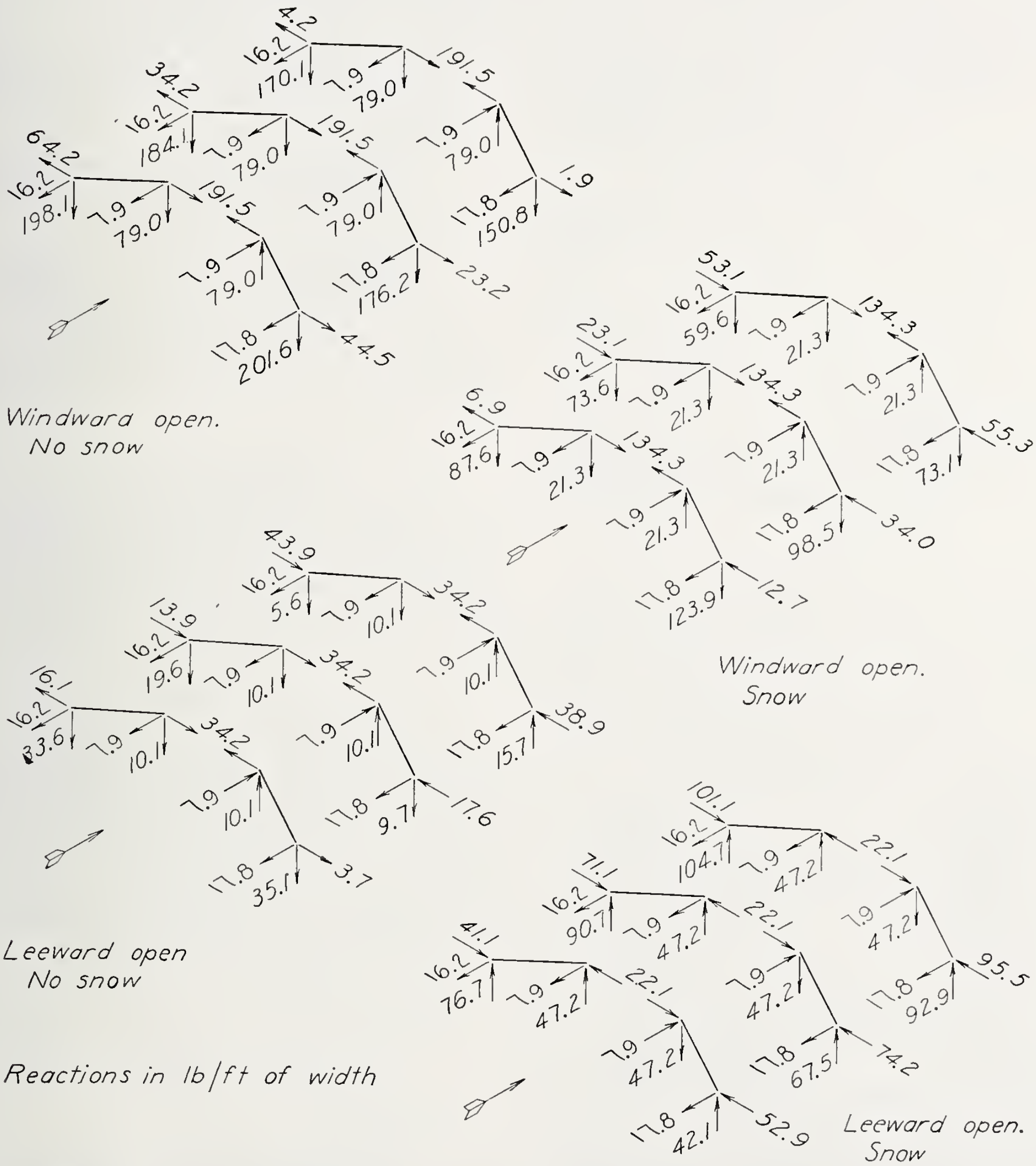

FIGURE 22.—*Reactions along ridge and wall in unsymmetrical sloping roof for ridge parallel to wind.*

(*g*) *Projection Method for Determining Vertical and Horizontal Reactions at Wall*

The vertical reactions at wall may readily be computed by considering the projection on a vertical plane through ridge of the resultant racking load due to wind on gable and longitudinal racking reaction at ridge (if any).

These reactions are shown in figure 23.

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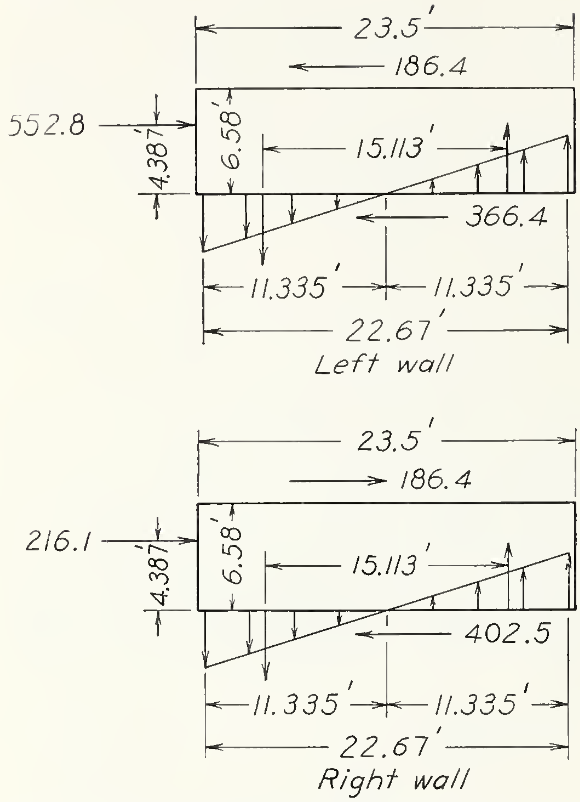

FIGURE 23.—*Racking loads in unsymmetrical sloping roof projected on vertical plane.*

The moment of racking couples are

	ft-lb
Left slope	—1199.0
Right slope	—2174.0

The resultant forces of opposing couple exerted by walls are

	lb
Left wall	79.26
Right wall	143.9

The greatest distributed reactions are

	lb/ft
Left wall	13.98 along wall.
Right wall	25.36 along wall.

These reactions due to wind load on gable may be combined with vertical reactions due to other loads.

The vertical reactions in plane normal to ridge due to all loads are

	lb/ft
Left wall:	
Windward edge	198.08 down.
Midwidth	184.1 down.
Leeward edge	170.12 down.
Right wall:	
Windward edge	201.56 down.
Midwidth	176.2 down.
Leeward edge	150.84 down.

The horizontal reactions due to wind load on gable may be determined by projecting the resultant racking loads on a horizontal plane; for this roof left wall 30.04 lb/ft, right wall 21.3 lb/ft.

For all loads, the horizontal reactions in plane normal to ridge are

	lb/ft
Left wall:	
Windward edge	64.24 out.
Midwidth	34.2 out.
Leeward edge	4.16 out.
Right wall:	
Windward edge	44.50 out.
Midwidth	23.2 out.
Leeward edge	1.90 out.

The projection method is convenient particularly if only the vertical reactions at wall are desired.

3. Projecting Roof

If there are projections from the house, such as the roof of a porch, supported along the wall and at outer edge, as by posts, the projection is designed in the same way as a roof, assuming that no bending moment is transmitted over wall.

(*a*) *Noncontinuous Projection*

If the projection is supported only along the wall, as for overhanging eaves, the design loads may be determined as described here. Consider House E having the unsymmetrical sloping roof but with eaves extending 6 ft from wall for both slopes. The velocity pressure on these overhanging eaves is taken as 16.2 lb/ft^2, the value at midheight of the second story. The wind loads on these projections are

	lb/ft²
Windward side of house—	
Slope 25°:	
Upper face (—0.387 q)	—6.27 up.
Lower face (—0.8 q)	—12.96 up.
Combined load (—1.187 q)	—19.23 up.
Slope 50°:	
Upper face (+0.387 q)	+6.27 down.
Lower face (—0.8 q)	—12.96 up.
Combined load (—0.413 q)	—6.69 up.
Leeward side of house—	
Both slopes:	
Combined load (—0.081 q)	—1.312 up.
Side of house parallel to wind—	
Both slopes:	
Combined load (—0.193 q)	—3.127 up.

For all directions of wind, the wind loads on these eaves act upward. The greatest values

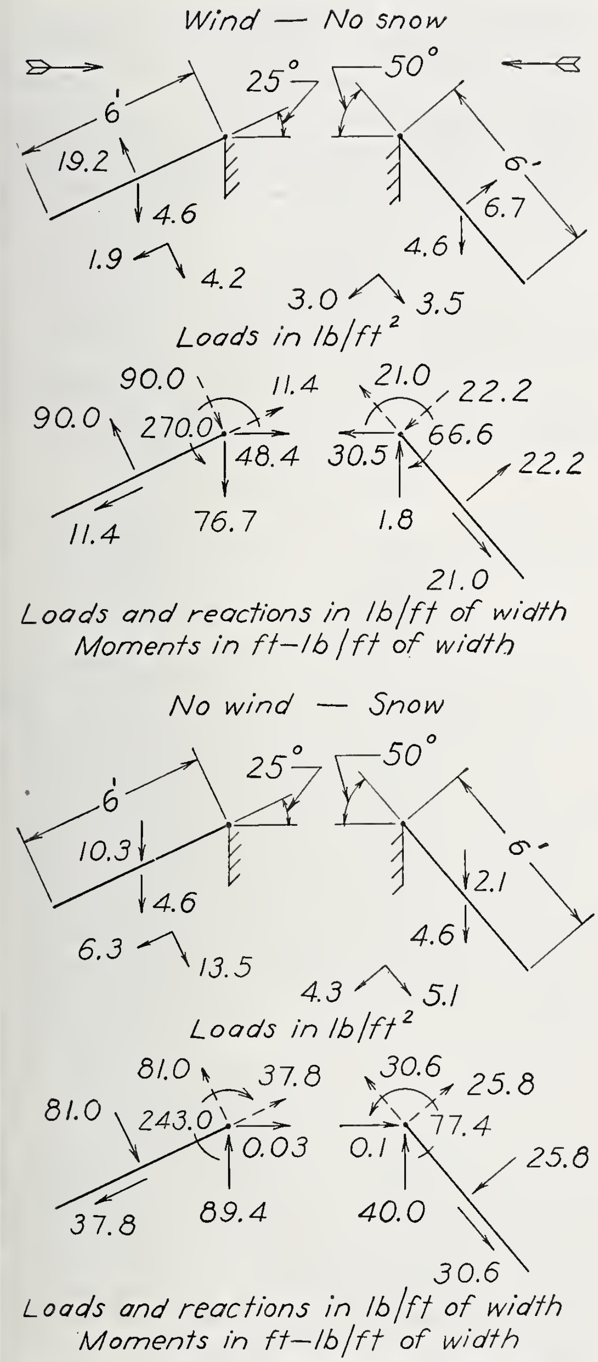

FIGURE 24.—*Some loads and reactions on non-continuous projecting roof.*

are for the projection on windward side of house. For design, the greatest load acting upward is the wind load on projection on windward side of house combined with the weight of projection. The greatest load acting downward is the snow load combined with the weight, no wind load. These conditions are shown on figure 24.

The reactions at wall are:

	Vertical *lb/ft*	Horizontal *lb/ft*	Moment *ft-lb/ft*
Wind, no snow:			
Slope 25°	76.7 down	48.4 in	270.0 counterclockwise
Slope 50°	1.8 up	30.5 in	66.6 clockwise
Snow, no wind:			
Slope 25°	89.4 up	0.03 in	243.0 clockwise
Slope 50°	40.0 up	0.1 out	77.4 counterclockwise

The moment decreases linearly from the value at wall to zero at outer edge of eave.

The greatest value in any direction is the design load, in that direction, for eave construction at wall and also for fastenings.

(*b*) *Continuous Projection*

In this roof the rafters in eaves are continuations of those in roof, notched over the outer edge of top plate in wall. The allowable bending moment at smallest section should be determined either by tests or by computation and compared with the design moments.

Provided the eave construction is a continuation of the roof construction and bending moment is transmitted past the wall, the effect of eave load on design transverse loads of roof between ridge and wall must be considered.

An interesting illustration of conditions that may occur in service is shown in figure 25. These are combinations of eave loads shown in figure 24 and corresponding roof load for windward openings on 25° slope, and leeward openings on 50° slope. Insofar as transverse loads are concerned, each slope is a simple beam overhanging one support having a uniformly distributed load between supports and a different uniformly distributed load on overhang.

For convenience, the left slope is shown upside down so that the distributed loads act downward. On right slope, the load between supports acts downward but that on overhang upward.

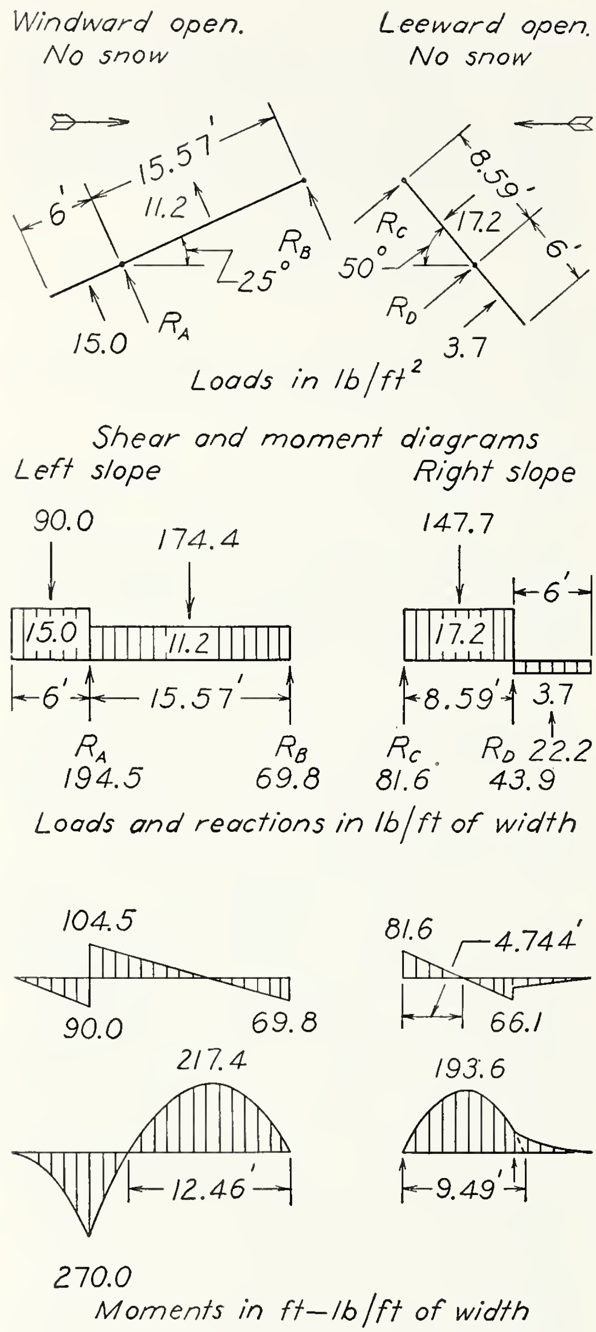

FIGURE 25.—*Some transverse loads and reactions on continuous projecting roof.*

For left slope, there are two design bending moments on roof between ridge and wall and they have opposite signs. One, the reverse moment over wall, is equal to the design moment for eave, and the other moment, that caused by the design transverse load between ridge and wall on an effective span equal to the distance from ridge to point of contraflexure. This effective span is shorter than the actual span.

For left slope the point of contraflexure between supports is where the moment is zero.

Let x be distance from ridge to point of contraflexure, then

$$M_x = 0 = +69.8x - 11.2x\ (x/2)$$
$$x = 12.46 \text{ ft.}$$

The second design transverse load therefore is 11.2 lb/ft on span 12.46 ft.

For right slope the moment is maximum where shear is zero. Let y be distance from ridge to point of maximum moment, then

$$V_y = 0 = 81.6 - 17.2y$$
$$y = 4.744 \text{ ft.}$$

and maximum moment between supports is

$$M_y = -81.6y + 17.2y\ (y/2)$$
$$M_y = 193.6 \text{ ft-lb.}$$

If there were no eave on right slope, the reactions at each support would be 73.88 lb and maximum moment 158.7 ft-lb, only 82 percent of maximum moment with eaves. It is evident that a roof construction between ridge and wall, safe under the transverse load 17.2 lb/ft² on span 8.59 ft, would be unsafe if there were an overhanging eave, provided for some condition of wind load the transverse load on roof acts in opposite direction to load on eave. For this condition, the design transverse load on roof between ridge and wall should be determined by finding the maximum moment, and the span producing this moment under the given transverse load.

For this roof, the design transverse load for roof between ridge and wall is 17.2 lb/ft² on a span of 9.49 ft, not on a span of 8.59 ft.

To summarize, if the construction in a projection is a continuation of the roof between ridge and eaves and the design transverse load on roof and projection act in the same direction, the design span for the roof is from ridge to point of contraflexure. If the design span is taken as the actual span, the roof will be safe but uneconomical.

If the design load on roof and projection act in opposite directions, the design span is greater than the actual span. It may be computed as the span causing the same maximum bending moment in a simple beam without projection as the maximum bending moment in the roof with projection. If the design span is taken as the actual span, the roof will be unsafe.

XII. HIP AND VALLEY RAFTERS

1. Auxiliary Roof Members

If the roof on a house slopes from three adjacent walls or if a house with sloping roofs has an ell, auxiliary members are required along the intersection of the two slopes to support the edges of the roofs. These members may be load-bearing partitions or other portions of the house. Usually, however, they are rafters (beams) extending from ridge to wall. If, at the rafter, the roof projects outward, the member is termed a "hip" rafter, if inward, a "valley" rafter. The method for computing the design load on either a hip or valley rafter is the same.

2. House With Hip Roof

To illustrate the method of determining design loads for hip rafters consider House E at Madison, Wis., which has the unsymmetrical roof shown in figure 12 combined with a roof sloping 40° on the south face only. This roof,

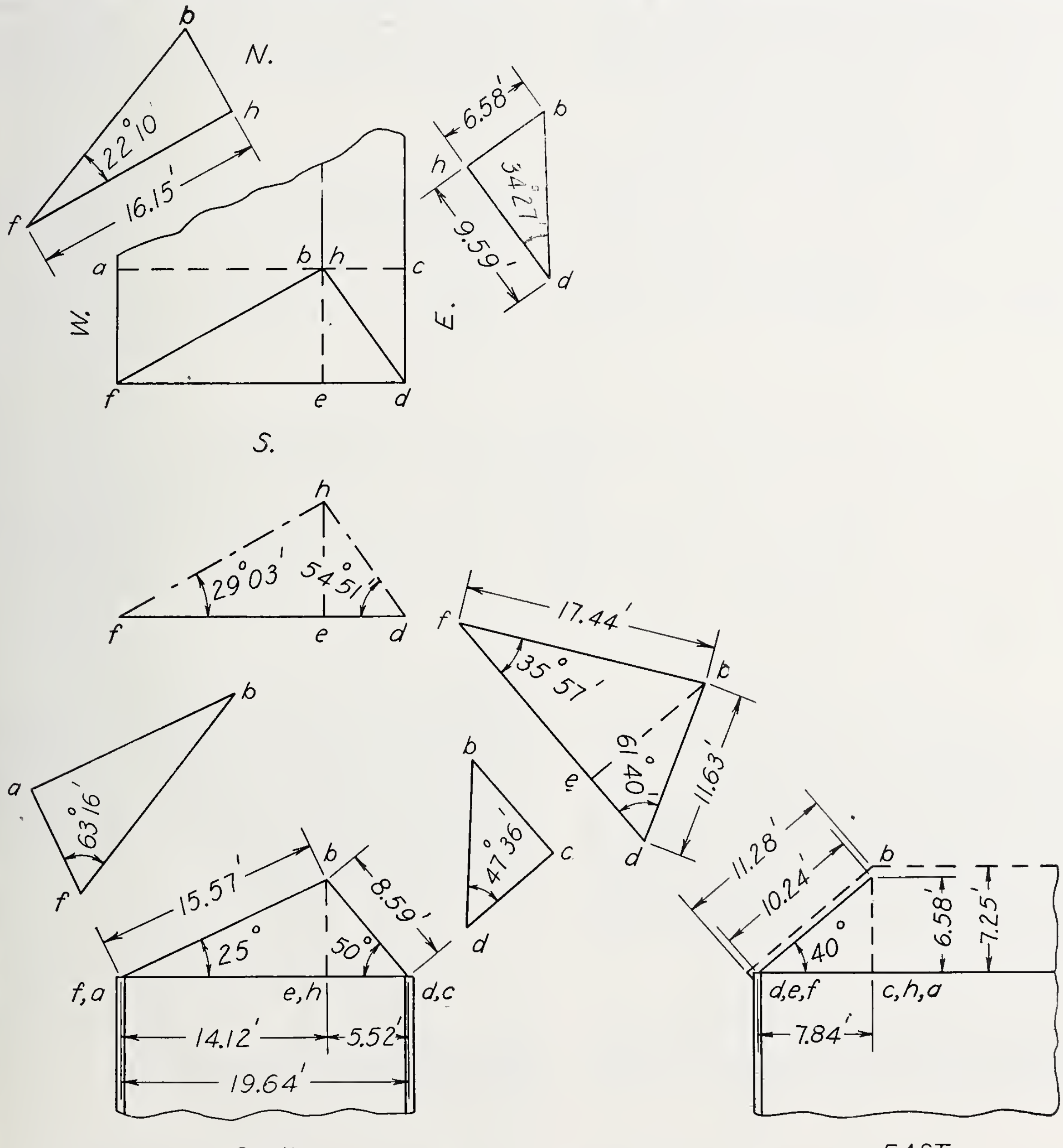

Figure 26.—*Roof of House E with unsymmetrical sloping roof and also sloping roof on south face.*

with dimensions, is shown in figure 26. The design loads for hip rafters *fb* and *db*, also design loads for fastenings, will be computed.

For the condition where there is no racking load on the east slope and west slope, due to wind load on gable ends, the horizontal and vertical reactions on the roof north of *c* in east wall and *a* in west wall may be taken from figures 13 and 14. These reactions are affected by the reaction at ridge of one slope upon the other slope.

The wall reactions south of points *c* and *a* must be determined from the reactions normal and parallel to the slope of roof; assuming, as for any sloping roof, that the parallel reaction is taken at the wall not at the hip rafter.

The loads and therefore the reactions are proportional to length of roof. For portion of roof *afb*, the length is greatest along *ab* and decreases linearly to zero at *f*. The reactions exerted by wall are greatest at *a* and decrease to zero at *f*. The reactions exerted by hip rafter are greatest at *b* and decrease to zero at *f*. Similarly the reactions on roof *fedb* are greatest at *e* on wall and *b* on hip rafter and decrease to zero at *f* and *d*. The reactions on portion of roof *dcb* are greatest at *c* on wall and *b* on hip rafter and decrease to zero at *d*.

For plane roofs, therefore, a hip rafter is a straight simple beam supported at ridge and wall and under zero load at wall with load increasing uniformly to ridge. A valley rafter also is a straight simple beam supported at ridge and wall but under zero load at ridge with load increasing uniformly to wall.

For both hip and valley rafters, the total load is the average load (half the greatest load) times the span of rafter. The reactions are one-third the total load at end where load is zero and two-thirds the total load where load is greatest.

3. Loads on South Roof

The south roof is construction *QC*, 2– by 6–in. rafters, weight 4.6 lb/ft^2; the same as east and west slopes.

The wind loads on south slope are given in table 11.

The snow load is 5.0 lb/ft^2 (basic snow load 13.0 lb/ft^2).

TABLE 11.—*Wind loads on south slope of roof on House E, Madison, Wis.*

[Slope of roof, 40°; velocity pressure at midheight of roof, 18.3 lb/ft^2; internal wind loads taken from table 4]

Direction of wind	External wind load		Internal wind load				Combined wind load			
			Windward opening		Leeward opening		Windward opening		Leeward opening	
	lb/ft^2	*Acts*	*lb/ft^2*	*Acts*	*lb/ft^2*	*Acts*	*lb/ft^2*	*Acts*	*lb/ft^2*	*Acts*
South	3.5	In	8.7	Out	7.1	In	5.2	Out	10.6	In
East	14.2	Out	8.2	Do	7.1	Do	22.4	Do	7.1	Out
North	10.6	Do	8.7	Do	7.1	Do	19.3	Do	3.5	Do
West	14.2	Do	8.2	Do	7.1	Do	22.4	Do	7.1	Do

4. Loads on Hip Roof

For each loading condition (direction of wind, opening, and snow) the load on each of the three roofs is given in table 12.

The loads (weight, wind, and snow) are resolved into components normal and parallel to roof slope and the combined components given under "unit load." The total loads, along lines *ab*, *eb*, and *cb*, are the unit loads times the length of roof and the values are in pounds per foot width of roof.

5. Reactions on Hip Roof

At wall, the normal reaction is half the total normal load and the parallel reaction all of parallel total load. At hip rafter, there is no parallel reaction and the normal reaction along the rafter is less than corresponding value for wall because length of rafter is greater than width of wall. The normal reaction at hip rafter is the normal reaction at wall times cosine of angle between rafter and wall; for this hip roof the angles are as follows:

Roof	Rafter	Angle
afb	*fb*	*afb* = 63° 16′
feb	*fb*	*efb* = 35° 57′
edb	*db*	*edb* = 61° 40′
dcb	*db*	*cdb* = 47° 36′

The reactions are given in table 13.

TABLE 12.—*Loads on hip roof*

[Hip roof is shown on figure 26. Values in bold-faced type are the greatest loads acting in and acting out]

Wind	Conditions	Slope *ab*							
		Unit load				Total load			
		Normal		Parallel		Normal		Parallel	
		lb/ft²	*Acts*	*lb/ft²*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:* No snow	18.7	Out	1.9	Down	**304.8**	Out	**31.0**	Down
	Snow	9.4	Do	6.3	Do	153.2	Do	**102.7**	Do
	Leeward openings: No snow	11.3	In	1.9	Do	184.2	In	31.0	Do
	Snow	20.6	Do	6.3	Do	**335.8**	Do	102.7	Do
East	*Windward openings:* No snow	14.6	Out	1.9	Do	238.0	Out	31.0	Do
	Snow	5.3	Do	6.3	Do	86.4	Do	102.7	Do
	Leeward openings: No snow	0.7	In	1.9	Do	11.4	In	31.0	Do
	Snow	10.0	Do	6.3	Do	163.0	Do	102.7	Do
North	*Windward openings:* No snow	18.7	Out	1.9	Do	**304.8**	Out	31.0	Do
	Snow	9.4	Do	6.3	Do	153.2	Do	102.7	Do
	Leeward openings: No snow	11.3	In	1.9	Do	184.2	In	31.0	Do
	Snow	20.6	Do	6.3	Do	**335.8**	Do	102.7	Do
West	*Windward openings:* No snow	11.2	Out	1.9	Do	182.6	Out	31.0	Do
	Snow	1.9	Do	6.3	Do	31.0	Do	102.7	Do
	Leeward openings: No snow	4.1	In	1.9	Do	66.8	In	31.0	Do
	Snow	13.4	Do	6.3	Do	218.4	Do	102.7	Do

Wind	Conditions	Slope *eb*							
		Unit load				Total load			
		Normal		Parallel		Normal		Parallel	
		lb/ft²	*Acts*	*lb/ft²*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:* No snow	1.7	Out	3.0	Down	19.2	Out	**33.8**	Down
	Snow	2.2	In	6.2	Do	24.8	In	**69.9**	Do
	Leeward openings: No snow	14.1	Do	3.0	Do	159.0	Do	33.8	Do
	Snow	18.0	Do	6.2	Do	**203.0**	Do	69.9	Do
East	*Windward openings:* No snow	18.9	Out	3.0	Do	**213.2**	Out	33.8	Do
	Snow	15.0	Do	6.2	Do	169.2	Do	69.9	Do
	Leeward openings: No snow	3.6	Do	3.0	Do	40.6	Do	33.8	Do
	Snow	0.3	In	6.2	Do	3.4	In	69.9	Do
North	*Windward openings:* No snow	15.8	Out	3.0	Do	178.2	Out	33.8	Do
	Snow	11.9	Do	6.2	Do	134.2	Do	69.9	Do
	Leeward openings: No snow	0.0	------	3.0	Do	0.0	------	33.8	Do
	Snow	3.9	In	6.2	Do	44.0	In	69.9	Do
West	*Windward openings:* No snow	18.9	Out	3.0	Do	**213.2**	Out	33.8	Do
	Snow	15.0	Do	6.2	Do	169.2	Do	69.9	Do
	Leeward openings: No snow	3.6	Do	3.0	Do	40.6	Do	33.8	Do
	Snow	0.3	In	6.2	Do	3.4	In	69.9	Do

Wind	Conditions	Slope *cb*							
		Unit load				Total load			
		Normal		Parallel		Normal		Parallel	
		lb/ft²	*Acts*	*lb/ft²*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:* No snow	19.9	Out	3.5	Down	**191.0**	Out	**33.6**	Down
	Snow	18.6	Do	5.1	Do	178.6	Do	**49.0**	Do
	Leeward openings: No snow	10.1	In	3.5	Do	**97.0**	In	33.6	Do
	Snow	11.4	Do	5.1	Do	109.4	Do	49.0	Do
East	*Windward openings:* No snow	1.9	Do	3.5	Do	18.2	Do	33.6	Do
	Snow	3.2	Do	5.1	Do	30.7	Do	49.0	Do
	Leeward openings: No snow	17.2	Do	3.5	Do	**165.1**	Do	33.6	Do
	Snow	18.5	Do	5.1	Do	**177.6**	Do	49.0	Do
North	*Windward openings:* No snow	19.9	Out	3.5	Do	**191.0**	Out	33.6	Do
	Snow	18.6	Do	5.1	Do	178.6	Do	49.0	Do
	Leeward openings: No snow	10.1	In	3.5	Do	97.0	In	33.6	Do
	Snow	11.4	Do	5.1	Do	109.4	Do	49.0	Do
West	*Windward openings:* No snow	15.8	Out	3.5	Do	151.7	Out	33.6	Do
	Snow	14.5	Do	5.1	Do	139.2	Do	49.0	Do
	Leeward openings: No snow	0.5	Do	3.5	Do	4.8	Do	33.6	Do
	Snow	.8	In	5.1	Do	7.7	In	49.0	Do

TABLE 13.—*Reactions on hip roof*

[Hip roof is shown on figure 26. Values in bold-faced type are the greatest reactions acting in and acting out]

Wind	Conditions	Slope *ab*					
		At wall *af*				At hip rafter *fb*	
		Normal		Parallel		Normal	
		lb/ft	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:* No snow	**152.4**	In	31.0	Up	**68.6**	In
	Snow	76.6	Do	**102.7**	Do	34.5	Do
	Leeward openings: No snow	92.1	Out	31.0	Do	41.4	Out
	Snow	**167.9**	Do	102.7	Do	**75.5**	Do
East	*Windward openings:* No snow	119.0	In	31.0	Do	53.5	In
	Snow	43.2	Do	102.7	Do	19.4	Do
	Leeward openings: No snow	5.7	Out	31.0	Do	2.6	Out
	Snow	81.5	Do	102.7	Do	36.7	Do
North	*Windward openings:* No snow	**152.4**	In	31.0	Do	**68.6**	In
	Snow	76.6	Do	102.7	Do	34.5	Do
	Leeward openings: No snow	92.1	Out	31.0	Do	41.4	Out
	Snow	**167.9**	Do	102.7	Do	**75.5**	Do
West	*Windward openings:* No snow	91.3	In	31.0	Do	41.1	In
	Snow	15.5	Do	102.7	Do	7.0	Do
	Leeward openings: No snow	33.4	Out	31.0	Do	15.0	Out
	Snow	109.2	Do	102.7	Do	49.1	Do

Wind	Conditions	Slope *eb*							
		At wall *fed*				At hip rafter *fb*		At hip rafter *db*	
		Normal		Parallel		Normal		Normal	
		lb/ft	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:* No snow	9.6	In	33.8	Up	7.8	In	4.6	In
	Snow	12.4	Out	**69.9**	Do	10.0	Out	5.9	Out
	Leeward openings: No snow	79.5	Do	33.8	Do	64.4	Do	37.7	Do
	Snow	**101.5**	Do	69.9	Do	**82.2**	Do	**48.2**	Do
East	*Windward openings:* No snow	**106.6**	In	33.8	Do	**86.3**	In	**50.6**	In
	Snow	84.6	Do	69.9	Do	68.5	Do	40.2	Do
	Leeward openings: No snow	20.3	Do	33.8	Do	16.4	Do	9.6	Do
	Snow	1.7	Out	69.9	Do	1.4	Out	0.8	Out
North	*Windward openings:* No snow	89.1	In	33.8	Do	72.1	In	42.3	In
	Snow	67.1	Do	69.9	Do	54.3	Do	31.8	Do
	Leeward openings: No snow	0.0	------	33.8	Do	0.0	------	0.0	------
	Snow	22.0	Out	69.9	Do	17.8	Out	10.4	Out
West	*Windward openings:* No snow	**106.6**	In	33.8	Do	**86.3**	In	**50.6**	In
	Snow	84.6	Do	69.9	Do	68.5	Do	40.2	Do
	Leeward openings: No snow	20.3	Do	33.8	Do	16.4	Do	9.6	Do
	Snow	1.7	Out	69.9	Do	1.4	Out	0.8	Out

Wind	Conditions	Slope *cb*					
		At wall *dc*				At hip rafter *db*	
		Normal		Parallel		Normal	
		lb/ft	*Act*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:* No snow	**95.5**	Ins	33.6	Up	**64.4**	In
	Snow	89.3	Do	**49.0**	Do	60.2	Do
	Leeward openings: No snow	48.5	Out	33.6	Do	32.7	Out
	Snow	54.7	Do	49.0	Do	36.9	Do
East	*Windward openings:* No snow	9.1	Do	33.6	Do	6.1	Do
	Snow	15.4	Do	49.0	Do	10.4	Do
	Leeward openings: No snow	82.6	Do	33.6	Do	55.7	Do
	Snow	**88.8**	Do	49.0	Do	**59.9**	Do
North	*Windward openings:* No snow	**95.5**	In	33.6	Do	**64.4**	In
	Snow	89.3	Do	49.0	Do	60.2	Do
	Leeward openings: No snow	48.5	Out	33.6	Do	32.7	Out
	Snow	54.7	Do	49.0	Do	36.9	Do
West	*Windward openings:* No snow	75.8	In	33.6	Do	51.1	In
	Snow	69.6	Do	49.0	Do	46.9	Do
	Leeward openings: No snow	2.4	Do	33.6	Do	1.6	Do
	Snow	3.8	Out	49.0	Do	2.6	Out

6. Loads on Wall

The load on wall is equal to reaction on roof but in opposite direction. The horizontal and vertical wall loads for south wind, windward opening, no snow are shown in figure 27.

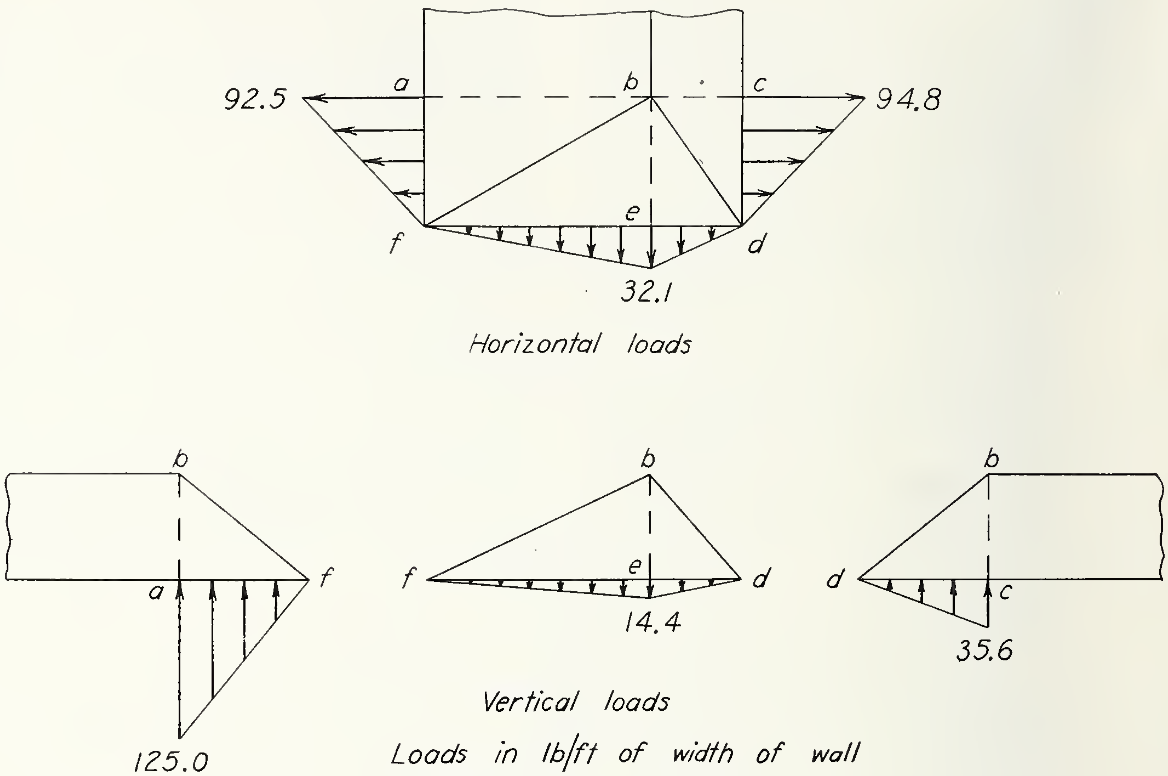

Figure 27.—*Loads on walls.*

The design loads for a wall are the greatest values for any condition of loading: out and in, also, up and down.

The normal and parallel loads on wall do not have the same ratio for all loading conditions but the design loads on a wall may be determined by selecting a few loading conditions for which the normal and parallel loads are large and computing the vertical and horizontal loads for each. The greatest of these values are the design loads for the wall.

7. Loads on Hip Rafter

The loads exerted on a hip rafter by the roofs on both sides through the fastenings are the only loads on the hip rafter. For designing the rafter, either by the usual engineering methods or from the results of laboratory tests, it is essential that the roof load normal to the plane of the roof be resolved into components along the three principal axes of the rafter—the longitudinal axis, P, and two axes normal to this axis; one, the axis for greatest section modulus, N_V, and the other, the axis for least section modulus, N_H.

The determination of roof loads on a hip rafter is greatly simplified by the fact that rafters always are placed with the plane for greatest section modulus vertical. If the hip rafter is wood, 2 by 8 inches, the 8-inch faces always are in a vertical plane.

The method for resolving the normal roof load into components along the principal axes of the hip rafter will be illustrated for west roof *afb* on rafter *fb* for south wind, windward openings, no snow. This roof is shown in figure 28.

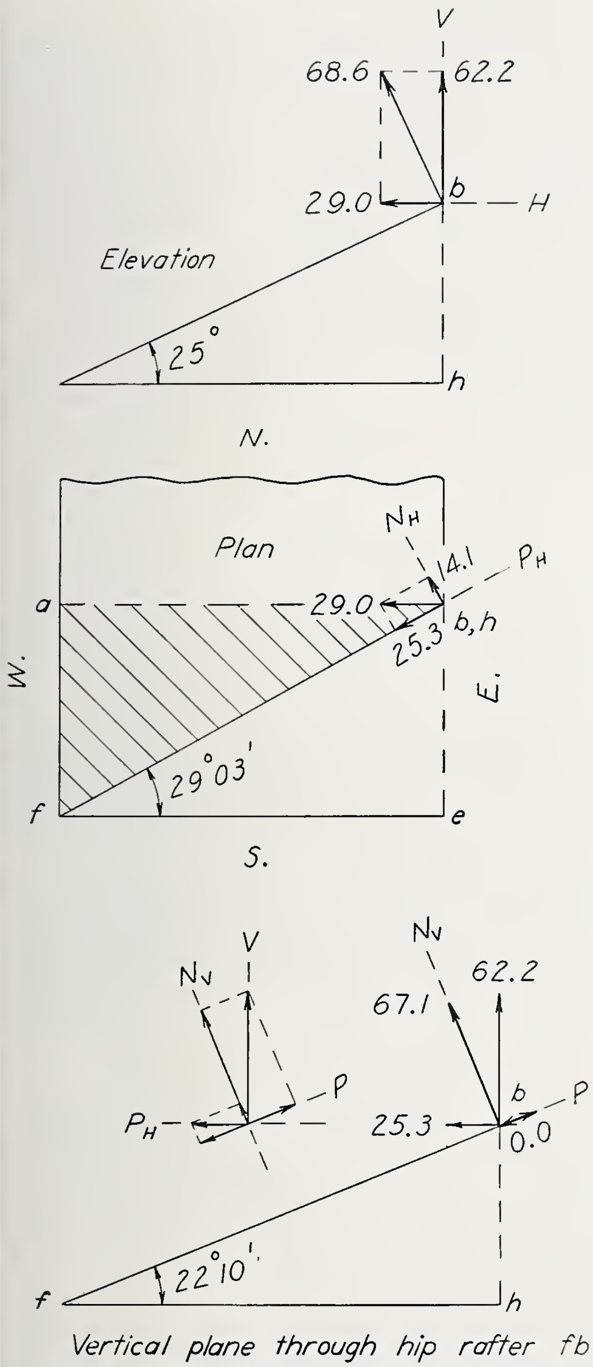

FIGURE 28.—*West roof afb, south wind, windward openings, no snow.*

Consider the load exerted by roof on hip rafter as shown in the elevation. From table 13, the load on rafter *fb* at *b* is 68.6 lb/ft along the rafter acting out and it lies in plane *ahb*. The vertical component is 62.2 lb/ft acting up and the horizontal component is 29.0 lb/ft acting to the west.

In the plan, for a horizontal plane through the ridge, take axis P_H in the vertical plane through the longitudinal axis of the rafter and N_H normal to P_H. N_H is the axis for least section modulus. The components of 29.0 lb/ft are, P_H, 25.3 lb/ft acting to southwest and N_H, 14.1 lb/ft acting to northwest.

In the vertical plane through the hip rafter, the components are *P*, 0.0, N_V, 67.1 lb/ft acting out.

For this loading condition, therefore, the loads exerted on rafter *fb* at *b* by roof *afb* are: parallel to longitudinal axis of rafter, *P*, 0.0; along axis for greatest section modulus, N_V, 67.1 lb/ft acting out, normal to rafter; along axis for least section modulus, N_H, 14.1 lb/ft acting to northwest, normal to rafter. Whether the longitudinal, *P*, component of the roof load is zero for all hip roofs is not known.

For roofs *afb*, *fdb*, and *dcb* the components of the roof loads along the principal axes of the hip rafters for all loading conditions are given in table 14.

The values in bold-faced type are the greatest values acting in opposite directions. If the roof loads are computed for *one* loading condition, the loads for any other condition are in the same ratio to the computed loads as the ratio of the normal reactions on the rafter. The direction of the load can be determined by inspection. A table similar to table 14 can be made readily by computing the rafter loads for one loading condition for each roof, then using the ratios of the reactions to obtain the loads for all the other loading conditions.

Table 14.—*Roof loads on hip rafters*

[The loads are along principal axes of rafter. See figure 28. *P*, longitudinal axis of rafter (for all conditions, loads along this axis are zero). NV, axis for greatest section modulus. N_H, axis for least section modulus. Values in bold-faced type are the greatest roof loads acting in opposite directions]

Wind	Conditions	Roof *afb;* rafter *fb*				Roof *fdb*								Roof *dcb;* rafter *db*			
						Rafter *fb*				Rafter *db*							
		N_V		N_H		N_V		N_H		N_V		N_H		N_V		N_H	
		lb/ft	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*	*lb/ft*	*Acts*
South	*Windward openings:*																
	No snow	**67.1**	Out	**14.1**	NW	6.5	Out	4.4	SE	4.3	Out	1.7	SW	**50.2**	Out	**40.3**	NE
	Snow	33.7	Do	7.1	NW	8.3	In	5.6	NW	5.5	In	2.2	NE	46.9	Do	37.7	NE
	Leeword openings:																
	No snow	40.5	In	8.5	SE	53.3	Do	36.2	NW	35.0	Do	13.9	NE	25.5	In	20.5	SW
	Snow	**73.9**	Do	**15.5**	SE	**68.0**	Do	**46.2**	NW	**44.8**	Do	**17.8**	NE	28.8	Do	23.1	SW
East	*Windward openings:*																
	No snow	52.3	Out	11.0	NW	**71.4**	Out	**48.5**	SE	**47.0**	Out	**18.7**	SW	4.8	Do	3.8	SW
	Snow	19.0	Do	4.0	NW	56.7	Do	38.5	SE	37.4	Do	14.8	SW	8.1	Do	6.5	SW
	Leeword openings:																
	No snow	2.5	In	0.5	SE	13.6	Do	9.2	SE	8.9	Do	3.5	SW	43.4	Do	34.9	SW
	Snow	35.9	Do	7.5	SE	1.2	In	0.8	NW	0.7	In	0.3	NE	**46.7**	Do	**37.5**	SW
North	*Windword openings:*																
	No snow	**67.1**	Out	**14.1**	NW	59.6	Out	40.5	SE	39.3	Out	15.6	SW	**50.2**	Out	**40.3**	NE
	Snow	33.7	Do	7.1	NW	44.9	Do	30.5	SE	29.6	Do	11.7	SW	46.9	Do	37.7	NE
	Leeward openings:																
	No snow	40.5	In	8.5	SE	0.0	-------	0.0	-------	0.0	-------	0.0	-------	25.5	In	20.5	SW
	Snow	**73.9**	Do	**15.5**	SE	14.7	In	10.0	NW	9.7	In	3.8	NE	28.8	Do	23.1	SW
West	*Windward openings:*																
	No snow	40.2	Out	8.4	NW	**71.4**	Out	**48.5**	SE	**47.0**	Out	**18.7**	SW	39.8	Out	32.0	NE
	Snow	6.8	Do	1.4	NW	56.7	Do	38.5	SE	37.4	Do	14.8	SW	36.6	Do	29.3	NE
	Leeward openings:																
	No snow	14.7	In	3.1	SE	13.6	Do	9.2	SE	8.9	Do	3.5	SW	1.2	Do	1.0	NE
	Snow	48.0	Do	10.1	SE	1.2	In	0.8	NW	0.7	In	0.3	NE	2.0	In	1.6	SW

8. Loads on Fastenings

The loads exerted on the fastenings, whether anchors or keys, are the same as the loads exerted by the roof on the walls and hip rafters.

The design loads, then, for fastenings will be the same as the design loads for walls and hip rafters.

9. Combined Loads on Hip Rafter

For each loading condition, the combined load on rafter at ridge in pounds per foot along the rafter is the resultant along each of the principal axes of the rafter of the loads exerted by the roof on each side.

The greatest combined loads on hip rafter *fb* are N_V, 126.7 lb/ft acting out and 141.9 lb/ft acting in; N_H, 30.7 lb/ft acting toward the northwest and 40.1 lb/ft toward the southeast. Those on rafter *db* are N_V, 89.5 lb/ft acting out and 73.6 lb/ft acting in; N_H, 39.9 lb/ft acting toward the northeast and 38.4 lb/ft toward the southwest.

10. Total Loads on Hip Rafter

Rectangular wood rafters usually have the same strength out as in and the same strength laterally either way; therefore, it is necessary to consider only the greatest load edgewise and the greatest flatwise. For both these hip rafters, the axial load is zero for all loading conditions.

The design loads for these hip rafters are:

Rafter *fb*, length 17.44 ft, edgewise (N_V) total load 1237.4 lb acting inward, reaction at wall 412.5 lb acting outward, reaction at ridge 824.9 lb acting outward; flatwise (N_H) total load 349.7 lb (SE), reaction at wall 116.6 lb (NW), reaction at ridge 233.1 lb (NW).

Rafter *db*, length 11.63 ft, edgewise (N_V) total load 520.4 lb acting outward, reaction at wall 173.5 lb acting inward, reaction at ridge 346.9 lb acting inward; flatwise (N_H) total load 232.0 lb (NE), reaction at wall 77.3 lb (SW), reaction at ridge 154.7 lb (SW).

These reactions are normal to rafter. The load on wall exerted by a hip rafter is the same magnitude as wall reaction on rafter but in opposite direction and is a "point" load. For designing the wall, this load may be resolved into components vertical and along the width of each of the two walls intersecting at the corner of the house.

Likewise, the load exerted by a hip rafter on the ridge is the same magnitude as ridge reaction on rafter but in opposite direction and is a point load. For designing the roof, the loads exerted by the two hip rafters may be resolved into components along the ridge and normal to ridge horizontally and vertically.

The total load exerted by the two hip rafters, then, is the combined components along the three axes. The load along the ridge, combined with any other loads on the roof acting parallel to ridge is a racking load on each slope intersecting at ridge.

The combined loads normal to ridge, horizontally and vertically, cause either tensile or compressive loads in the roofs along lines *ab* and *cb* because the roofs along these lines are two-force elements. Practically, these axial loads in the roof may be so small that no special members need be put in the roof to carry them. However, consideration should be given to special rafters to take the point loads at ridge (*b*) and the point reactions at wall (*a* and *c*).

For this assumed distribution of the ridge reactions on the hip rafters, there can be no racking in the triangular portions of the east slope (*dcb*) and west slope (*afb*).

XIII. WALLS

1. Design Loads

For compressive load and for racking load, the design load is in kips per linear foot along top of wall; for transverse load, in pounds per square foot uniformly distributed on a span equal to story height of wall.

For concentrated load, the design load is in pounds on the disk, one inch in diameter; for impact load, in feet height-of-drop of the 60-pound sand bag.

The dimensions of the house and the geographical location have no bearing on the design concentrated load nor on the design impact load. They depend only upon the occupancy and are the values which, in the opinion of the designer, provide reasonable insurance against local damage.

2. Walls Which Have no Openings

If a wall has neither doors nor windows the design loads are:

(*a*) *Compressive Load*

The design compressive load is the greatest vertical reaction along top of wall exerted by construction above. The reactions may be either uniformly distributed along wall or vary linearly from value at one edge to a different value at other edge.

(*b*) *Transverse Load*

The greatest wind load on span equal to story height of wall is the design transverse load. There are two transverse loads, one acting outward and the other inward.

(*c*) *Racking Load*

The design racking load on wall is the greatest reaction exerted by construction above; it may act in either direction.

3. Walls Which Have Openings

(*a*) *Structural Properties of Walls Which Have Openings*

The design loads for walls that have openings depend upon the structural properties of the walls at doors and windows. These properties may be determined in the laboratory by testing specimens similar to those for walls which have no openings, preferably specimens composed entirely of doors and entirely of windows with the surrounding framework and portion of wall above or below.

No specimens which have openings were included in the program for house constructions, therefore no properties are given in the BMS reports.

With the exception of concentrated and impact loads, it appears probable that door and window constructions can be designed and fabricated which have properties equal to those for continuous walls.

For most wall constructions, it appears probable that the strength at openings is much less than elsewhere. Therefore, it is assumed for this report that portions of wall at doors and windows carry none of the compressive, transverse, or racking load. Any error then, is on the side of safety.

(*b*) *Compressive Load*

If the compressive load over an opening is carried by the portions of wall adjacent to the opening, it is necessary to select a load distribution approximating the actual distribution.

The only discussion of this subject found in the literature is Adams' [26] treatments of bressummers for dividing the weight of a brick wall over a doorway.

(*1*) *Bressummers*

Adams says:

> The next case to be taken is to find the section required to carry a brick partition with a 4-ft door opening 3 ft from one side over a clear span of 15 ft, as shown in Fig. 110. [Fig. 29 in this report]. The load over the doorway is transmitted down the two sides, but the exact distribution of the loading is somewhat doubtful. There are two possible views: the simpler one is that half the load goes each way, and that it is divided over the intervening space in regular proportion from maximum to nothing. Thus, load over door $= 4 \times 5 \times .75 \times 1 = 15$ cwts.,

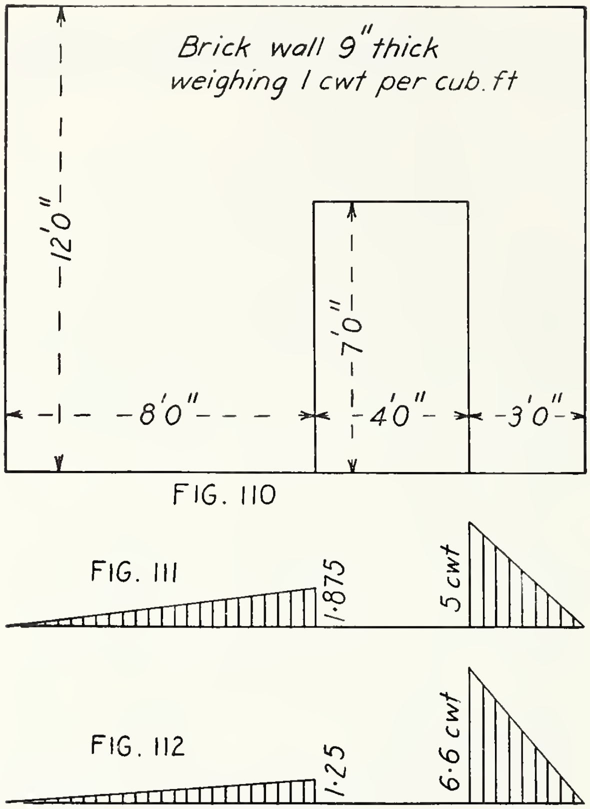

Fig. 110.— Elevation of Brick Partition to be carried by Girder.
Figs. 111 and 112.— Methods of dividing Load over doorway onto the sides.

FIGURE 29.—*Adams' brick wall with doorway.*

[Adams' fig. 110, elevation of brick partition to be carried by girder. Figs. 111 and 112, methods of dividing load over doorway on to the sides]

> half each way = 7.5 cwts.; then for left side 7.5/8 = .9375 per foot run, and .9375 × 2 = 1.875 cwts. increase of load at door jamb as shown in Fig. 111. In a similar manner the extra load on the right will be 7.5 × 2/3 = 5 cwts. at door jamb. The other view is that the load over doorway is transmitted to each abutment in inverse proportion to the distance from its center of gravity, and divided over the intervening space as before, viz., on the left
>
> $\frac{2+3}{15} \times 15 \times \frac{2}{8} = 1.25$ cwts., and on the right
>
> $\frac{2+8}{15} \times 15 \times \frac{2}{3} = 6.6$ cwts., as shown in Fig. 112.
>
> As the former view gives the greater bending moment on the girder, it will be best to adopt it; the weight of the wall is $12 \times .75 \times 1 = 9$ cwts. per foot run, then the load diagram will be as shown in fig. 113. [Fig. 30 in this report].

It is evident that only the first "view" applies to distribution of reactions along top of wall.

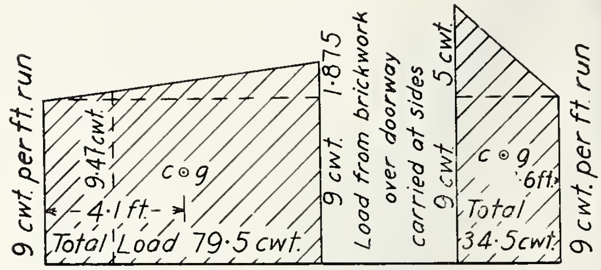

Fig. 113.— Load diagram for Fig. 110.

FIGURE 30.—*Adams' load diagram for brick wall.*

[Adams' fig. 113, load diagram for fig. 110]

(2) *Illustration of Adams' Method*

To apply Adams' method to a numerical example of reactions at top of wall, consider the wall shown on figure 31, 22 ft 8 in. wide, with one door and two windows, one at right edge of wall. There is an attic floor, the joists supported on wall and load-bearing partition, span 10 ft. If weight of this floor is 5 lb/ft^2 and load is 20 lb/ft^2, the reactions on wall are 25 lb/ft from weight and 100 lb/ft from load; combined reaction 125 lb/ft width of wall. Taking the reactions from roof as windward edge 191.9 lb/ft and leeward edge 242.7 lb/ft, both acting downward, the combined reactions along top of wall are 316.9 lb/ft at windward edge and 367.7 lb/ft at leeward edge of wall.

The values of all reactions are shown and also the total at each edge of opening. The greatest, 737.7 lb/ft, is the design compressive load for this wall, provided a greater value does not occur for some other direction of wind.

It is evident that by Adams' method the load at edge of opening is inversely proportional to width of adjacent wall and may be much greater on one edge than on the other, as is shown by reactions at edges of door. Probably the load over an opening is carried mostly by portion of walls quite near the opening and is more uniformly distributed than Adams shows.

(3) *Recommended Method for Distributing Vertical Reactions on Wall*

For this report, therefore, the vertical reaction over half the opening is rotated back over the adjacent wall. The combined reaction on adjacent wall is, then, a uniformly distributed load equal to twice the original value at edge of opening. Unlike Adams' method which requires some computing, this recommended method, in general, involves only doubling the reaction at each edge of opening.

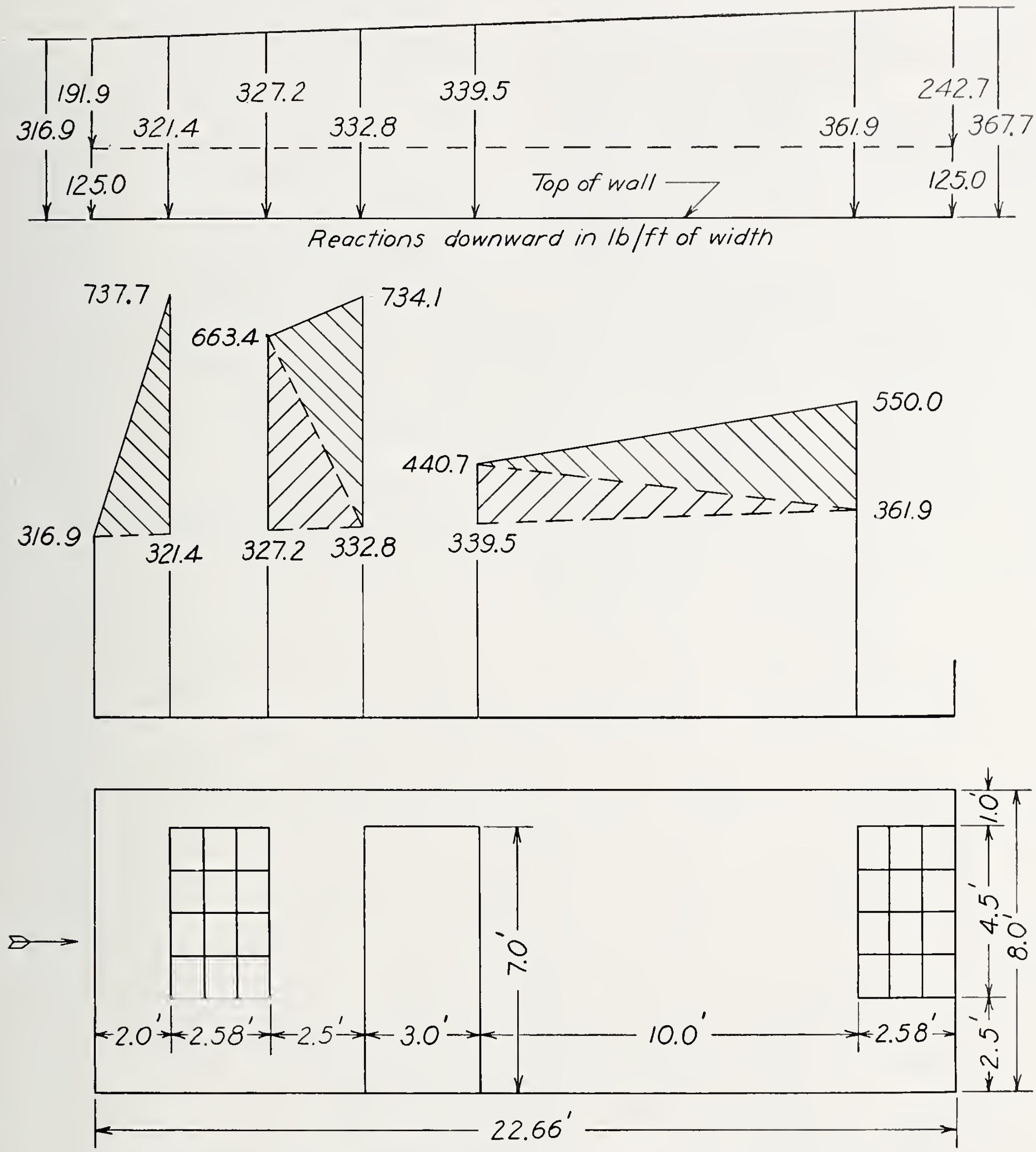

FIGURE 31.—*Adams' method for distributing vertical reactions on wall.*

(4) *Illustration of Recommended Method*

To apply the recommended method to a numerical example, consider the same wall and loading, as shown in figure 32. The original reaction at edges of openings is shown, also the rotation over adjacent wall of reactions over openings. In general, the final reaction adjacent to openings is twice the original reaction.

The width *f* between window and door is less than the sum of the half openings, therefore a strip of the reaction from door must be added to the reaction from window for the overlapping distance giving a load of 990.2 lb/ft for 0.29 ft. It is ridiculous to believe that the reactions over openings are distributed in any such way. An engineer certainly would expect the load to be greater at edges of opening than at intermediate points.

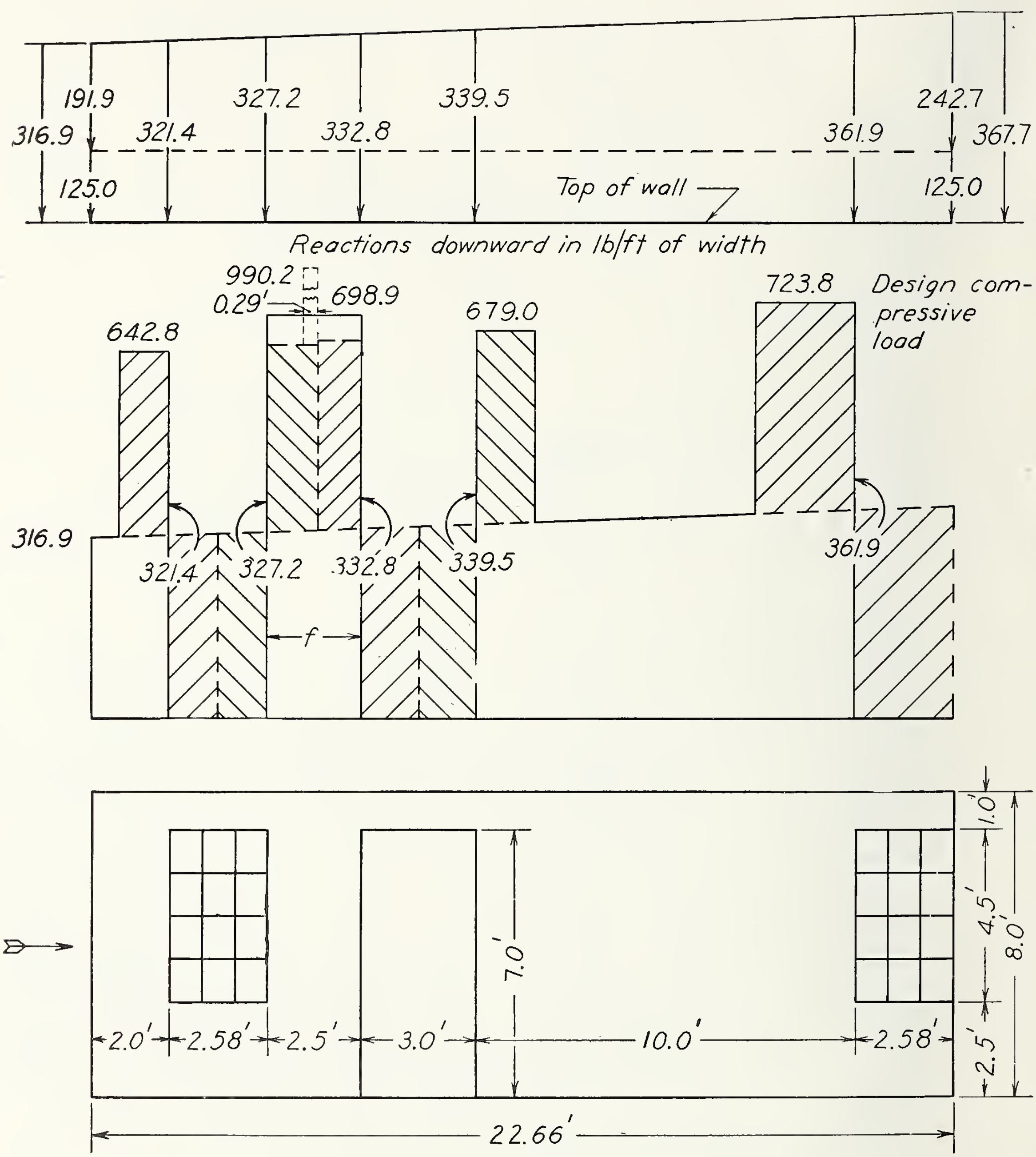

FIGURE 32.—*Recommended method for distributing vertical reactions on wall.*

If the width of wall between openings is less than the sum of the half openings, it appears reasonable to take the sum of the loads over the half openings, divide by width of intervening wall and add to the average original reaction.

On left of width f, the load on half opening is

$$\frac{324.3+327.2}{2}\times\frac{2.58}{2}=420.2 \text{ lb},$$

and on right,

$$\frac{332.8+336.2}{2}\times\frac{3.0}{2}=501.8 \text{ lb}.$$

The sum is 420.2+501.8=922.0 lb. Dividing by width f,

$$\frac{922.0}{2.5}=368.8 \text{ lb/ft}.$$

Average original reaction on width f,

$$\frac{327.2+332.8}{2}=330.0 \text{ lb/ft.}$$

Final reaction on width f,

$$368.8+330.0=698.8 \text{ lb/ft.}$$

This value is slightly greater than if there were no overlap.

For the window at leeward edge of wall, it may be assumed that there is no adjacent wall on the right, particularly because edge windows usually are in pairs, one in each of the intersecting walls. For an edge door or window, it seems reasonable to double the reaction at edge and apply for a distance equal to width of opening.

The greatest reaction, 723.8 lb/ft, is the design compressive load for this wall provided a greater value does not occur for some other direction of wind.

It is evident that the less the width of wall between openings the greater the load and it may be more economical to provide a special member (column) between openings than to make the entire wall of a construction adequate for this localized load. Most houses appear to have ample wall between openings to avoid overlapping of reactions.

For this particular illustration, there is no great numerical difference between the design compressive loads obtained by the Adams' method and the recommended method. Adams' value, however, is close to the windward edge of the wall and the recommended value close to the leeward edge.

It seems evident that, in general, as the distance between openings increases the values obtained by Adams' method decrease, but the recommended values remain the same. Because houses have many openings, probably there is little difference in the design loads obtained by the two methods.

If the wind load on roof is outward, the reaction on wall may act upward and the design "compressive" load on wall is a tensile load, the value of which may be determined by the method just described.

If the reaction along top of wall is upward,

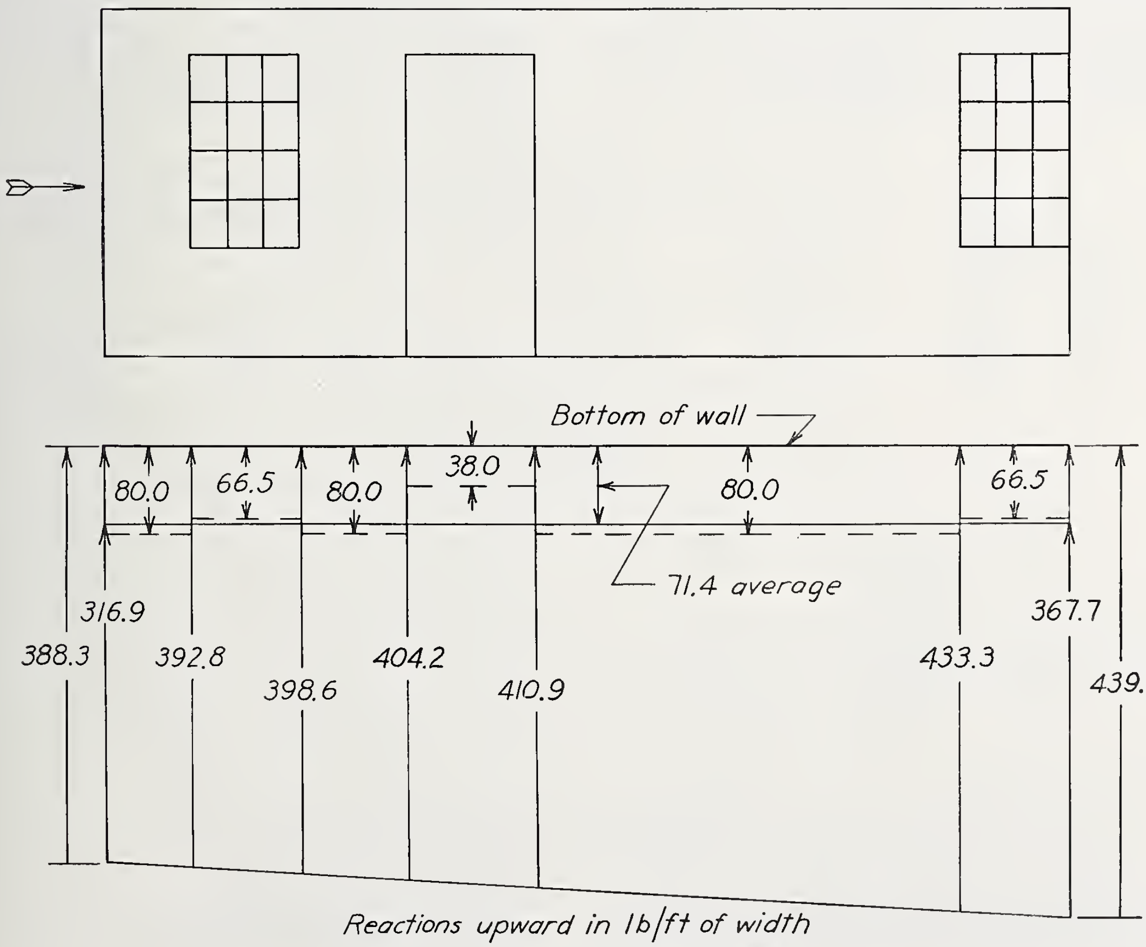

FIGURE 33.—*Reactions at bottom of wall.*

anchors are necessary and the design load for anchors is the greatest upward reaction for any direction of wind.

(5) *Vertical Reaction at Bottom of Wall*

The weight of the wall causes a vertical reaction at bottom of wall as shown in figure 33. The weight of walls included in this program is given in the BMS reports on structural properties.

If this wall is conventional wood-frame construction *QA*, the weight is 10.0 lb/ft^2. The door weighs 4 lb/ft^2 and the windows 7 lb/ft^2. The upward reaction where there are no openings is 80 lb/ft, at the door 38 lb/ft, and at the window 66.5 lb/ft. These nominal reactions are shown although probably the actual weight distribution will be less near midopening and greater at edge of opening, particularly the edge of doorway on which the door is hung.

Probably, because of the stiffness of wall and supporting elements, the actual reactions vary linearly from the value at one edge of wall to a different value at the other edge, but because openings in houses are rather uniformly spaced, it appears adequate to take the average reaction uniformly distributed along the wall.

The vertical reaction at bottom of wall due to the reaction along top of wall is the same magnitude but opposite in direction. For this wall, the combined upward reaction at bottom of wall is 388.3 lb/ft at windward edge of wall and 439.1 lb/ft at leeward edge.

If the reaction along bottom of wall is downward, anchors in the foundation are necessary and the design load for anchors is the greatest downward reaction for any direction of wind.

(*c*) *Transverse Load*

As for compressive load, there are no laboratory data on the transverse properties of walls having openings.

Assuming that wind load at openings is carried by adjacent walls, it is recommended that wind load at openings be distributed in the same way as compressive load and for the same reasons.

(*1*) *Illustration of Recommended Method*

As a numerical illustration, take the same wall as for the compressive load and assume that the wind load on this second story is the value shown in table 5 for the greatest outward wind load of 18.1 lb/ft^2 acting out. The distribution of load is shown on figure 34.

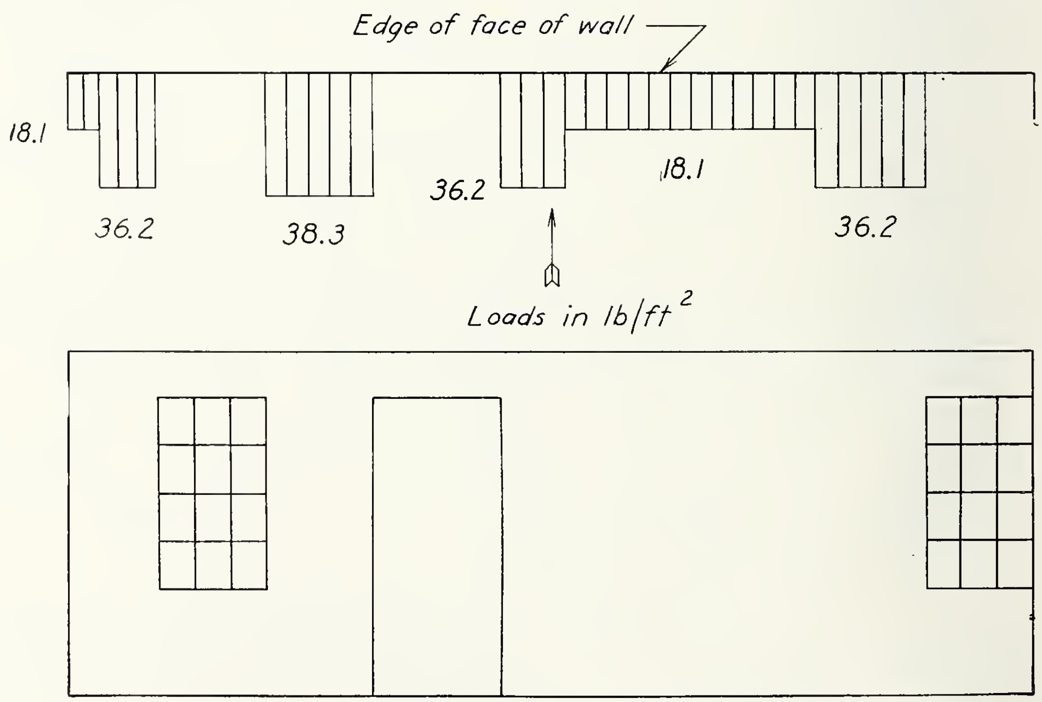

FIGURE 34.—*Recommended method for distributing wind load on wall.*

The design transverse load acting out is 38.3 lb/ft^2 on the span of 8 ft unless a greater value acting out occurs for some other direction of wind. The design transverse load acting in is obtained by the same method.

The transverse reactions at top and bottom of wall are each the design transverse load times half the story height, acting normal to wall and opposing the transverse load. Keys are necessary along the wall at top and bottom. The design load for key is the greatest reaction, either outward or inward, for any direction of wind.

If, where the greatest transverse reaction is exerted on the wall, there is also compressive load on the wall, the resulting frictional resistance may be subtracted from the design load for key. The frictional resistance at bottom of wall is always greater than that at top of wall. The friction may be determined in the laboratory or may be estimated from the compressive load and a coefficient of friction selected by the designer. If the frictional resistance exceeds the transverse reaction, theoretically no key is necessary.

If, where the greatest tranverse reaction is exerted, there is a tensile load on the wall, there is, in general, no friction. There may be friction, provided the anchors exert compressive forces on the top or on the bottom of wall and the adjacent elements *under all service conditions*, particularly when wood plates and sills have shrunk.

(*d*) *Racking Load*

The racking load on walls and load-bearing partitions is a statically indeterminate problem because these loads depend upon the racking deformations of the elements.

(*1*) *Racking Modulus*

If a house construction one-foot square is fixed along one edge and a racking (shearing) force applied along the opposite edge, the *racking modulus* is the force causing a racking deformation of one foot computed from the initial rate of deformation.

The racking modulus for each wall and load-bearing partition in the program was computed from the graph in the BMS report. The allowable racking load, obtained by methods described later, was marked on the graph and the racking deformation noted. This deformation divided by 12 to give the deformation in feet and by 8 ft, the nominal height of specimen, gave the deformation of a specimen one foot high. The allowable load divided by this deformation is the racking modulus in kips per foot.

For some constructions, particularly high-strength masonry, the deformation is so small at allowable load that it is impossible to obtain a value from the graph. In such cases a deformation of 0.0001 inch is taken.

The racking modulus for each wall and load-bearing partition is given in table 15..

Table 15.—*Racking modulus for walls and load-bearing partitions*

Construction symbol	BMS	Element	Allowable load	Deflection in 8 ft	Racking modulus
			Kips/ft	*Inches*	*Kips/ft*
AA	5	Wall	2.50+	0.0001	2,403,000
AB	5	Do	2.50+	.035	6,850
AC	5	Do	2.50+	.035	6,850
AD	5	Do	1.42	.02	6,825
AE	5	Do	1.37	.01	13,170
AF	5	Do	1.20	.015	7,690
AH	9	Do	0.94	.72	125.4
AL	12	Do	.41	.59	66.7
AP	11	Do			
AR	20	Do	4.41+	.0175	24,200
AT	24	Do			
AU	24	Do	2.01	.01	19,330
AV, no braces	27	Do	0.40	.24	160.0
AV, braces	27	Do	.81	.75	103.7
AX	21	Do	2.36	.0125	18,160
AZ	18	Do	0.60	.28	205.5
BD	23	Do	1.98	.015	12,690
BE	22	Do	0.79	.0125	6,080
BF	32	Do	2.08	.010	20,000
BG	31	Do	0.87	.30	280.8
BH	31	Do	1.00	.39	246.3
BI	31	Do	0.88	.40	211.6
BJ	31	Do	1.50	.36	400.0
BK	31	Do	4.50	.76	568.0
BL	31	Do	1.04	.38	262.7
BO	32	Do	2.20	.0041	51,500
BP	32	Do	0.64	.0025	24,620
BQ	36	Do	.60	.10	576.0
BU	30	Do	.88	.40	211.7
BV	26	Do	3.00	.0305	9,440
BW	40	Do	1.50	.05	2,885.0
BX	42	Do	0.97	.39	239.0
BY	42	Do	.78	.61	122.8
CB	37	Do	.13	.10	125.0
CF	38	Do	.72	.0035	19,720
CG	38	Do	.80	.0008	96,200
CH	39	Do	.23	.20	110.6
CI	47	Do	.80	.52	147.8
CJ	47	Partition	.48	.25	184.6
CM	48	Wall	.34	.127	257.5
CN	48	Do	.47	.15	301.3
CO	48	Partition	.47	.081	557.0
CP	53	Wall	2.40+	.0008	288,500
CQ	46	Do	0.30	.13	222.2
CX	67	Do	1.50	.285	505.0
CZ	61	Do	2.50+	.0025	96,200
DA	61	Do	2.50+	.002	120,200
DB	78	Do	1.00	.002	48,100
DC	78	Do	1.00	.007	13,700
DD	78	Do	2.50+	.0006	400,000
DE	78	Do	1.92+	.0001	1,845,000
DF	78	Do	0.65	.0033	18,900
DG	74	Do	.33	.072	440.0
DH	74	Do	.25	.04	601.0
DK	72	Do	.69	.106	624.0
DL, no braces	90	Do	.03	.10	28.84
DLa, braces	90	Do	.16	.80	19.2
DP	86	Do	1.29	.0028	44,200
QA	25	Do	0.50	.185	259.0
QD	25	Partition	.38	.21	173.5

(*2*) *Method for Determining Racking Load on Wall*

The method for computing the racking load on walls and load-bearing partitions is most clearly explained by a numerical case. Consider the plan shown on figure 35 for the first story of a two-story house.

The outside dimensions are 16 by 30 ft and the walls are 8 ft high. The south wall (front door) and east wall are construction *AB*, medium-strength brick, modulus 6,850 kips/ft; the north wall and west wall are construction *QA*, conventional wood frame, gypsum plaster inside and bevel siding outside, modulus 259.0 kips/ft; both the partitions are construction

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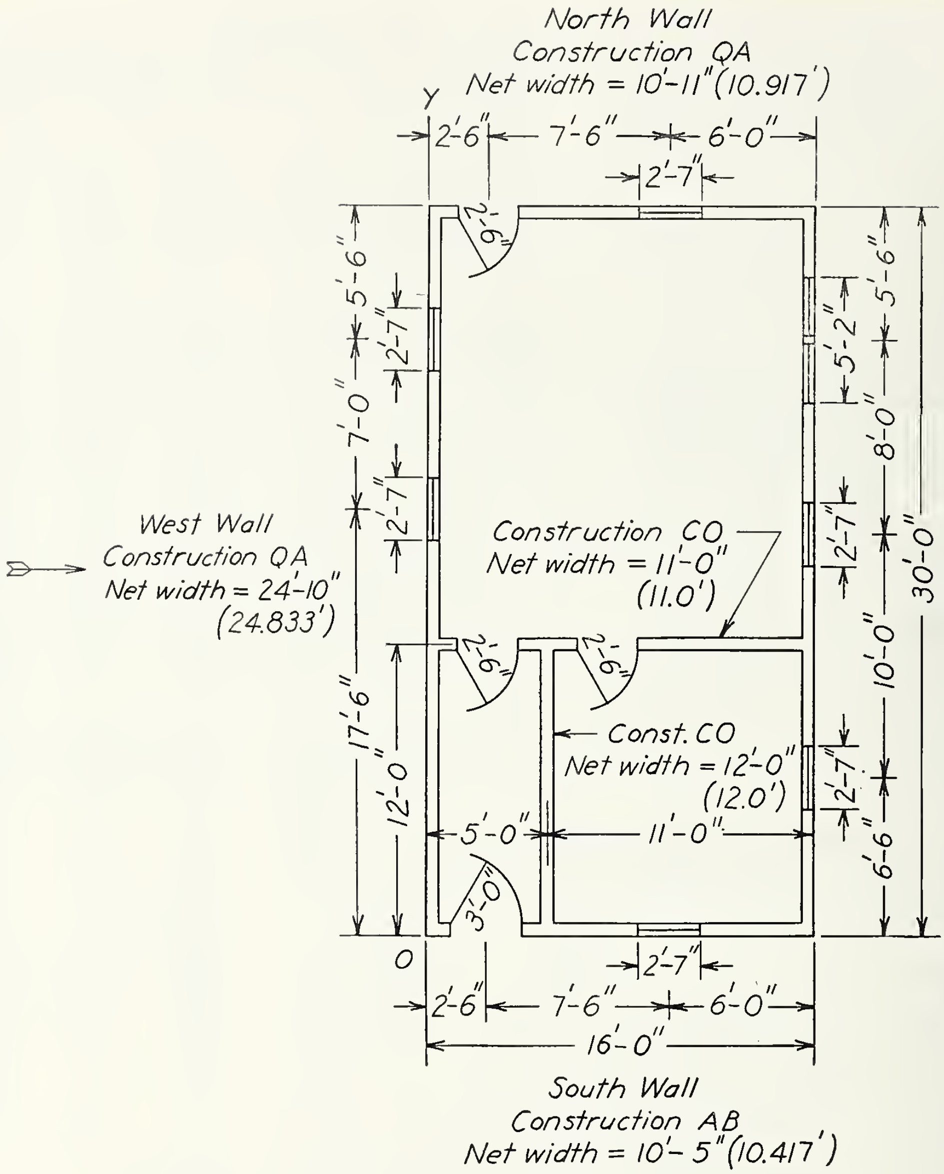

FIGURE 35.—*Plan of first story of two-story house.*

CO, wood frame, both faces fiberboard glued to frame, modulus 557.0 kips/ft.

The west wind acting on the story above and the upper half of this story exerts a wind load, taken as 7.0 kips to leeward on the floor at top of this story. This floor is in equilibrium under the wind load and reactions due to racking forces on the walls below the floor.

The reactions on floor, due to racking forces on walls, are equal and act in opposite directions to racking forces on walls. For convenience in this discussion, these reactions on floor are termed "racking reactions."

Provided all openings have closures and roof and all stories above are symmetrical, the resultant wind load may be considered as acting on the floor at midwidth of the windward edge. If these conditions are not fulfilled, the resultant may not act at midwidth.

In the BMS reports there are no racking properties for walls having openings, therefore, it is assumed that only portions of wall between openings resist racking load; moreover; it is assumed that resistance is proportional to width.

The net width of each wall is given on the plan, being the outside width minus the width of each opening.

For a given racking deformation, the force on net width of wall may be computed. The deformation divided by height of wall is deformation at height of one foot. This value times racking modulus is the force on wall one foot wide, which when multiplied by net width of wall is racking force on the wall.

Translation.—Assume, for the time being, that the floor moves to leeward, parallel to position before wind load is applied, a distance r_1, — the racking deformation of walls and partitions parallel to wind.

The magnitude of racking reactions on floor due to translation are found by computing the racking load on each wall and partition parallel to wind for the deformation r_1. These reactions are

North wall:

$$(r_1/8)\times 259.0\times 10.917 = -353.4\,r_1.$$

East-west partition:

$$(r_1/8)\times 557.0\times 11.0 = -765.9\,r_1.$$

South wall:

$$(r_1/8)\times 6850\times 10.417 = -8919.5\,r_1.$$

$$\Sigma F_x = 0 = 7.0 - 353.4\,r_1 - 765.9\,r_1 - 8919.5\,r_1.$$

$$r_1 = 0.0006973 \text{ ft.}$$

Substituting this value of r_1 gives

North wall .... —0.2464 kip.
East-west partition .... —.5341 kip.
South wall .... —6.2190 kip.

$$\Sigma F_x = 0 = 7.0 - 0.2464 - 0.5341 - 6.2190$$

$$7.0 = 6.9995 \text{ (check)}$$

$$\Sigma F_y = 0 \text{ (check)}$$

$$\Sigma M_o = 0 = (-7.0\times 15) + (0.2464\times 30) + (0.5341\times 12)$$

$$= -105 + 7.392 + 6.409$$

$$= -105 + 13.801 = -91.199 \text{ ft-kips clockwise.}$$

Rotation.—The resultant clockwise moment of 91.199 ft-kips rotates the floor clockwise about the center of rotation causing an increase in the racking deformation of north wall and racking reaction on floor, and a decrease in deformation of south wall and reaction on floor.

The center of rotation for this story is a fixed point in the floor and is at the intersection of the action lines of the resultant of racking *rates* for east-west walls and resultant of racking *rates* for north-south walls. The center of rotation lies within the boundaries of the floor.

The *racking rate for a wall*, i.e., the rate of increase in racking load with increase in racking deformation, is the racking modulus divided by height of wall times net width of wall.

The center of rotation is computed as follows:

East-west coordinate, x

Racking rate:

West wall .... $\frac{259.0}{8}\times 24.833 =$ 804.0

North-south partition .... $\frac{557.0}{8}\times 12.0 =$ 835.5

East wall .... $\frac{6850}{8}\times 19.667 =$ 16839.0

Resultant = 18478.5

Moments, origin at O:

West wall	804.0×0	= 0
North-south partition	835.5×5.0	= 4177.5
East wall	16839.0×16.0	= 269400.0
	Total	= 273577.5

$$x = \frac{273577.5}{18478.5} = 14.805 \text{ ft.}$$

North-south coordinate, y

Racking rate:

North wall .... $\frac{259.0}{8}\times 10.917 =$ 353.4

East-west partition ... $\frac{557.0}{8}\times 11.0 =$ 765.9

South wall .... $\frac{6850}{8}\times 10.417 =$ 8919.5

Resultant = 10038.8

Moments, origin at O:

North wall	353.4×30.0	= 10600.0
East-west partition ...	765.9×12.0	= 9191.0
South wall	8919.5×0	= 0
	Total	= 19791.0

$$y = \frac{19791.0}{10038.8} = 1.972 \text{ ft.}$$

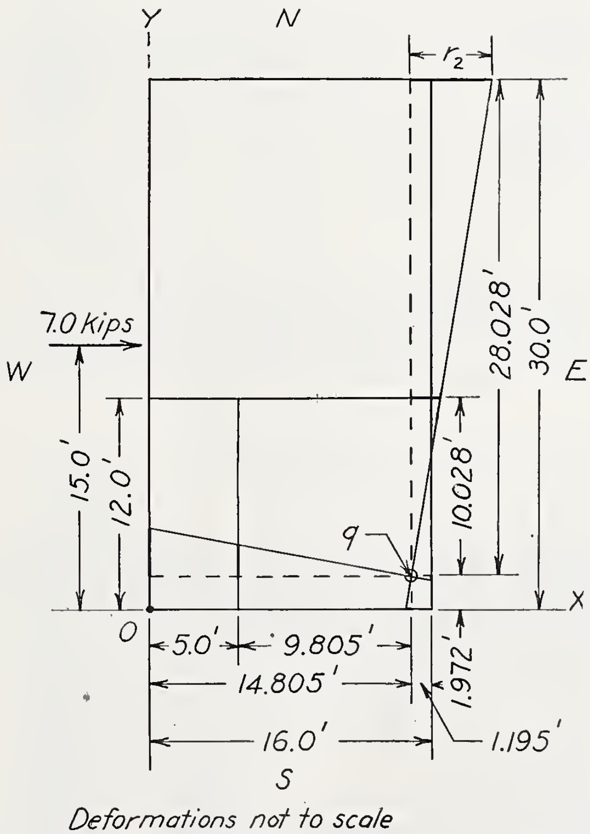

FIGURE 36.—*Racking deformation of walls due to rotation of floor.*

The racking deformation of each wall due to rotation of floor is proportional to the moment arm of wall about center of rotation as shown on figure 36.

If the rotation causes deformation r_2 of north wall, the deformation of each wall is

West wall .... $\frac{14.805}{28.028}r_2=+0.528\ r_2$.

North-south partition... $\frac{9.805}{28.028}r_2=+0.3495\ r_2$.

East wall............. $\frac{1.195}{28.028}r_2=-0.04265\ r_2$.

North wall............ $\frac{28.028}{28.028}r_2=+\ r_2$.

East-west partition.... $\frac{10.028}{28.028}r_2=+0.3578\ r_2$.

South wall............ $\frac{1.972}{28.028}r_2=-0.07036\ r_2$.

Each deformation times the racking rate for the wall gives the racking force and this times the moment arm to center of rotation gives the moment on wall due to rotation of floor. These moments are

West wall...	$0.528\ r_2\times$	$804.0\times$	$14.805=$	$-6280.0\ r_2$.
North-south partition..	$0.3495\ r_2\times$	$835.5\times$	$9.805=$	$-2863.0\ r_2$.
East wall ...	$0.04265\ r_2\times$	$16839.0\times$	$1.195=$	$-857.5\ r_2$.
North wall	$r_2\times$	$353.4\times$	$28.028=$	$-9905.1\ r_2$.
East-west partition..	$0.3578\ r_2\times$	$765.9\times$	$10.028=$	$-2748.1\ r_2$.
South wall ..	$0.07036\ r_2\times$	$8919.5\times$	$1.972=$	$-1237.6\ r_2$.
			Total=	$-23891.3\ r_2$.

The moments of the racking reactions on floor are equal and of opposite sense.

Then for floor after rotation, the sum of the moments of racking reactions equals the resultant moment due to translation.

$$23891.3r_2=91.199 \text{ ft-kips.}$$
$$r_2=0.003817 \text{ ft.}$$

This is 28.08 seconds of arc.

Substituting this value for r_2 into expressions for the deformation for each wall for rotation gives the deformation in feet.

The racking deformation of walls due to translation and rotation are

	Translation, r_1	Rotation, r_2	Total, r_1+r_2
	ft	*ft*	*ft*
West wall.......	0.0	0.002016	0.002016
North-south partition........	.0	.001334	.001334
East wall	.0	—.0001628	—.0001628
North wall......	.0006973	.003817	.004514
East-west partition........	.0006973	.001365	.002062
South wall	.0006973	—.0002684	.0004289

The deformation for each wall times the racking rate for the wall gives the following racking force:

West wall.....	0.002016 $\times$	804.0=	1.620 kips.
North-south partition......	.001334 $\times$	835.5=	1.114 kips.
East wall......	.0001628 $\times$	16839.0=	—2.740 kips.
North wall......	.004514 $\times$	353.4=	1.595 kips.
East-west partition......	.002062 $\times$	765.9=	1.580 kips.
South wall	.0004289 $\times$	8919.5=	3.825 kips.

The racking reactions on floor are equal and opposite to these racking forces on walls. The floor is in equilibrium under the wind load and these racking reactions. These reactions on floor are shown on figure 37.

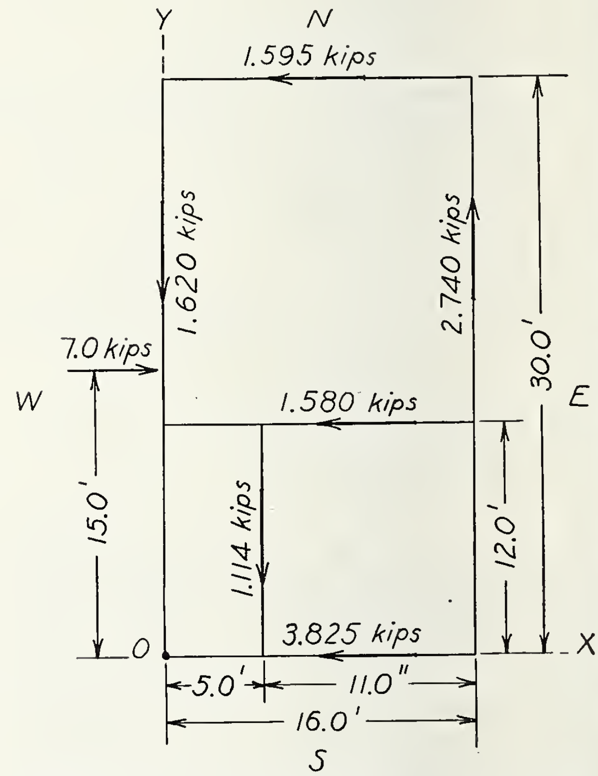

FIGURE 37.—*Reactions on floor due to wind load and racking of walls.*

$$\Sigma F_x=0=-1.595-1.580-3.825+7.0$$
$$7.0=7.0 \text{ (check)}$$
$$\Sigma F_y=0=-1.620-1.114+2.740$$
$$2.734=2.740 \text{ (check)}$$
$$\Sigma M_o=0=+1.595\times30.0+1.580\times12.0+2.740\times16.0$$
$$-1.114\times5.0-7.0\times15.0$$
$$110.650=110.570 \text{ (check).}$$

The designer should check each floor of house for equilibrium. It is evident that in this illustration the racking forces for translation only are inadequate for design as shown by the following camparison.

	Translation only, A Kips	Translation and Rotation, B Kips	Ratio, A/B
West wall	0	1.62	0
North-south partition	0	1.114	0
East wall	0	2.740	0
North wall	0.2464	1.595	0.1545
East-west partition	.5341	1.580	0.3381
South wall	6.219	3.825	1.625

Manifestly, if the forces for translation only are taken as approximations, both partitions and all walls except south wall may be unsafe because the estimate racking forces are much too small. On the other hand, the south wall may be about twice the necessary strength.

The racking load is the racking force divided by the net width of wall. These loads are:

West wall .... $\frac{1.620}{24.833}=0.0652$ kip/ft.

North-south partition... $\frac{1.114}{12.0}=0.0928$ kip/ft.

East wall............. $\frac{2.740}{19.667}=0.1393$ kip/ft.

North wall............ $\frac{1.595}{10.917}=0.1460$ kip/ft.

East-west partition..... $\frac{1.580}{11.0}=0.1437$ kip/ft.

South wall............ $\frac{3.825}{10.417}=0.3670$ kip/ft.

For each wall and load-bearing partition the greatest racking load for any direction of wind is the design racking load. It may act in either direction along wall.

(3) *Method for Determining Righting Reaction on Wall*

The racking loads at top and bottom of wall exert an overturning couple on wall equal to the racking load times the story-wall height.

For the wall to be in equilibrium, there also must be a righting couple of the same magnitude. The vertical reactions of the righting couple act downward on windward portion of wall and upward on leeward portion.

Because roof or floor at top of wall cannot exert the righting couple, it must be applied at bottom of wall. If the wall is supported by posts or piers and there are more than two supports, for determining righting reactions, it is divided into portions each of which is a simple beam having only two supports. The righting reactions are point loads and the moment arm is the distance between supports. The reactions are readily computed by dividing the overturning moment by the moment arm. Posts may be designed to carry upward reactions (anchors) as well as downward reactions but they can carry no racking load. Even if the supports have a large cross section, as for brick piers two or three feet square, there are no laboratory reports on their racking resistance and assigning a design racking load presents difficulties.

Certainly an engineer would hesitate to assign a design racking load for a pipe 4 inches in diameter functioning as a post in the cellar of a house.

The racking load at bottom of wall, therefore, must be transferred laterally by a floor to walls parallel to the racked wall inversely proportional to the distance between the racked wall and the parallel wall.

If, on the other hand, the wall is in contact along the bottom with supporting walls or other members, the righting reactions are line loads.

The greatest downward righting reaction is at the windward end of the line of contact and the reaction decreases linearly to zero at the middle of the line. The reaction, similarly, increases upward from middle of line to the greatest upward reaction at leeward end of line.

Let d be the length of line of contact in feet and take origin of moments at middle of line. Let Q be the righting reaction in pounds per foot at windward and leeward end of line of contact.

For the half contact between end of line and origin, the total righting reaction is $\frac{Q}{2}\times\frac{d}{2}$ and righting moment $\frac{Q}{2}\times\frac{d}{2}\times\frac{2}{3}\times\frac{d}{2}$, or $\frac{Qd^2}{12}$, for each half contact. The total righting reaction for the wall is $\frac{Qd^2}{6}$ foot-pounds.

The value of Q is obtained by equating this expression to the overturning moment and solving for Q.

Some walls and load-bearing partitions bear on floors between the supports for the floor. If the wall is normal to the joists in the floor, the righting reaction on windward portion of wall tends to lift the floor and on leeward portion to depress the floor. These vertical righting reactions are transferred by the floor to the adjacent floor supports inversely proportional to the distance from racked wall to the support.

The effect of righting reactions is to either increase or decrease the resultant of all vertical loads on wall or other element due to weight of roof, wall, and floor above and also to wind load and snow load on roof and surface load on floor. By this means, the design compressive load on each wall is determined and also the design loads for fastenings.

When combining the resultant of the vertical loads and the righting reactions, it is helpful to make diagrams to scale for each story of each wall showing the location of openings. There should be a diagram for each direction of wind, south, east, north, and west, and for house occupied and unoccupied. Plot the resultant of vertical loads and add or subtract the righting reaction.

XIV. PARTITIONS

1. Load-bearing Partitions

The discussion of walls also applies to load-bearing partitions and the design loads are determined by the same methods.

(*a*) *Compressive Load*

The compressive load is the vertical reaction from floor above only, therefore always uniformly distributed along the partition. If there are floors adjacent to both faces of a partition, the compressive load is the sum of the reactions from each floor. There can be no upward "compressive" load on a partition. The bearing of a floor on a partition is one-third the thickness of structural members from the face toward the floor.

(*b*) *Transverse Load*

There is no transverse load on a load-bearing partition because it is assumed that the barometric pressure is the same everywhere within the house.

(*c*) *Racking Load*

The racking load on a load-bearing partition is obtained at the same time that racking loads on walls are computed.

2. Nonload-bearing Partitions

The only design loads on nonload-bearing partitions are concentrated and impact loads. These depend only upon the occupancy.

Care must be taken that there is sufficient clearance between top of a nonload-bearing partition and floor above so that none of the floor load is transmitted to the partition.

XV. FLOORS

1. Design Load

As the floor is considered a rigid diaphragm, loads in the plane of the floor may be neglected when computing design loads. The design loads, then, are transverse, concentrated, and impact. Concentrated and impact loads depend only upon the occupancy, and the selection of the design values will depend upon the judgment and experience of the architect.

(*a*) *Transverse Load*

The design transverse load is expressed in pounds per square foot, uniformly distributed over the floor span.

For any floor, the span is the distance between bearings on adjacent supports, such as walls or load-bearing partitions.

2. No Partition Above Floor

If there is no partition above the floor and between the supports, the applied surface load on floor is the design transverse load on the floor.

3. Partition Above Floor

If there is a partition above the floor and between supports, the reaction on floor is a line load taken as acting at midthickness of partition. This line load is the sum of the average weight of partition plus reactions, if any, from floor above. The average weight is the sum of the net weight of partition plus weight of closures divided by width of partition.

Because there can be no surface load on the floor in the area occupied by the partition, the line load equivalent to the surface load should be subtracted from the line load for the partition.

There are two cases to be considered; partition normal to structural members in floor and partition parallel to members.

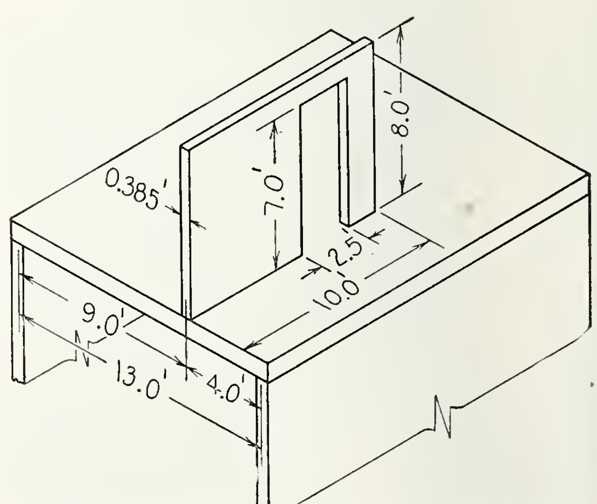

Figure 38.—*Partition normal to floor members.*

(*a*) *Partition Normal to Floor Members*

The floor is a simple beam under the applied surface load and line load exerted by partition. From the maximum bending moment under this loading, an equivalent surface load is obtained. This is the design transverse load for the floor.

Illustration of method for determining design transverse load.—The floor shown on figure 38 is supported by walls or load-bearing partitions, span 13 ft, surface load 40 lb/ft². The partition above is construction *CJ*, height 8 ft, width 10 ft, thickness 4 5/8 in., weight 5.42 lb/ft². The door is 2 ft 6 in. by 7 ft. The line load on partition from floor above is 182.0 lb/ft of width.

The applied loads on floor are

From partition—

Weight:

Gross area of partition....$10\times8=$ 80.0 ft².
Area of door.............$2.5\times7=$ 17.5 ft².
Net area of partition...............62.5 ft².
Net weight of partition........$62.5\times5.42=338.8$ lb.
Weight of door..........$17.5\times4=$ 70.8 lb.
Total weight of partition.........408.8 lb.
Average weight of partition..

$$\frac{408.8}{10}=40.88 \text{ lb/ft of width.}$$

Reactions from floor above..........182.0 lb/ft of width.

Deducting the equivalent of the applied surface load, the partition reaction is

$$(40.88+182.0)-(0.385\times40.0)=207.48 \text{ lb/ft of width.}$$

The applied surface load is 40.0 lb/ft². These loads on the floor are shown on figure 39.

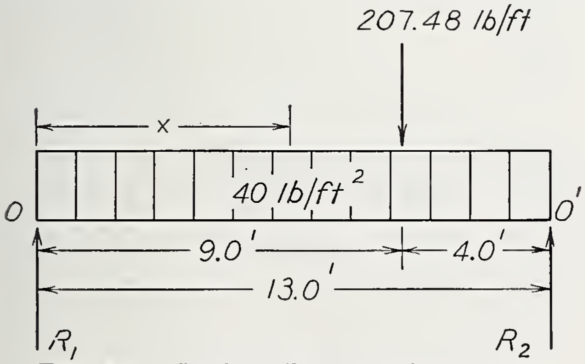

FIGURE 39.—*Loads on floor, partition normal to floor members.*

The reactions may be found separately for the surface load and the line load and the total reaction at each support is the sum of the values for the separate loads. The reactions are

For surface load—

Total surface load...$40.0\times13.0=520.0$ lb/ft of width.

$$R_{S_1}=R_{S_2}=260.0 \text{ lb/ft of width.}$$

For line load—

$$\Sigma M_o=0=(R_{L_2}\times13.0)-(207.48\times9.0)$$
$$R_{L_2}=143.64 \text{ lb/ft of width.}$$

$$\Sigma M_o'=0=(207.48\times4.0)-(R_{L_1}\times13.0)$$
$$R_{L_1}=63.84 \text{ lb/ft of width.}$$

$$R_1=260.0+63.84=323.84 \text{ lb/ft of width.}$$

$$R_2=260.0+143.64=403.64 \text{ lb/ft of width.}$$

$$\Sigma F_y=0=323.84+403.64-520.0-207.48 \text{ (check)}$$

The moment is a maximum where the shear is zero.

$$\Sigma F_y=0=323.84-40x$$
$$x=8.096 \text{ ft.}$$

The maximum moment is

$$M_x=-R_1x+\left[40x\left(\frac{x}{2}\right)\right]=1310.0 \text{ ft-lb/ft of width.}$$

The surface load which gives the same maximum bending moment is

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

$$M=1310.0 \text{ ft-lb/ft of width.}$$
$$w=62.0 \text{ lb/ft}^2.$$

Therefore, the design transverse load is 62.0 lb/ft² on a 13.0-ft span.

If there is no partition above floor, the design transverse load is 40.0 lb/ft² on a 13.0-ft span.

(*b*) *Partition Parallel to Floor Members*

The line load on floor is determined in the same way as for partition normal to members. The line load equivalent to applied surface load is deducted. The floor must be reinforced under the partition to support the line load exerted by the partition.

4. Floor With Opening

The method of determining the design transverse load for floor with opening is illustrated by the attic floor shown on figure 40. The floor, span 13.0 ft, has a 2.0- by 3.0-ft scuttle opening. The surface load is 20 lb/ft².

Inner bearing of portions *cdgh* and *efij* is taken at inner edge of opening.

Design transverse load on each portion is

abcf, 20.0 lb/ft², span 13.0 ft.
cdgh, 20.0 lb/ft², span 8.0 ft.
efij, 20.0 lb/ft², span 3.0 ft.
gjkm, 20.0 lb/ft², span 13.0 ft.

The weight of the floor is taken as 10.6 lb/ft².

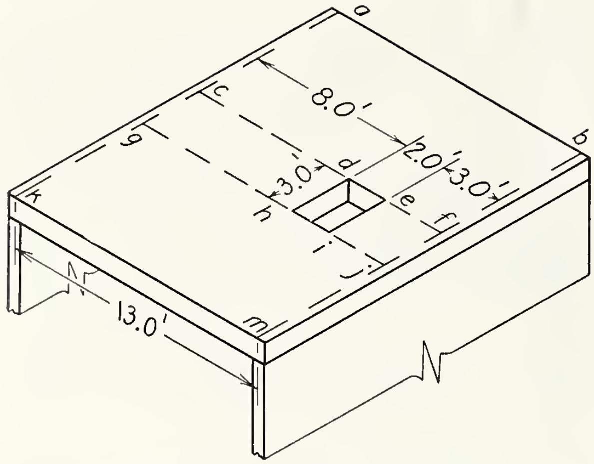

FIGURE 40.—*Floor with opening.*

The inner end of portion *cdgh* bears on header *dh*. The design transverse load for header *dh* is 122.4 lb/ft on 3.0-ft span. Reaction on each of joists *cf* and *gj* is 183.6 lb. These reactions are design loads for attaching header to joists.

The inner end of portion *efij* bears on header *ei*. The design transverse load on header *ei* is 45.9 lb/ft on 3.0-ft span. Reaction on each of joists *cf* and *gj* is 68.8 lb. These reactions are design loads for attaching header to joists.

Along line *cf* and *gj* the floor must be either reinforced or additional joists provided to support the header reactions.

The design load for fastening portion *cdgh* to header *dh* is 122.4 lb/ft and for fastening portion *efij* to header *ei* 45.9 lb/ft. These design loads may be tension, compression, or shear, depending upon the method of attaching floor to header.

5. Floor With Stair Opening

The first-story floor of two-story house shown in figure 41 has a span of 22.0 ft, and a 3.17- by 9.33-ft opening for cellar stairs. Top of stairs is fastened to header *cg;* bottom rests on the cellar floor. Total rise of stairs is 7.83 ft; total run is 7.5 ft.

The stairs to second story consist of two parts; a straight portion having 8 treads and a 90°-turned portion having three treads. If one carriage is fastened to the wall and the other to a partition, the stairs are statically indeterminate. When computing the design loads on wall and partition, it is safe to assume that all of the load on stairs is carried by these walls. The outer end of each tread of the turn is fastened either to wall or to the header in second floor; the inner end of each riser of the turn is fastened to a post. Straight portion of stairs has a total rise of 6.0 ft and a total run of 6.67 ft.

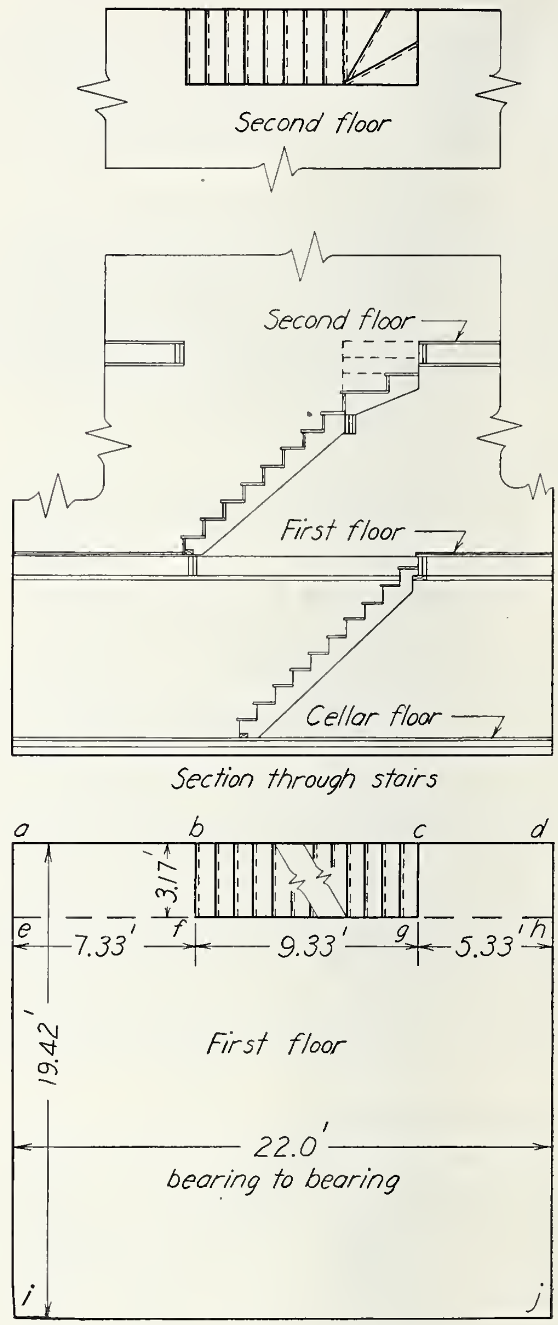

FIGURE 41.—*Floor with stair opening.*

For this report, the applied load on each tread is taken as 150 lb and the weight of stairs as 25 lb per tread.

The total applied load on cellar stairs is 1,500 lb, total weight of stairs 250 lb. Vertical reaction upward on each end of stair carriage is 437.5 lb.

For straight portion of stairs to second story, the load on supporting wall and partition, due to applied load per tread and weight per tread, is 105 lb/ft of width. The load on wall and header due to total load of each tread of the turn (applied load per tread plus weight per tread) is 39.4 lb/ft. Half the total load of turn is carried by the post, total 262.5 lb compression.

The surface load on floor is 40 lb/ft^2. Inner bearing of portion *abef* and *cdgh* are taken at inner edge of opening.

The design transverse load on each portion of floor is

abef, 40 lb/ft^2, span 7.33 ft.
cdgh, 40 lb/ft^2, span 5.33 ft.
ehij, 40 lb/ft^2, span 22.0 ft.

The inner end of portion *abef* bears on header *bf*. The design transverse load on header *bf* is 146.6 lb/ft on 3.17-ft span. Reaction on each of joists *ad* and *eh* is 232.4 lb. These reactions are design loads for attaching header to joists.

The inner end of portion *cdgh* bears on header *cg*. The design transverse loads on header *cg* are; 106.6 lb/ft on 3.17-ft span, and two symmetrically spaced loads of 437.5 lb from cellar stairs. Reaction on each of joists *ad* and *eh* is 606.5 lb. These reactions are design loads for attaching header to joists.

In addition to header reactions, joist *eh* carries a load of 105 lb/ft for a distance of 6.67 ft, the weight of the partition, and a load from post of 262.5 lb.

Along line *eh* the floor must be either reinforced or an additional joist provided.

The design load for fastening portion *abef* to header *bf* is 146.6 lb/ft, and for fastening portion *cdgh* to header *cg*, 106.6 lb/ft. These design loads may be tension, compression, or shear, depending upon the method of attaching floor to header.

XVI. WEIGHTS OF HOUSE CONSTRUCTIONS

1. Description of Constructions

A brief description of each of the constructions covered by the BMS reports on structural properties is given in table 16.

Table 16.—*Summary of house constructions from BMS structural properties reports*

Construction symbol	BMS report	Element	Sponsor, trade name, and description	Weight, actual, based on face area.
MASONRY CONSTRUCTION SECTION, NBS				
				lb/ft^2
AA	5	Wall	Units, high-strength brick; cement mortar; excellent workmanship.	96.00
AB	5	Do	Units, medium-strength brick; cement-lime mortar; commercial workmanship.	73.90
AC	5	Do	Units, medium-strength brick; cement-lime mortar; excellent workmanship.	78.90
AD	5	Do	Units, structural clay tile on end; cement-lime mortar; excellent workmanship.	42.60
AE	5	Do	Units, structural clay tile on side; cement-lime mortar; excellent workmanship.	38.90
AF	5	Do	Units, stone-concrete block; cement-lime mortar; excellent workmanship.	54.50
H. H. ROBERTSON CO., "KEYSTONE BEAM STEEL FLOOR"				
AG	10	Floor	Frame and lower face, sheet steel; upper face, concrete fill and composition finish floor.	55.60
INSULATED STEEL CONSTRUCTION CO., "FRAMELESS-STEEL"				
AH	9	Wall	Frame, sheet-steel; both faces, sheet steel.	7.47
AI	9	Partition, nonload-bearing.	Frame, sheet-steel; both faces, gypsum board.	4.04
AJ	9	Floor	Frame and deck, sheet-steel; upper face, wood sleepers and wood finish floor; lower face, sheet steel.	9.52
AK	9	Roof	Frame and deck, sheet-steel; upper face, insulation and built-up roofing; lower face, sheet steel.	10.73
STEEL BUILDINGS, INC., "STEELOX"				
AL	12	Wall	Frame and outside face, sheet steel; inside face, wood furring strips and insulating fiberboard.	4.96
AM	12	Partition, nonload-bearing.	Frame, wood; both faces, insulating fiberboard.	3.04
AN	12	Floor	Frame and lower face, sheet steel; upper face, wood sleepers and wood finish floor.	8.08
AO	12	Roof	Frame and lower face, sheet steel; upper face, wood furring strips, insulation, and built-up roofing.	6.99
CURREN FABRIHOME CORPORATION, "FABRIHOME"				
AP	11	Wall	Frame, sheet steel; inside face, gypsum board; outside face, plywood.	4.17
AQ	11	Partition, nonload-bearing.	Frame, sheet steel; both faces, gypsum board.	5.10
CONNECTICUT PRE-CAST BUILDINGS CORPORATION, "TWACHTMAN"				
AR	20	Wall	Slab, reinforced concrete	36.80
AS	20	Floor	Do	37.70
STRUCTURAL CLAY PRODUCTS INSTITUTE				
AT	24	Wall	Units, clay brick; cement mortar; deformed steel bars (vertical and horizontal).	88.70
AU	24	Do	Units, facing, brick, and backing, structural-clay tile on side; cement mortar; air space and steel wall ties.	62.30
BENDER BODY CO., "BENDER STEEL HOME"				
AV	27	Wall	Frame, sheet steel; inside face, insulating fiberboard; outside face, insulating fiberboard and sheet steel.	7.24

Construction symbol	BMS report	Element	Sponsor, trade name, and description	Weight, actual, based on face area.
			TILECRETE FLOORS, INC., "TILECRETE"	
AW	16	Floor	Joists, rolled steel; fillers, structural clay tile; fill, concrete; upper face, wood parquetry finish floor.	51.50
			NATIONAL CONCRETE MASONRY ASSOCIATION	
AX	21	Wall	Units, facing and backing, concrete block; cement mortar; air space and steel wall ties.	44.10
			WHEELING CORRUGATING CO., "WHEELING LONG-SPAN STEEL FLOOR"	
AY	15	Floor	Frame and deck, sheet steel; upper face, asphalt roofing, wood sleepers, and wood finish floor; lower face, metal lath and plaster.	16.10
			HARNISCHFEGER CORPORATION, "PRE-FAB"	
AZ	18	Wall	Frame, sheet steel; inside face, furring strips, paper, gypsum-board lath, and plaster; outside face, insulating fiberboard, furring strips, plywood, paper, and wood shingles.	10.60
BA	18	Partition, nonload-bearing.	Frame, sheet steel; both faces, furring strips, gypsum-board lath, and plaster.	12.10
BB	18	Floor	Frame, sheet steel; upper face, wood nailing strips, wood subfloor, paper, and wood finish floor.	7.59
BC	18	Do	Frame, sheet steel; upper face, wood nailing strips, wood subfloor, paper, and wood finish floor; lower face, wood nailing strips, paper, gypsum-board lath, and plaster.	14.10
			BRICK MANUFACTURERS ASSOCIATION OF NEW YORK, INC.	
BD	23	Wall	Units, clay brick; cement-lime mortar; air space and steel wall ties.	67.60
			W. E. DUNN MANUFACTURING CO., "DUNN-TI-STONE"	
BE	22	Wall	Units, concrete slabs; cement-lime mortar; air space and steel tie bars.	lb/ft² 49.50
			NATIONAL CONCRETE MASONRY ASSOCIATION	
BF	32	Wall	Units, facing, shale brick, and backing, concrete block; cement-lime mortar.	60.90
			INSULITE CO., "INSULITE"	
BG	31	Wall	Frame, wood; inside face, insulating-fiberboard lath and plaster; outside face, insulating fiberboard and wood-bevel siding.	8.80
BH	31	Do	Frame, wood; inside face, interior fiberboard; outside face, insulating fiberboard and wood-bevel siding.	5.02
BI	31	Do	Frame, wood; inside face, interior fiberboard; outside face, insulating fiberboard and wood-bevel siding.	4.51
BJ	31	Do	Frame, wood; inside face, insulating-fiberboard lath and plaster; outside face, insulating fiberboard, paper, metal lath, and stucco.	20.00
BK	31	Do	Frame, wood; inside face, insulating-fiberboard lath and plaster; outside face, insulating fiberboard and brick veneer.	50.50
BL	31	Do	Frame, wood; inside face, insulating-fiberboard lath and plaster; outside face, insulating fiberboard, wood furring strips, and wood shingles.	10.60
BM	31	Partition, nonload-bearing.	Frame, wood; both faces, insulating-board lath and plaster.	11.20
BN	31	Do	Frame, wood; both faces, interior fiberboard.	3.12

Construction symbol	BMS report	Element	Sponsor, trade name, and description	Weight, actual, based on face area.
			NATIONAL CONCRETE MASONRY ASSOCIATION	
BO	32	Wall	Units, facing, shale brick, and backing, concrete block; cement-lime mortar.	60.80
BP	32	Do	Units, concrete block; cement-lime mortar.	30.70
			WESTON PAPER & MANUFACTURING CO., "RED STRIPE"	
BQ	36	Wall	Frame, wood; inside face, corrugated double wall fiberboard lath and plaster; outside face, wood sheathing, paper, and wood-bevel siding.	8.94
BR	36	Partition, nonload-bearing.	Frame, wood; both faces, corrugated double wall fiberboard lath and plaster.	9.16
BS	36	Floor	Frame, wood; upper face, wood subfloor, paper and wood finish floor; lower face, corrugated double-wall fiberboard lath and plaster.	13.00
BT	36	Roof	Frame, wood; upper face, wood sheathing and built-up roofing; lower face, corrugated double-wall fiberboard lath and plaster.	11.50
			DOUGLAS FIR PLYWOOD ASSOCIATION	
BU	30	Wall	Frame, wood; inside face, plywood; outside face, plywood and wood shingles.	4.23
			NELSON CEMENT STONE CO., INC., "NELSON PRE-CAST CONCRETE FOUNDATION"	
BV	26	Wall	Slab, reinforced concrete	51.70
			KNAP AMERICA INC., "KNAP CONCRETE WALL UNITS"	
BW	40	Wall	Units, flanged reinforced-concrete slabs; wood splines (vertical); steel pins.	32.30
			CELOTEX CORPORATION, "CELOTEX"	
BX	42	Wall	Frame, wood; inside face, insulating-fiberboard lath and plaster; outside face, insulating fiberboard and wood-bevel siding.	9.10
BY	42	Do	Frame, wood; inside face, rigid fiberboard; outside face, insulating fiberboard and wood-bevel siding.	4.70
BZ	42	Partition, nonload-bearing.	Frame, wood; both faces, insulating-fiberboard lath and plaster.	11.10
CA	42	Do	Frame, wood; both faces, rigid fiberboard.	3.03
			PALISADE HOMES, "PALISADE HOMES"	
CB	37	Wall	Frame and outside face, wood plank (vertical); inside face, plywood.	5.24
CC	37	Partition, nonload-bearing.	Frame, wood plank (vertical); both faces, plywood.	4.20
CD	37	Do	Frame, wood; both faces, plywood.	2.10
CE	37	Floor	Frame and deck, wood plank; upper face, wood-finish floor.	7.80
			W. E. DUNN MANUFACTURING CO., "DUNSTONE"	
CF	38	Wall	Units, concrete block; cement-lime mortar.	52.60
CG	38	Do	Do	54.80
			WISCONSIN UNITS CO., "PFEIFER UNITS"	
CH	39	Wall	Units, flanged reinforced-concrete slabs; steel plates (vertical); steel bolts.	15.00

Construction symbol	BMS report	Element	Sponsor, trade name, and description	Weight, actual, based on face area.
			AMERICAN HOUSES, INC., "AMERICAN HOUSES"	
CI	47	Wall	Frame, wood; inside face, gypsum wallboard; outside face, plywood, paper, and wood shingles.	5.46
CJ	47	Partition, load-bearing.	Frame, wood; both faces, gypsum wallboard.	5.42
CK	47	Partition, nonload-bearing.	Do	5.33
CL	47	Floor	Frame, wood; upper face, plywood, paper, and wood finish floor.	7.16
			HOMASOTE CO., "PRECISION-BUILT"	
CM	48	Wall	Frame, wood; both faces, fiberboard.	*lb/ft²* 3.60
CN	48	Do	Frame, wood; inside face, fiberboard; outside face, fiberboard and wood-bevel siding.	4.38
CO	48	Partition, load-bearing.	Frame, wood; both faces, fiberboard.	3.62
			MUNLOCK ENGINEERING CO., "MUNLOCK DRY WALL BRICK"	
CP	53	Wall	Units, brick; cement mortar	76.70
			GLOBE-WERNICKE CO., "SCOT-BILT"	
CQ	46	Wall	Frame and outside face, sheet steel; inside face, wood nailing strips and sheet-steel panels.	6.84
CR	46	Floor	Frame and lower face, sheet steel; upper face, wood nailing strips, wood subfloor, and wood finish floor.	9.91
CS	46	Roof	Frame and deck, sheet steel; upper face, insulating fiberboard and asphalt roofing; lower face, wood nailing strips and gypsum wallboard.	11.90
			TILECRETE CO., "TILECRETE TYPE A"	
CT	51	Floor	Joists, rolled steel; fillers, asbestos-cement board; fill, concrete; upper face, wood finish floor.	44.20
			HERMAN A. MUGLER, "MU-STEEL"	
CU	67	Wall	Frame and inside face, sheet steel; outside face, insulating fiberboard and wood-bevel siding.	6.81
CV	67	Partition, nonload-bearing.	Frame and "inside" face, sheet steel; one "outside" face, sheet steel.	7.08
CW	67	Floor	Frame and lower face, sheet steel; upper face, wood subfloor and wood finish floor.	9.16
CX	67	Roof	Frame and lower face, sheet steel; upper face, insulating fiberboard, wood sheathing, and built-up roofing.	9.08
			PORTLAND CEMENT ASSOCIATION	
CY	62	Floor	Joists and bridging, precast reinforced concrete; upper face, reinforced concrete.	36.60
			MASONRY CONSTRUCTION SECTION, NBS	
CZ	61	Wall	Nonreinforced concrete, 1 part of portland cement, 2.71 parts of sand, and 5.31 parts gravel, by dry weight.	74.90
DA	61	Do	Nonreinforced concrete, 1 part portland cement, 2.53 parts sand, and 3.41 parts gravel, by dry weight.	75.10

Construction symbol	BMS report	Element	Sponsor, trade name, and description	Weight, actual, based on face area.
			OFFICE OF INDIAN AFFAIRS AND NATIONAL YOUTH ADMINISTRATION	
DB	78	Wall	Units, plain adobe (earth) block, molded by hand; cement-lime mortar.	121.00
DC	78	Do	Units, bitudobe (earth with admixture of bituminous stabilizer) block; cement-lime mortar.	117.00
DD	78	Do	Monolithic terracrete (earth with admixture of portland cement), rammed by hand.	157.00
DE	78	Do	Units, terracrete (earth with admixture of portland cement) block, machine pressed; cement-lime mortar.	134.00
DF	78	Do	Monolithic earth, rammed by hand.	152.00
			TENNESSEE COAL, IRON & RAILROAD CO., "U.S.S. PANELBILT"	
DG	74	Wall	Frame, sheet steel, No. 14 BWG; inside face, insulating fiberboard; outside face, galvanized sheet-steel box-rib siding and batten strips.	3.99
DH	74	Do	Frame, sheet steel, No. 11 BWG; inside face, insulating fiberboard; outside face, galvanized sheet-steel box-rib siding and batten strips.	4.47
DI	74	Partition, nonload-bearing.	Frame (similar to DG), sheet steel; both faces, insulating fiberboard.	4.01
DJ	74	Roof	Frame, sheet steel; upper face, galvanized sheet-steel box-rib roofing and batten strips.	2.68
			HOMASOTE CO., "PRECISION BUILT, JR."	
DK	72	Wall	Frame, wood; both faces, insulating fiberboard.	3.13
			PHC HOUSING CORPORATION, "PHC"	
DL	90	Wall without braces, demountable.	Frame, wood; inside face, plywood; outside face, insulating fiberboard and plywood.	6.63
DLa	90	Wall, with braces, demountable.	Frame, wood; inside face, plywood; outside face, insulating fiberboard and plywood.	6.98
DM	90	Floor, demountable.	Frame, wood; both faces, plywood.	5.93
DN	90	Roof, demountable.	Frame, wood, rafters 5⅝ in. deep; upper face, insulating fiberboard and plywood.	5.36
DO	90	Do	Frame, wood, rafters 7⅝ in. deep; upper face, insulating fiberboard and plywood.	5.97
			GENERAL SHALE PRODUCTS CORPORATION, "SPEEDBRIK"	
DP	86	Wall	Units, shale (cellular); cement-lime mortar.	41.37
			HOMASOTE CO., "PRECISION-BUILT, JR." (SECOND CONSTRUCTION)	
DQ	89	Wall	Frame, wood; inside face, insulating fiberboard; outside face, paper and wood-bevel siding.	2.99
			LEON H. WITTNER, "MULTIPLE BOX-GIRDER PLYWOOD PANELS"	
DR	99	Wall	Frames, two, wood, plywood between frames; both faces, plywood.	4.38
DS	99	Floor	Frames, two, wood, plywood between frames; both faces, plywood.	8.15
DT	99	Roof	Frames, two, wood, plywood between frames; both faces, plywood.	7.96
DU	99	Roof demountable.	Frames, two, wood, plywood between frames; both faces, plywood.	8.38

Construction symbol	BMS report	Element	Sponsor, trade name, and description	Weight, actual, based on face area.
FOREST PRODUCTS LABORATORY				
QA	25	Wall	Frame, wood; inside face, wood lath and plaster; outside face, wood sheathing, paper, and wood bevel siding.	10.00
QB	25	Floor	Frame, wood; upper face, wood subfloor, paper, and wood finish floor; lower face, wood lath and plaster.	13.20
QC	25	Roof	Frame, wood; upper face, wood sheathing, paper, and wood shingles.	4.60
QD	25	Partition, load-bearing.	Frame, wood; both faces, wood lath and plaster.	12.80

2. Weight

Specimens were weighed and values obtained were divided by the measured face areas.

For the discussion of structural properties, it was found helpful to group the constructions as wood, steel, or masonry according to material in the principal structural members.

Weights of the constructions in each group, arranged in order beginning with the lowest, are shown in figure 42.

In designing a house, weight of the construction is important because the compressive load on a wall depends upon the weight of walls, floors, and roof above. For the most economical design, therefore, other factors being equal, the construction weighing the least should be selected.

Weight has an important effect on the price of prefabricated constructions. The expense of shipping raw material to the manufacturing plant and of transporting the processed unit to the house site is a large portion of the total cost. These transportation charges limit the marketing area for prefabricated constructions. If the processed sections of the house are very large, the marketing area to be served will be smaller than it would be for the customary unassembled building materials such as lumber, brick, and cement. In many cases, prefabricated constructions have to be transported by special conveyances because they cannot safely be shipped by common carrier. For large prefabricated constructions where the weight is low the marketing area is increased if the sections are so fabricated that processed units can be assembled at the site without hoisting equipment. In general, decreasing the weight of a prefabricated construction increases its marketing area. By enlarging the marketing area, greater investment in the fabricating plant is justified and uniform production at reasonable cost is made possible by the use of highly specialized automatic equipment.

Figure 42 shows the types of construction with low weights which deserve serious consideration, especially for prefabricated houses. It, also, indicates trends which should be helpful in the development of new constructions.

3. Walls

(*a*) *Wood*

Two constructions weigh much more than other wood walls—*BJ* stucco and *BK* brick veneer. Therefore, they were not included in this study.

Weight of the other walls ranged from 3 to 11 lb/ft^2, averaging 6.04.

Except for constructions *CB*, *DL*, and *DR*, all wood walls have conventional frames of studs and plates, some with girts under transverse joints in the fiberboard or plywood faces. In all these framed walls, except for *DK* and *DQ*, the studs are 2 by 4 in., spaced 1 ft 4 in. In *DK*, the studs are 2 by 2 in. and in *DQ*, 2 by 3 in., both spaced 1 ft. These two constructions are lightest of all the wood wall constructions (about 3 lb/ft^2).

Of walls weighing less than the average, the lighter ones (3 to 5 lb/ft^2) are charcterized by faces of fiberboard or plywood. Walls with fiberboard, overlaid with either wood shingles or wood siding outside, weigh more than walls with fiberboard only on the outside. The weight of *CB* (planks vertical) and *CI* (gypsum board inside) is less than 5.5 lb/ft^2.

Of the walls weighing more than the average, *DL* is demountable in small pieces and has many steel fastenings. It has plywood inside and insulating fiberboard overlaid with plywood outside, and the weight is only 6.6 lb/ft^2. All the others have plaster inside, the weights ranging from 9 to 11 lb/ft^2, although most of them have fiberboard lath inside and fiberboard sheathing outside, overlaid with either shingles or wood-bevel siding.

Conventional wood wall *QA*, which has wood lath with plaster inside and wood sheathing with wood-bevel siding outside, weighs 66 percent more than the average. Stucco wall *BJ* and brick-veneer wall *BK* also have fiberboard lath with plaster inside.

(*b*) *Steel*

All the steel walls have sheet-steel channel-shaped structural members. Of the nine constructions, the three lightest and the three heaviest have a steel frame to which the faces are fastened. In the three intermediate constructions, the structural members are steel sheets flanged to form a one-piece face with studs. For two of these constructions, this face is outside and for the other, it is inside. Weights range from 4 to 11 lb/ft^2, averaging 6.28 lb/ft^2.

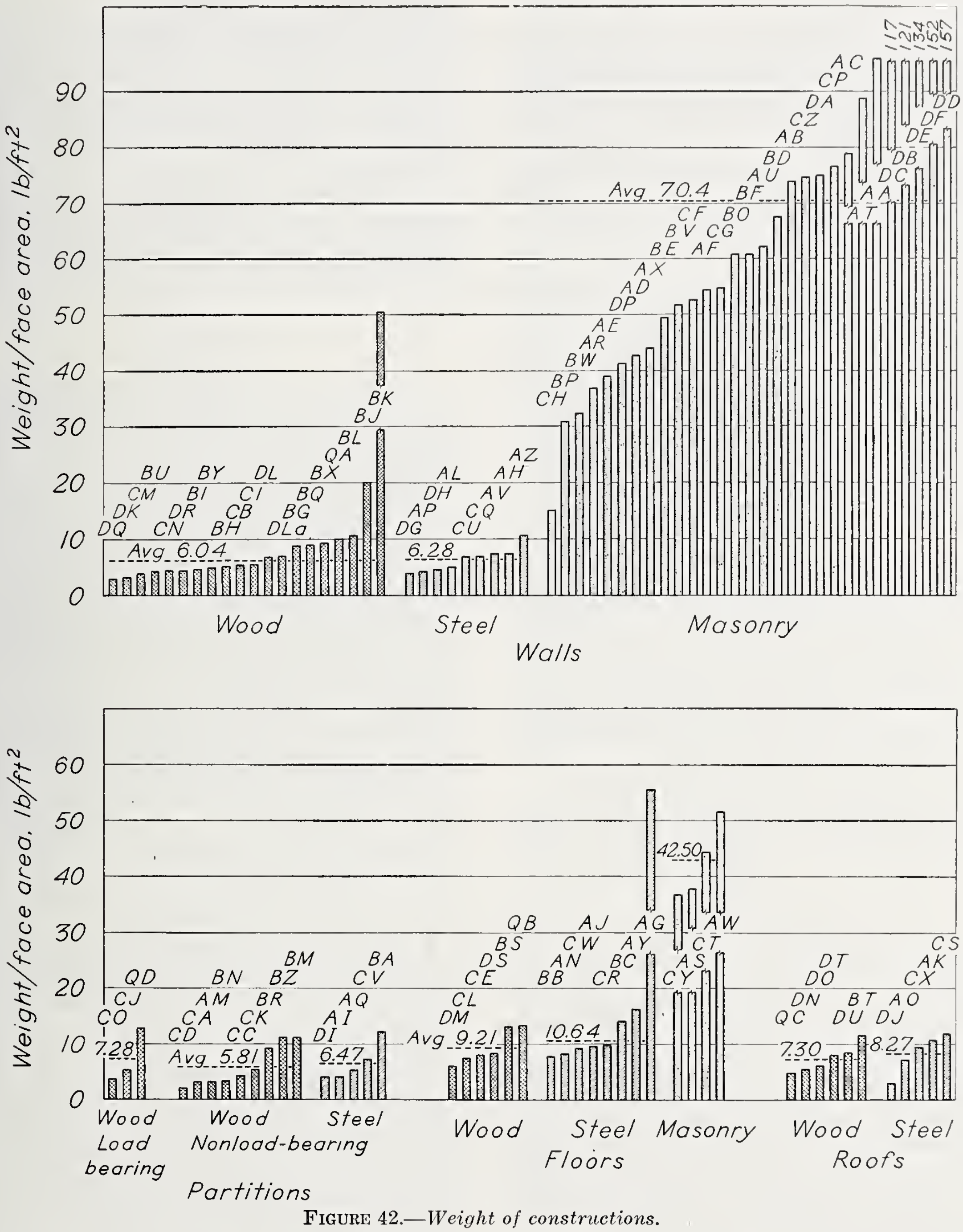

FIGURE 42.—*Weight of constructions.*

Inside faces are fiberboard on four constructions, sheet steel on three, and gypsum board on two, one of the latter being plastered.

Outside faces are sheet steel on five constructions, fiberboard overlaid with either sheet steel or wood (shingles or siding) on three, and plywood on one.

(*c*) *Masonry*

Weights of the masonry walls range from 15 to 157 lb/ft², averaging 70.4.

Of the 29 constructions, 17 weigh less than the average, the thickness ranging from 4 to 10 in. All except *BV* and *CH* have air spaces in the wall. Of these walls, four (*AU*, *AX*, *BD*, *BE*) are cavity walls; four (*AF, BF, BP, BO*) are hollow concrete block; three (*AD, AE, DP*) are structural clay tile; two (*CF, CG*) are solid concrete block laid to leave air spaces; one (*BW*) is solid concrete units connected to vertical wood splines; and one (*AR*) is a concrete slab enclosing air spaces. Of the constructions weighing less than the average, among the lighter are *CH* and *BW* (solid concrete units connected to vertical splines) and structural clay tile; among the heavier are cavity walls, hollow block, and solid block laid to leave air spaces.

The 12 constructions weighing more than the average, are all solid walls. (There are perforations through the clay units in wall *CP*.) Thickness ranges from 6 to 14 in. Among the lighter of these solid walls are monolithic concrete (6 1/16 in. thick) and brick (8 1/16 to 8 1/2 in. thick), with weights ranging from 74 to 96 lb/ft^2. The heavier solid walls are all earth constructions (11 3/4 to 14 in. thick) with weights ranging from 117 to 157 lb/ft^2.

4. Load-bearing Partitions

All load-bearing partitions have conventional wood frames of studs and plates, studs being 2 by 4 in., spaced 1 ft 4 in. Weights range from 4 to 13 lb/ft^2, averaging 7.28 lb/ft^2.

The lightest partition has fiberboard faces, the next heavier has gypsum-board faces, and the heaviest has wood lath with plaster faces.

5. Nonload-bearing Partitions

All nonload-bearing partitions have either wood or sheet-steel structural members.

(*a*) *Wood*

All wood nonload-bearing partitions, except *CC*, have conventional wood frames of studs and plates. With the exception of those used in *CD* and *CK*, studs are 2 by 4 in., spaced 1 ft 4 in. Weights range from 2 to 11 lb/ft^2, averaging 5.81 lb/ft^2.

Of partitions weighing less than the average, the lightest, *CD*, has a wood frame (studs 1 3/8 by 1¾ in., spaced 1 ft) with plywood faces, and the next three have fiberboard faces. Plank partition *CC* has plywood faces and weighs about 1.5 lb/ft^2 less than the average. Partition, *CK*, (studs 2 by 3 in., spaced 1 ft 4 in.) weighs a little less than the average.

Of the partitions weighing more than the average, all have faces of fiberboard lath with plaster and weigh from 9 to 11 lb/ft^2.

(*b*) *Steel*

All the steel nonload-bearing partitions have sheet-steel structural members. Except for *CV*, these members are channels having the webs perpendicular to the faces. For *CV*, the structural members are steel sheets flanged to form a one-piece "inside" face with studs. The weights range from 4 to 12 lb/ft^2, averaging 6.47 lb/ft^2.

Of the partitions weighing less than the average, (4 to 5 lb/ft^2), the lightest has fiberboard faces and the other two have gypsum-board faces.

Of the partitions weighing more than the average, *CV* has sheet steel on the "outside" face and the heaviest construction, *BA*, has gypsum-board lath with plaster face.

6. Floors

(*a*) *Wood*

The wood floors *BS, CL,* and *QB* have conventional frames of joists and headers, the joists being 2 by 8 in., spaced 1 ft 4 in. The other floors have unusual wood frames.

The weights of the floors range from 6 to 13 lb/ft^2, averaging 9.21 lb/ft^2.

Of the wood floors weighing less than the average, the lightest, *DM*, is demountable in pieces and has plywood panels on both faces. The next heavier, *CL*, has plywood subfloor and wood finish floor. The plank floor *CE* has a wood finish floor on the deck and *DS* has plywood on both faces.

Both floors weighing more than the average have plaster on the lower face and weigh 13 lb/ft^2. One, *BS*, has a wood subfloor with wood finish floor on the upper face and fiberboard lath with plaster on the lower face. The other, *QB* (conventional), has wood subfloor with wood finish floor on the upper face and wood lath with plaster on the lower face.

(*b*) *Steel*

All the steel floors have sheet-steel structural members. In six of the eight constructions these members are steel sheets formed into a one-piece deck or lower face with either one or two joists.

The two heaviest floors are *AG* and *AY*. Floor *AG* has a concrete fill on the cellular sheet-steel lower face and weighs more than three times *AY*. Therefore, *AG* was not included in this study. The weights of the other floors range from 8 to 16 lb/ft^2, averaging 10.64 lb/ft^2.

All the steel floors have upper faces of nailing strips, sleepers, or wood subfloor overlaid with wood finish flooring. Of the floors weighing less than the average, all except *BB* have sheet-steel lower faces. Floor *BB*, the lightest, has no lower face.

Of the floors weighing more than the average, all have lower faces of plaster on either gypsum or metal lath.

(c) *Masonry*

All the masonry floors are reinforced concrete and the weight ranges from 37 to 52 lb/ft^2, averaging 42.50.

Of the floors weighing less than the average, one, *CY*, has precast concrete joists on which a slab is poured and the other, *AS*, is a precast concrete slab having enclosed air spaces. For both, the concrete is the upper face.

Of the floors weighing more than the average, both (*AW*, *CT*) have joists of expanded rolled I-beams, fillers resting on the lower flanges of the joists, concrete fill, and wood finish floor.

7. Roofs

(a) *Wood*

The lightest and the heaviest wood roofs have conventional frames of rafters and headers (*QC*, rafters 2 by 6 in., spaced 2 ft) (*BT*, rafters 2 by 8 in., spaced 1 ft 4 in.). All the others have unusual frames. The weights range from 5 to 12 lb/ft^2, averaging 7.30 lb/ft^2.

The lightest roof, *QC* (conventional), has wood sheathing with wood shingles on the upper face and no lower face. Heavier roofs, *DN* and *DO*, are similar and demountable in pieces but have different depths of rafters. The upper face is fiberboard overlaid with plywood. The next heavier roofs, *DT* and *DU*, are similar (one demountable in panels). Both faces are plywood. The heaviest roof, *BT*, has wood sheathing with built-up asphalt roofing on the upper face and fiberboard lath with plaster on the lower face.

(b) *Steel*

All the steel roofs have sheet-metal structural members. In four of the five constructions, these members are steel sheets formed into a one-piece deck or lower face with rafters. The weights range from 3 to 12 lb/ft^2, averaging 8.27 lb/ft^2.

The lightest roof has a sheet-steel upper face and the others have fiberboard sheathing overlaid with built-up roofing. Roof *CX* has wood sheathing between the insulating fiberboard and the built-up roofing.

The lightest roof has no lower face and the heaviest has nailing strips with gypsum-board lower face. The other roofs have sheet-steel lower faces.

XVII. VARIATIONS IN STRENGTH OF HOUSE CONSTRUCTION

1. General

It is generally believed that for a given house construction there are differences in strength caused by differences in the materials and workmanship. While the constructions were being tested, visitors to the laboratory frequently commented that obviously the specimens were much better in materials and workmanship and therefore stronger than the same constructions in houses. Because all the specimens for a construction were built at the same time by the same workmen and from the same lots of material, it seemed probable that the differences in strength of the specimens were less than the differences in strength of the same constructions in houses.

The BMS reports on structural properties were studied to determine whether they would throw any light on this question. The methods for determining the compressive, transverse, concentrated, impact, and racking strengths are described in BMS2. For each construction and for compressive, transverse, and racking loads the difference between the greatest and the least maximum load was divided by the average maximum load and taken as the variation in strength.

Many constructions did not fail under the concentrated or the impact loads. For the concentrated load, the test was discontinued if the specimen did not fail under a load of 1,000 lb and for the impact load under a height-of-drop of 10 ft; therefore, no variations were computed for either concentrated or impact loads. Only concentrated and impact loads were applied to nonload-bearing partitions.

For the racking load, the test was discontinued if the specimen did not fail under a load of 50,000 lb; therefore, no racking-load variations were obtained for some constructions, mostly masonry walls.

The variations for wood, steel, and masonry constructions are given in table 17.

2. Comparison of Variations in Strength

For wood walls under transverse loading, the variation when loaded on the outside face is almost twice the variation when loaded on the inside face, probably because there were different materials in the faces. Under the racking load, the structural members (studs) do not fail, only the faces or the fastenings fail, but the variation is about one-half the variation for compressive and transverse loadings.

For steel walls, the variations are about the same for all loadings and about one-half the variation of the wood walls under compressive and transverse loadings.

For masonry walls, the variations are about the same for all loadings. Almost one-half the masonry walls are not included in the variation for racking load because they did not fail. Apparently, the variations for masonry walls

TABLE 17.—*Variations in strength of house constructions*

Element	Structural members	Compressive-load variation		Transverse-load variation				Racking-load variation		All-loads variation
				Loaded inside face		Loaded outside face				
		Range	Average	Range	Average	Range	Average	Range	Average	Average
		Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Wall	Wood	9 to 72	29	0 to 39	19	8 to 63	34	1 to 44	14	24
Do	Steel	3 to 24	13	2 to 25	9	3 to 13	7	4 to 24	14	11
Do	Masonry	1 to 53	20	5 to 92	26	5 to 92	23	5 to 60	19	22
Partition, load-bearing.	Wood	18 to 28	22	15 to 33	22	15 to 33	22	4 to 15	8	18
Floor	Do			10 to 53	37					
Do	Steel			2 to 7	5					
Do	Masonry			2 to 20	12					
Roof	Wood			6 to 43	28					
Do	Steel			2 to 14	8					

are about the same as the variations for wood walls.

All the load-bearing partitions were wood. The difference in the variations of wood partitions and wood walls probably is due to the fact that there were nineteen walls and only three partitions. For compressive and transverse loadings, the variations of the partitions and walls are about the same. The variation under racking load is about one-third the variations for these partitions under compressive and transverse loading and about one-half the variations for any of the walls under racking load.

For wood floors, the variation (transverse loading only) is greater than the variation of wood walls under transverse loading and greater than the variation of any other house element under any loading. Because of the inherent characteristics of wood the variation under transverse loading may increase as the span increases. For walls, the span is 7 ft 6 in., for floors 12 ft, and for roofs 14 ft.

For steel floors, on the other hand, the variation is less than the variations for steel walls under transverse loading and less than the variation for any other element under any loading.

For masonry floors, the variation is greater than for steel floors but less than the variations for masonry walls under transverse loading, probably because all the masonry floors are reinforced concrete and very few of the masonry walls were reinforced.

For wood roofs, the variation is about the same as the variations for wood walls under transverse loading although the span is 14 ft (not 7 ft 6 in.).

For steel roofs, the variation is about twice the variation for steel floors and a little greater than the variations for steel walls under transverse loading.

The weighted average variation for all the wood construction is 26 percent; for all the steel constructions, 8 percent; and for all the masonry constructions, 21 percent. Whether or not the variations for houses is greater or less than the variations for laboratory specimens, it seems probable that the variations for walls, load-bearing partitions, floors, and roofs in houses are about the ratio: steel 8, masonry 21, and wood 26.

3. Relation of Materials to Variations

The variations in strength depend upon the materials in the structural members (studs, joists, and rafters) and also the materials in the faces. For many masonry constructions, the same material is both a structural member and a face. The relations between materials and variations were studied to find the relation for structural members and for each face separately, although they are so interrelated that the results indicate little more than trends, which in some cases may be misleading.

(*a*) *Walls*

(*1*) *Wood*

Studs.—Except wall *CB* (plank, variation 13 percent), all wood walls have studs. Assuming that variations in strengths depend upon differences in size and species of wood of studs, the variations range from 17 to 36 percent. In order, they are Douglas fir studs less than 2 by 4 in., then all other red pine, Douglas fir, and southern pine 2- by 4-in. studs.

Inside face.—For the materials in inside face, the variations range from 10 to 32 percent. In order, the materials are wood lath with plaster, fiberboard, plywood, fiberboard with plaster, and gypsum board.

Outside face.—For the materials in the outside face, average variations range from 13 to 36 percent. In order, the materials are plank, wood siding, plywood, wood sheathing with wood siding, fiberboard with stucco, fiberboard with wood siding, fiberboard with wood shingles, fiberboard with brick veneer, fiberboard, plywood with wood shingles, and fiberboard with plywood.

(*2*) *Steel*

Studs.—For all the steel walls, the average variations are small, ranging from 8 to 12 percent depending upon the studs. The variations are least for channel-shaped members having the studs and a face in one piece (8 percent) and greatest for channel-shaped studs to which the faces are fastened (12 percent).

Inside face.—For the materials in the inside face, average variations range from 9 to 12 percent. In order, the materials are sheet steel, fiberboard, gypsum-board lath with plaster, and gypsum board.

Outside face.—For the materials in the outside face, average variations range from 8 to 18 percent. In order, the materials are fiberboard with wood siding, sheet steel, fiberboard with plywood and wood shingles, plywood, and fiberboard with sheet steel.

(*3*) *Masonry*

For all the masonry walls, average variations are nearly the same for the solid walls and those having air spaces in the wall. For those having air spaces, the materials in order are brick, structural tile on end, brick with structural tile on side, concrete block, reinforced concrete, structural tile on side, brick with concrete block, averaging 21 percent. For solid walls, the materials in order are reinforced brick, brick, earth, reinforced concrete, plain concrete, averaging 24 percent.

Inside face.—For the materials in the inside face, average variations range from 17 to 27 percent. In order, the materials are structural tile on end, brick, tile on side, reinforced brick, concrete block, reinforced concrete, earth, and plain concrete.

Outside face.—For the materials in the outside face, average variations range from 17 to 27 percent. In order, the materials are structural tile on end, concrete block, reinforced brick, brick, reinforced concrete, tile on side, earth, and plain concrete.

Apparently, the variations for earth are more than for many masonry materials widely used at present. Therefore, the average variation for each construction was obtained, and ranged from 6 to 65 percent. Adobe walls *DB* and bitudobe walls *DC* had very small variation; monolithic earth wall *DF* and terracrete block wall *DE* were very near the middle of the range; and monolithic terracrete wall *DD* had the greatest variation. Whether the differences in variations were due to differences in material or to differences in workmanship is not known.

(*b*) *Load-bearing Partitions*

(*1*) *Wood*

Studs.—The three load-bearing partitions all had 2- by 4-in. Douglas fir studs, spaced 1 ft 4 in., and average variations ranged from 16 to 22 percent.

Faces.—For the materials in the faces, the variations ranged from 16 to 22 percent, plaster with wood lath having the least variation, fiberboard next, and gypsum board the greatest.

(*c*) *Floors*

(*1*) *Wood*

Joists.—There does not appear to be any relation between the size and species of wood in the joists and the variations.

Upper face.—For materials in the upper face, average variations range from 10 to 53 percent. In order, the materials are smallest, plank subfloor with wood-finish floor; near the middle of the range, plywood and wood subfloor with wood-finish floor; the greatest, plywood subfloor with wood-finish floor.

Lower face.—For materials in the lower face, average variations range from 32 to 46 percent. In order, the materials are no lower face, plywood, wood lath with plaster, and fiberboard lath with plaster.

(*2*) *Steel*

There is no significant difference in the average variations due either to the shape of the cross-section of the joists or the materials in the faces. The greatest variation is 6 percent.

(*3*) *Masonry*

There is no significant difference in average variations attributable to the kind of reinforcement or to the materials in the faces.

(*d*) *Roofs*

For both wood and steel roofs, there is no significant difference in the average variations due to the rafters or the materials in the faces.

4. Relation of Workmanship to Variations

(*a*) *Walls*

(*1*) *Wood*

For wood walls, the average variation for all prefabricated constructions is nearly the same as the average variation for all the constructions having the pieces assembled on the house site.

To members of the laboratory staff, it appeared that there were great differences in the care and skill with which the sponsors built the specimens, but there is no relation between the average variation and the care and skill with which the specimens were built. There is some indication that constructions having the smaller variations were built with the least care and skill.

The conventional wood-frame constructions, wall *QA*, partition *QD*, floor *QB*, and roof *QC*, were built and tested at the Forest Products Laboratory, Madison, Wis. It is reasonable to believe that they were built with more care and skill than is commercially available for the construction of wood houses.

For wall *QA*, the average variation is 10 percent, and the average variation for all wood walls 24 percent; for load-bearing partition *QD*, 16 percent and for all load-bearing partitions 18 percent; for floor *QB*, 40 percent, and for all wood floors 37 percent; and for roof *QC*, 43 percent, and for all wood roofs 28 percent. There were many wood walls in the program, but few partitions, floors, and roofs.

It seems reasonable to suppose that the variation in the strength of like specimens depends upon the workmanship; the better the workmanship, i.e., the greater the care and skill with which the construction is built, the less the variation in strength. Skilled workmen make better joints and sort the material to some extent, placing each piece to the best advantage.

For wall *QA*, the variation is about half the average for all the wood walls. To determine whether there was any relation between the variation and the strength of *QA* under compressive, transverse, and racking loads, these strengths were compared with the average of the strengths of all the other wood walls having 2- by 4-in. studs of Douglas fir, No. 1, common, spaced 1 ft 4 in. For compressive load, *QA* is 27 percent greater; for transverse load, inside 8 percent and outside 19 percent less; and for racking load, 68 percent greater. Probably the great racking strength of *QA* may be attributed to the wood sheathing laid diagonally outside and the wood lath with plaster inside. Many of the wood walls did not have wood sheathing and *QA* is the only wall having plaster on wood lath. Because the racking strength depended more on the materials in the faces than upon the workmanship, only the compressive and transverse strengths should be compared. If good workmanship accounts for the greater compressive strength of *QA*, why is the transverse strength considerably less than for the other walls?

For partition *QD*, the variation is a little less than average, but the average strength is about 24 percent greater than averages of the strengths for all the other load-bearing partitions.

For floor *QB*, the variation is about the same and the strength only 70 percent of average strength for the similar floors having 2- by 8-in. joists of Douglas fir or longleaf Southern pine, spaced 1 ft 4 in.

The one other roof with which roof *QC* can be compared for strength has deeper rafters more closely spaced and of a different species of wood, therefore roofs are not included in this study.

It seems evident that for the wood constructions tested, which were built with unusual care and skill, there is no very definite relation between the variations and the strength of the constructions.

(2) *Steel*

Some steel constructions were submitted by large companies presumably having well-equipped plants skillfully managed and others by small companies having much less experience with steel-house construction. No relation was found between the experience of the sponsor and the variations in strength.

(3) *Masonry*

The specimens for some masonry constructions (reinforced concrete) were made in the sponsor's plant, but most of the specimens were built in the laboratory by local workmen.

Of the 29 masonry constructions, 13 were built under the direct supervision of the Bureau's Masonry Construction Section. For some constructions, the workmanship was purposely poor and for others it was purposely good. The weighted average (22 percent) of the variations for the constructions built by this Section is almost the same (24 percent) as the weighted average of the variations for all the other masonry constructions.

Because none of the constructions supervised by the Masonry Construction Section (masonry units and mortar) duplicated any of those supervised by the other sponsors, it is impossible to compare strengths.

Because with workmen from the building industry it has been found extremely difficult even under direct supervision to change the workmanship (such as filling all joints with mortar), it seems probable that the laboratory specimens quite accurately duplicate the workmanship of masonry houses in the District of Columbia.

If it is desired to know the strength of masonry built in other localities by other workmen, specimens should be made and tested.

(*b*) *Load-bearing Partitions, Floors, and Roofs*

Consideration of the load-bearing partitions, floors, and roofs did not result in anything which would be an addition to this discussion of workmanship.

XVIII. ALLOWABLE LOADS ON HOUSE CONSTRUCTIONS

1. General

When designing a house, the architect must know not only the loads that will be applied in service but, also, the allowable loads that the constructions will carry safely. These loads serve the same purpose as allowable or working stresses for wood, concrete, and steel in building codes.

Houses have never been designed like engineering structures. Since prehistoric times, safe house constructions have been found by the tedious and wasteful method of trial and error. If the modern research that has proved so successful in the solution of other problems had been applied to houses, not only would homes be more satisfactory as dwellings but, much more important, the cost would be much less. This would be an outstanding contribution to the problem of providing acceptable houses for the low-income groups in this country.

Any attempt to apply conventional design methods for engineering structures (tall buildings, bridges) encounters perplexing difficulties.

(*a*) *New Materials*

Strength and other properties are known for common building materials such as wood, concrete, steel, etc., but for many of the new materials advocated for house construction there is little reliable information as to strength and performance. In use for such a short time, the deterioration of these new materials under service conditions has not been definitely determined and, therefore, an architect is justified in hesitating to use them until necessary basic data are available.

The BMS reports on structural properties do give data on the strength properties of constructions built with some of these newer materials but give nothing on deterioration in service.

(*b*) *Fastenings*

The pieces of material in a house are fastened in many ways. Mortar is used for masonry units; wood constructions are fastened by nails, drive screws, and glue; sheet materials, including sheet steel and plastics, may be fastened by devices like spot welds, self-tapping screws, special rivets, and interlocking connections.

(*c*) *Unusual Structural Members*

Of late years, new and unusual structural members for houses have been advocated although little is known of their behavior under load.

An outstanding example is sheet-steel members (studs, joists) having a channel, I-beam, or other cross section. The behavior under load of rolled-steel sections (thickness 1/8 in. or more) has been established by long experience, but their use in houses is uneconomical because the loads are light. As yet there is no assurance that the methods for computing the strength of rolled sections can be applied safely to sheet-steel sections. Experience with structures of thin metal for aircraft shows that too often they fail under loads less than the computed loads by buckling where there is compressive stress. If a house wall consists of sheet-steel studs 2 in. deep and a sheet-steel face is spot-welded to the stud, the face may be considered the flanges of a tee beam when computing the transverse strength. However, under load, the face, if on the compression side, may buckle and the actual strength be much less than the computed strength. If the face is attached by drive screws or self-tapping screws, the strength may be much less than if it was attached by spot welds because under load the face and the stud do not behave as a unit.

Members built up of pieces of wood fastened by nails or glue have been suggested for floor joists. Although they can be so designed and fabricated that they are satisfactory, they have not come into general use because solid (one-piece) joists are cheaper for the spans usually required for houses.

(*d*) *Stressed-skin Constructions*

When designing large structures, it is customary to assume that walls, floors, and roof contribute nothing to the strength and stiffness of the building. All the loads, including the weight of these portions of the structure, are carried by the frame. For airplanes, on the other hand, the weight of the completed structure can be reduced greatly if the covering of the fuselage, wings, and control surfaces contribute to the strength and stiffness, that is, if they are stressed-skin construction.

Automobiles and especially streamlined trains of stressed-skin construction weigh much less than constructions of previous design and have proved satisfactory in service. Although it is probably uneconomical to carry the stressed-skin principle as far in houses as in mobile equipment because of the high cost of fabrication, this principle should receive serious consideration.

In a brick house, the walls not only exclude the weather but contribute to the strength; in fact, they are structural members.

Although it long has been recognized that, in a wood-frame house, the sheathing nailed to the studs resists most of the racking load, it was assumed that the studs carry all the compressive and transverse (wind) load and that under these loads the sheathing and siding outside and lath and plaster inside function only to exclude the weather and to provide a satisfactory appearance architecturally.

The fallacy of this assumption was shown by the Forest Products Laboratory in BMS25. Specimens with conventional faces both inside and out had about twice the compressive and transverse strength of specimens consisting only of studs with top and bottom plates.

Consideration of other structural reports leads to the belief that, in many of the constructions, the faces necessarily must function as stressed members, although inefficiently.

2. Efficient Use of Material

Wood was wastefully used from an engineering standpoint by early settlers in building cabins of logs cut from trees which had to be removed to clear the farm. When power-driven sawmills became available, wood frames having lighter members were developed and less wood was required. Because houses having joists, studs, and rafters spaced 16 inches were satisfactory, this spacing became a rigid trade practice. It exerted a paralyzing influence on frame house construction by fixing the commercial sizes of sheets of fiber, gypsum, and plywood. There appears to be a great need for the application of sound engineering principles to the development of new construction having just the necessary strength with which to use material efficiently. Undoubtedly, the trend in the future will be less material and more fabrication which may result in prefabrication in well-equipped factories. In the past, abundance of raw material has made it seem unnecessary to carry on extensive laboratory research on structures and houses or to spend the time and money to design them as carefully as they could be designed.

3. Criteria for Allowable Loads

The BMS structural reports give strength data on about 100 house constructions, both conventional and unconventional, and are an adequate sampling of the constructions available at present. The requirements for specimens and the methods of testing are described in BMS2.

A study of these data led to the following criteria for determining the allowable load. If, in the future, experience shows that other criteria are more useful, they should replace these criteria. The allowable load for each criterion should be determined and the lowest value used when designing a house.

The selection of working stresses (allowable loads) has been discussed from time to time but no entirely logical method has been developed. Salmon, in a long discussion of this subject, concludes, "What then is the real basis which should determine the working stress in a material? The answer is *successful practice.*"

To avoid misunderstandings, it is necessary to emphasize that these criteria apply only if the other portions of this report are followed.

(*a*) *Design Loads*

If the maximum load on a house is definitely known, the allowable load on the construction should be greater than if the maximum load is uncertain or if the assumed maximum load may be exceeded greatly under conditions which may occur in the life of the structure.

The following criteria, particularly criterion a for the allowable load on a construction, are only justified if the design loads on the house are the greatest which in any likelihood will occur.

The design live load on a floor (40 lb/ft^2) may be approached at a large gathering if the room is filled with people most of whom are standing. A greater load is very unlikely because it would automatically be prevented by lack of floor space.

If the design wind load and the snow load obtained in accordance with sections IV and V of this publication are not the greatest which occur in the life of the house, then the observed Weather Bureau data should be extended by more recent values at existing and additional stations.

The house is designed for all loads to be applied at the same time when, actually, the occurrence of the design value for any one of them is unlikely and the occurrence of all design loads simultaneously is highly improbable. It is very difficult to conceive of the greatest snow load on the roof occurring when the wind is blowing a gale or of there being no standing room on any of the floors during a record breaking blizzard.

Experience may show that a house is safe if designed for a value less than the sum of all the design loads. The law of probability may indicate that one-half or two-thirds of the sum is a suitable value.

(*b*) *Design of Walls*

For design, the compressive, transverse, and racking loads are carried by the portions of the wall between openings; that is, by the net width of the wall, not the gross width.

(*c*) *Construction*

Allowable loads apply only to constructions for which the strength has been determined by test and then only if the construction in the house is as strong as the specimens.

Because the specimens each have several structural members, the test results are necessarily averages and more representative of the strength in a house than the results of tests on individual members or other pieces.

In the manufacture of automobiles, guns, and electrical equipment, methods have been developed for determining the dimensions of each part so that they are interchangeable. The strength and wear are controlled through chemical composition and heat treatment. Often defective parts are eliminated by nondestructive tests such as magnaflux and X-ray.

In the house building industry, on the contrary, it is customary to rely on plans and specifications. Often the latter describe in rather general terms the materials and workmanship, leaving to the builder considerable latitude for the exercise of independent judgment.

Sometimes the specifications for the house state that the material shall comply with ASTM or other specifications, but the material delivered is seldom tested for compliance. Practically all specifications require first-class workmanship or workmanship acceptable to the inspector but, with the exception of dimensions, there are no quantitative measures of workmanship. This is particularly true for constructions of masonry units and mortar.

It is evident that for houses there are difficulties in determining whether or not a construction in a house has the strength of laboratory specimens. If a house is built commercially in accordance with the description in a BMS report, there are some uncertainties about the strength. If the wood is graded visually by a competent inspector and masonry units and mortar tested for compliance with the values in the BMS report, there is less uncertainty, but this leaves the workmanship in doubt, particularly because the reports give almost no information about the workmanship.

If, on the other hand, specimens of the construction made under the same conditions as the house are tested in a laboratory, the strength is known. Obviously, it is unnecessary to test specimens for each house; for constructions built frequently, several tests a year should be sufficient.

For this report, when determining allowable loads, it was assumed that the constructions in the house are as like the specimens as is commercially practicable and that the strength is very nearly the strength of the specimens.

If specimens of the actual construction are tested, the percentages for criterion a may be increased and if on the other hand, there is no inspection and the owner or the building official believes that the contractor either cannot or will not exercise the care and skill characteristic of the best commercial practice, he is perfectly justified in fixing a lower percentage. How much lower, must necessarily be determined by individual judgment based on a knowledge of the conditions.

During discussions of specifications and codes for structural steel in buildings, it has been emphasized repeatedly that no design stress, however low, can be set up which will assure the safety of a building designed and erected by ignorant, careless, and unskilled contractors. Obviously, this also applies to houses.

The same allowable load applies, despite minor changes in the construction, unless there is reason to believe that the change decreases the strength. The application of plaster to the inside face of a masonry wall should not decrease the strength, but the use of thin fiberboard sheathing instead of wood sheathing on a wood-frame wall might decrease the racking strength and there is no way to estimate how much except by testing the wall.

(*d*) *Least Maximum Load*

It is obvious that the working load cannot exceed the maximum load on the specimens. The maximum loads for the three specimens loaded in the same way were not exactly the same. It may be assumed that the differences were caused by differences in both materials and workmanship. Therefore, the working load for the construction should not exceed the least of the three maximum loads.

Either greater or smaller least maximum loads should be substituted for the values in the reports if and when other values are determined by tests.

(*e*) *Criterion* a, *Percentage of Least Maximum Load*

In accordance with good engineering practice, the allowable load should be somewhat less than the least maximum load and this ratio may be expressed in percent. For a particular construction, the greater the ratio of the allowable load to the maximum test load the more efficient the use of material and the less the cost of the house, but only provided the allowable load is not appreciably greater than the design load. It is uneconomical to use a 2-in. bolt if a ½-in. bolt is strong enough.

Logically, there should be a ratio for each construction based on experience, but, for convenience, it is customary in the building industry to group similar constructions for design purposes and when applying building code regulations. Therefore, the ratios recommended here are estimates applying to all the constructions in the groups wood, steel, and masonry. They are values selected by engineering judgment based on experience with the constructions which have been widely used for years.

The values for criterion a are given in table 18.

TABLE 18.—*Values for criterion* a

Load	Wood	Steel, reinforced brick, or reinforced concrete	Masonry
WALLS AND LOAD-BEARING PARTITIONS			
	Percent	*Percent*	*Percent*
Compressive	60	75	40
Transverse	90	75	40
Concentrated	90	90	90
Impact	90	90	90
Racking:			
Faces—			
Wood (sheathing)	90	75	40
Other materials	60		
NONLOAD-BEARING PARTITIONS			
Impact	90	90	90
Concentrated	90	90	90
FLOORS			
Transverse	60	75	40
Concentrated	90	90	90
Impact	90	90	90
ROOFS			
Transverse	65	75	40
Concentrated	90	90	90

(*1*) *Wood*

When selecting the criteria for wood, consideration was given to the observed behavior of wood constructions, that under heavy service load minor adjustments or movements occur between the parts, particularly the structural members, but that the effect on the strength of the construction is negligible. This behavior is very marked at bearings if the forces are perpendicular to the grain of the wood as in plates over and under the ends of studs and at the ends of joists and rafters where they bear on walls.

The strength of wood increases as the moisture content decreases while the wood seasons. The relation for small clear specimens is given in Forest Products Laboratory Report No. 1313 [27]. When computing the strength of wood structures, it is customary to take the strength of unseasoned material. The moisture content of structural members of the wood house constructions averaged 12 percent. Therefore, the actual strength was greater than for unseasoned material and greater than the strength computed by the usual methods.

Peck [28] found for six cities, widely scattered in this country that, over a period of 16 months, the moisture content of interior woodwork did not exceed 12 percent and of exterior woodwork 14 percent. The only exception was New Orleans where the values were 13 and 16 percent.

The moisture content of the wood in a house varies somewhat with the season of the year but the greater values are only for a few months and for the season when the wind loads and snow loads, if any, are small.

The moisture content of the BMS constructions average 12 percent which is the value recommended in Wood Handbook [29] for most of this country when designing wood buildings.

The strength of wood depends upon the length of time the load is applied—the longer the time the less the strength, as brought out by Markwardt and Wilson [30]. When selecting the criteria, the ratio of the strength of walls, floors, and roofs in a house to the strength found in the laboratory can be estimated by considering the time each of the loads is applied.

Wood walls and partitions.—For compressive loads, the long-time loads (1 year or more) on the first-story wall of a two-story house may be as much as half the design load. The long-time load is much less in a one-story house, but for the criterion only the more severe condition was considered.

The transverse load (wind only) is applied to a wall for a few hours at most.

The conditions under which an impact load is applied to a wall were closely simulated by the impact tests in the laboratory; therefore, the duration of the load was not considered. A large value is justified because any damage is local and cannot cause the house to collapse. The effect of the concentrated load is still less serious.

For the racking load, if the sheathing or other members resisting the load (wind only) are wood, the duration of the load is the same as for transverse load and the criterion should be the same. Many constructions having wood structural members (studs) have faces of fiberboard or other materials and a smaller value seems desirable until more information is available.

Wood floors.—The long-time transverse loads on a floor may be as much as half the design load; therefore, the value is the same as that for compressive load on walls.

If there are several heavy permanent loads in line along the joists, such as heavy bookcases and kitchen or bathroom fixtures, the increase in ratio of long-time load may make it advisable to provide additional joists under these loads.

The values for concentrated and impact loads are the same as for these loads on walls and partitions and for the same reasons.

Wood roofs.—In the northern United States there may be snow on the roof for months, but the duration is shorter than for the long-time compressive loads on walls or transverse loads on floors; therefore, the criterion should be greater. If, on the other hand, the only appreciable transverse load applied on the roof is the effective wind pressure, a still greater value is justified, for some locations as much as 85 percent.

(2) *Steel*

When selecting the a criteria for steel, consideration was given to the ductility of the steel sheets from which house constructions are fabricated. The sheets must be ductile to permit cold forming of the members. It also permits members that are slightly bent in handling to be straightened cold without injury. If one member is more heavily loaded than adjacent members and the load reaches the yield point, the member deforms plastically without a decrease in the load and the load on adjacent members increases, thus distributing the load more uniformly under subsequent loadings. After being deformed slightly, the member behaves under load practically as it did before the deformation occurred. However, local dents in members reduce the compressive load at failure. This load for these formed members, even if undamaged, usually is much less than the yield strength of the material; therefore, the ductility of the material has little effect on the failure which occurs suddenly by buckling.

For steel house construction, the duration of the load has no effect on the strength and there is no fatigue effect. Corrosion may decrease the strength by decreasing the thickness especially because the members are very thin. Corrosion sufficient to decrease the strength of the structure has never been found in hotels or office buildings. Conditions in steel houses may be more severe, therefore, adequate protection is advisable to insure a satisfactory service life. Inspection of sheet-steel houses indicates that in some cases the appearance had suffered through lack of painting but there was little structural damage. If in the future, it is found that these criteria do not provide a satisfactory life, it may be necessary to take lower values although this is an inefficient way to solve the problem. Greater protection of the steel against corrosion and possibly improved maintenance are more satisfactory solutions.

The strength properties of steel are more uniform than those of any other material for houses and the dimensions much nearer the nominal size, particularly if prefabricated in a well-equipped shop.

The National Emergency Specifications for the Design, Fabrication, and Erection of Structural Steel for Buildings, [31] Section 10 requires that, in the calculations for the design, the stress be 24 kips/in.^{2} for structural steel in tension and, for some cases, in compression. The structural steel shall comply with the requirements for A–7 Structural Steel [32] (yield strength, minimum, 33 kips/in.^{2}). The ratio of the allowable stress to the yield strength, therefore, is 73 percent but this ratio applies to steel structures designed by the usual engineering methods. That the actual stresses in the completed structure under load may be appreciably greater or less than the computed stresses is recognized by engineers. Few complete steel structures have been tested by loading to failur although many individual members have been tested. It is true that the specimens of house constructions were not completed houses but they were large portions of a house. Because the uncertainties in the strength of house constructions which have been tested are less than the uncertainties in the strength of designed steel structures, the criterion should be somewhat greater for compressive, transverse, and racking load.

The effect of an impact load is local and does not endanger the house and that of a concentrated load is of negligible structural importance, therefore large values for these loads are justified.

(3) *Reinforced Brick and Concrete*

The strength of reinforced brick and concrete constructions are not greatly affected by the duration of the load; also, the concrete protects the steel reinforcement from corrosion. The effect of local damage to the reinforcement is very much less than in steel constructions. On the other hand, reinforced concrete constructions lack the unity of steel constructions. That is, the concrete and the steel under the loads to which a house is subjected in service do not act together as completely as the members of a steel house.

(4) *Masonry*

When selecting the a criteria for masonry, consideration was given to the inherent property of masonry (whether monolithic or built of units) that the thickness is usually much greater than for other constructions; that the components are united in a monolithic mass which acts as a unit under load, therefore, the loads are more uniformly distributed; and that the support of the foundation is uniformly distributed. Masonry materials suitable for houses are readily available in most localities and they are very durable.

On the other hand, if a portion of masonry structure is subjected to forces which approach the strength of the materials, they do not deform plastically to any great extent and distribute the forces to adjacent material, but failure occurs locally. Forces are applied locally by temperature changes, settling of foundations, and settling around openings, such as doors and windows. These concentrated stresses may be twice the average stress. Moreover, the transverse, impact, and racking strength of unit masonry depends greatly upon the rate of absorption of the units and the mix and consistency of the mortar, and it has not been found economically practicable to control these factors closely even in the best commercial practice. Likewise, the strength of concrete depends greatly upon the proportions of aggregate, cement, and water and upon the placing of the concrete.

No floor or roof of masonry without reinforcement was submitted for the programs on structural properties. Presumably, a vaulted or barrel-arch floor or roof comes in this category, but it is difficult to visualize the use of either in an ordinary house; therefore, the **a** criterion for transverse load on masonry floors and roofs has no useful significance.

The effect of an impact load is local and does not cause the house to collapse. Similarly, the effect of a concentrated load has no structural significance.

(*f*) *Criterion* **b**, *Damage*

For many constructions, damage was observed under loads considerably less than the maximum, often less than one-half. Therefore, the allowable load should not exceed the least load damaging any one of the specimens.

Damage is taken as either cracking or breaking of any piece in the specimen also cracking or breaking of fastenings or joints.

Damage may be classified as either structural damage that affects the strength of the construction or utility damage that affects the appearance or usefulness of the house as a dwelling; but, it is difficult, if not impossible, to determine whether a particular damage is one or the other.

If, for the load at which the damage occurred, there is a perceptible change in the slope of the deformation or set curves in the graphs, it is reasonable to consider it structural damage. Often there is no perceptible change.

For reinforced concrete, small cracks perpendicular to the reinforcement are not considered structural damage by engineers and visible buckling of faces (plywood or sheet steel), which affects appearance only and probably disappears when the load is removed, should also be disregarded.

However, a crack in a masonry wall either in the units or in a mortar joint is structural damage as is splitting or breaking of wood members (studs, joists, rafters) whether or not the effect is visible on the load graph. Likewise, the failure of a fastening (nail, screw, bolt, or spot weld) is structural damage although the effect on the maximum load may be negligible.

For most of the constructions having plaster on a face, cracks were plainly visible under loads very much less than the maximum, particularly if the transverse load was applied to the opposite face causing tensile stresses in the plaster. For this report, cracks in plaster were taken as damage and for many constructions determined the allowable load particularly because, as shown in BMS25, the plaster contributed to the strength of the construction. The fact that similar wood constructions, particularly floors with plaster on the ceiling, have been used extensively for about 100 years and have proved safe for much higher loads raises the question of the structural significance of cracks in plaster. Although deflection caused by loading undoubtedly is one cause of cracks, there are others, such as the settling of foundations and the shrinking and swelling of wood members with variations in moisture content. If, contrary to usual engineering practice, the allowable load may safely exceed the load causing damage, criterion **b** should be disregarded and the allowable load based on the other criteria.

(*g*) *Criterion* **c**, *Slope of Curve for Both Deformation and Set*

In addition to the strength of a house construction, the deformation under load and the set (permanent deformation) must be considered when determining the allowable load. Therefore, load-deformation and load-set graphs are shown in all the BMS reports on structural properties.

It is customary to obtain load-deformation (or stress-deformation) graphs for engineering materials and structural members by applying the load in increments and measuring the deformation. For many metals, the set can be determined with sufficient precision from the load-deformation graph; as, by the "off-set" method for determining the yield strength of metals [33]. Because the elastic behavior of house constructions was unknown when the BMS structural reports were prepared, it was considered desirable to obtain the set by applying the load in increments, measuring the deformation, removing the load, and measuring the set. Although this procedure increased somewhat the time required to make the tests, it provided needed information on the elastic behavior of the constructions. In addition, it subjected the specimens to cycles of loading and unloading which simulated the cycles of loading applied to a house. If the load had been increased continuously to the maximum, partial failure of some of the members or of the fastenings might not have an appreciable effect on the behavior of the specimens because of the friction between the members caused by the load. It is conceivable that the maximum load might have been greater if the specimen had been loaded continuously.

Because for most engineering structures the set is negligible for loads less than the so-called elastic limit, it seemed probable that this also was true for house constructions and that the load-set graph would be helpful when determining the allowable load. These graphs show large sets for many constructions, sometimes one-half the deformation. This condition may have been caused by plastic material such as rammed earth, described in BMS78, to friction between the faces and the structural members in a framed construction, or to permanent deformation (bending) of the fastenings. The significance of the set readings for wood-frame construction is discussed in BMS25.

If they serve no other purpose, the load-set graphs show that many constructions that have given satisfactory service are not elastic, however violently this jolts our engineering concepts.

For most engineering structures, the load-deformation curve is a straight line below the elastic limit, then the deformations increase much more rapidly than the load. The allowable load is taken somewhere on the straight portion of the curve. On the contrary, the curves for many house constructions have no straight portion, even approximately. No straight portion should be expected for the impact load because, theoretically, the curves should be parabolas.

However, the criterion can be applied that the allowable load should not exceed the value for which the deformation and the set increase greatly for a small increase in load.

Johnson [34] says the French Commission defined the "apparent elastic limit" as "the load per square millimeter of the original section, where the deformation begins to increase sensibly with no increase in the external force applied (corresponding to the dropping of the beam in testing machinery)." He points out that for most materials there is no such point other than the ultimate strength and since for these materials an elastic limit corresponding to sensible deformations is required for practical purposes he proposed for all elastic materials and to be universally used in all kinds of practical tests the following: "The apparent elastic limit is the point on the stress-diagram of any material, in any kind of test, at which the rate of deformation is fifty percent greater than it is at the origin."

Johnson states that for certain stones and concretes the stress graphs are reverse curves and no kind of "elastic limit" can be attributed to them.

Some of the load-deformation graphs for house constructions are also reverse curves (BMS30, 31, 47, 48, and 67), particularly the graphs for load-shortening under compressive load. It is probable that the rate of deformation (shortening) near the origin was great because the members (studs and plates) were not in intimate contact; then, as the load increased, the rate was less until as the maximum load was approached the rate of deformation again increased. Undue consideration should not be given to the portion of the curve for small loads. Instead of considering the rate of deformation at the origin, it is suggested that the least rate be taken.

Johnson's "apparent elastic limit" applies to engineering materials and gives a value for which the deformation is very small. For house constructions, on the contrary, the deformation under the allowable load may be considerable without impairing the usefulness of the house; therefore, the allowable load may be taken as the load at which the rate of deformation is three times the least rate. This criterion, also, may be applied to the set curve and the lower of the values obtained from the deformation curve and the set curve taken as the allowable load. The deformation curve and the set curve are so related that a load which causes a marked increase in the rate of deformation also causes a marked increase in the rate of set. If the rate for either curve has increased to three times the least rate, engineering judgment leads to the conclusion that material is being rapidly deformed plastically, and that failure is approaching.

Difficulty was found, however, in applying this criterion to the set curves, perhaps because they were close to the axis of the loads. Most of the allowable loads determined from the set curves were very much greater (average 40 percent) than those determined from the deformation curves. Therefore, this criterion was applied only to the deformation curves.

The most convenient method of measuring the rate of deformation from a graph is to consider the angle which the tangent to the curve at a given point makes with the load (vertical) axis. The rate of deformation is the tangent of this angle.

(*h*) *Criterion* **d**, *Structurally Significant Deformation*

The strength of walls was determined separately under compressive and transverse loading. In a house, these loads are applied simultaneously and the deflection of a thin wall caused by the transverse load might result in failure. Specimens of thin walls should be tested under combined loading.

Until such tests are made, it appears advisable to take an allowable transverse load which does not deflect the wall more than one-sixth the actual thickness of the structural members nor cause a set of half this distance.

The deflection caused by an impact load is instantaneous, therefore, it is unnecessary to limit the deflection but the set should not exceed one-twelfth this thickness because if applied repeatedly the sets may become great. Unlike compressive and transverse loads, an impact load affects only a small portion of the wall which is supported laterally by adjacent portions.

For *walls* the **d** criteria are

Transverse load—

- **d1** lateral deflection, one-sixth the actual thickness of the structural members
- **d2** set, one-twelfth the thickness

Impact load—

- **d2** set, one-twelfth the thickness

(*i*) *Criterion* **e**, *Deformation Affecting the Utility of a House*

In addition to the deformation affecting the structural integrity of a house there are deflections and sets affecting the utility of the house as a dwelling. Recommended values are given in table 19.

TABLE 19.—*Values of criteria* **e**

Element	Compressive load	Transverse load	Concentrated load	Impact load	Racking load
Walls and load-bearing partitions:	*in.*	*in.*	*in.*	*in.*	*in./8 ft*
e1, deflection	1.00	1.00		1.00	1.00
e2, set	0.50	0.50	0.10		0.50
e3, shortening	.50				
e4, set	.25				
Nonload-bearing partitions:					
e1, deflection				1.00	
e2, set			.10		
Floors:					
e1, deflection		2.00		2.00	
e2, set		1.00	.10	1.00	
Roofs:					
e1, deflection		4.00			
e2, set		2.00	.50		

Deformations.—There are doors and windows in the walls and doors in the partitions. Although the house is subjected to the allowable loads very infrequently, the resulting deformations should not break windows, especially the glass, nor damage either doors or windows so that they do not function reasonably well when load returns to normal.

Ordinarily these closures do not fit the frames closely, therefore a wall or partition may deform considerably before appreciable load is applied to the closure. As the deformation increases, the closures deform elastically before appreciable permanent damage occurs. Until more definite information is available, it is recommended that the permissible lateral deflection of walls and partitions be taken as 1.0 in. for compressive, transverse, and impact loading. Under racking load, the openings in walls and load-bearing partitions deform in their plane, the top being displaced horizontally with respect to the bottom, and may exert compressive forces in the plane of the windows and doors; the permissible deformation should be taken as 1.0 in.

The shortening of walls and partitions under compressive loading should be taken as 0.5 in. Usually the height of doors and windows is less than the story height, therefore the deflection and shortening at the opening is proportionally less than for the wall or partition. Also, the walls and partitions are subjected to considerable compressive load before the doors and windows are fitted to the frames, therefore, under the allowable compressive load the shortening of the openings is considerably less than the total shortening of the walls and partitions. Because there is no impact load and very little wind load when the closures are fitted, this is not the case for deflections under transverse and impact loads nor for deformation under racking load.

If for a particular construction the deformation under the allowable load damages conventional doors and windows, the architect is at liberty to design closures which are not damaged either by the deformations suggested here or by greater deformations.

For floors under transverse and impact loading, the deflection under the allowable load does not affect the strength, provided the shortening of the floor due to the deflection does not decrease the bearing area on the walls enough to cause crushing failure or enough to cause the floor to drop off the supports. For a wood floor 12-ft span, a deflection of 2 in. shortens the floor about 1/16 in., which is a movement of 1/32 in. at each end. This is less than tolerances customary in the building trades for many constructions. It has been customary in engineering design to limit the deflection to 1/360 the span, not only for houses but also for other buildings. If this value is not exceeded, it is assumed that plaster will not crack.

Although this value for the permissible deflection of plastered constructions has given reasonably satisfactory results, it appears to be a rule-of-thumb requirement unsupported by convincing technical data. If a construction tested for structural properties has plaster in the faces, it is unnecessary to limit the deflection to 1/360 the span because the test load causing the plaster to crack is reported.

The limit for deflection which does not crack plaster should not be applied generally to shop and mill buildings in which there is no plaster because of the mistaken impression that the building is unsafe if this deflection is exceeded. Insofar as utility is concerned, if there is no plaster in a house, the deflection is limited only by the value having a definite detrimental effect, physical or mental, on the occupants. The deflection of floors in boats and summer camps is often much greater than is customary in houses. Because foundations settle, floors in older houses are sometimes out of level an inch or more. There is however no record of any relation between the deflection or sagging of floors and the well-being of the occupants. It is evident that permissible deflection is a matter of personal opinion. Therefore, for economy in construction, permissible deflections should be large. Almost never is a floor actually loaded to 40 lb/ft^2 over its entire area and if, under this load, the deflection is 2.0 in., when the load returns to normal (perhaps 8 lb/ft^2) the deflection is much less.

For most conventional floors, the deflection under the allowable load determined by these criteria is much less than 2.0 in.

Many people do not distinguish between the deflection of a floor and the vibration under moving or impact loads. From an engineering viewpoint, however, there is no direct relation between the deflection under a static load and either the period or amplitude of vibration under dynamic load. At present, there is no information to guide the architect on the permissible vibration of floors. In general, both the deflection and the vibration can be decreased, but only at an increase in price.

The problem can be solved practically by making houses available with floors having widely different depth of joists to sell or rent at amounts proportional to construction costs. The occupants could then appraise the floor in terms of dollars and cents and might conclude that it was not worth the price to prevent the tinkling of crockery on the pantry shelf.

Under repeated applications of the increasing test loads, the permanent set should not exceed 1.0 in., which amount is hardly noticeable. This is evidenced by the fact that the floors of many summer cottages and old houses are out of level this much, probably because the foundations have settled slightly. This does not, however, indicate that the building is unsafe for occupancy.

If the partitions are transverse to the floor joists, it is unlikely that the doors will be damaged by deflection of the floor; but if the partitions are parallel to the joists, doors may be damaged if the deflection of the floor is excessive, particularly if the partitions are load-bearing. A door frame at midspan of the floor is not deformed by the deflection, but, as the distance between the door and midspan increases, the racking deformation of the door frame increases. The location of doors should be considered when designing the house and the doors fitted with clearances above and below which prevent damage.

For roofs, the transverse loads are wind and snow (short-time loads) and, unless leaks occur, the deflection may be great without affecting the utility of the house provided the roof is draining. The deflection causing leaks depends upon the construction and materials but there is almost no information on this subject.

The set for deflections, deformations, and shortening has an indirect relation to the utility. If the sets are great, they are additive under repeated applications of the allowable load and the house may fail. For many satisfactory house constructions, the sets are great; therefore, the permissible set is taken as one-half the corresponding deformation except for concentrated loads.

The effect of the concentrated load on a disk 1.0 in. diameter is local and only decreases the utility of the house if the disk punches a hole or the depression after the load is removed (set) retains dust and disease germs detrimental to health. For all constructions except roofs, the permissible set for concentrated load is taken as 0.1 in. For roofs, the set should not cause leaks and is taken as 0.5 in. This value is too great for clay tile and perhaps for tin roofing and there is a need for tests to determine for each kind of roof the set which causes leaks.

4. Loads

The values for the allowable loads on walls are given in table 20, on load-bearing partitions in table 21, on nonload-bearing partitions in table 22, on floors in table 23, and on roofs in table 24.

TABLE 20.—*Allowable loads on walls*

[Values for criterion **a**, percentage of least load, are given in table 18. Values for criterion **b** are the loads at which damage occurs (See p. 82). Values for criterion **c**, slope of curve for both deformation and set, are three times the least slope (See p. 82). Values for criterion **d**, structurally significant deformations, are given on page 84. Values for criterion **e**, deformation affecting utility of a house, are given in table 19]

Construction symbol	Compressive load; height, 8 ft		Transverse load; span, 7 ft 6 in.				Concentrated load; diameter of disk, 1 in.				Impact load; span, 7 ft 6 in.; weight of sandbag, 60 lb				Racking load	
			Inside face		Outside face		Inside face		Outside face		Inside face		Outside face			
	Load	Criterion	Load	Criterion	Load	Criterion	Load	Criterion	Load	Criterion	Drop	Criterion	Drop	Criterion	Load	Criterion
WOOD																
	Kips/ft		lb/ft^2		lb/ft^2		*lb*		*lb*		*ft*		*ft*		*Kips/ft*	
BG	5.84	a	75	b	75	b	191	a	210	b	1.0	b	1.0	b	0.87	c
BH	3.00	b	120	d1	115	d1	215	e2	558	a	2.6	e1	3.2	e1	1.00	c
BI	2.50	b	100	d1	115	d1	127	a	558	a	2.2	e1	2.5	b & e1	0.88	c
BJ	3.50	b	97	b	70	b	248	a	900+	a	2.0	b	1.0	b	1.50	c
BK	3.30	a	50	b	64	b	202	a	900+	a	2.5	b	7.0	b	4.50	c
BL	3.71	a	75	b	60	b	225	a	650	b	1.0	b	2.5	b	1.04	c
BQ	4.00	b	90	b	50	b	248	a	400	b	1.5	b	0.5	b	0.60	c
BU	2.90	a	80	d1	90	d1	575	e2	680	e2	2.0	e1	2.1	e1	.88	b
BX	4.77	a	90	b	50	b	277	a	300	b	1.0	b	1.0	b	.97	a
BY	1.90	b	106	d1	100	d1	91	a	450	a	2.0	b & e1	1.5	b	.78	a
CB	3.50	c	50	d1	50	d1	459	a	900+	a	1.8	e1	1.6	e1	.13	c
CI	2.53	a	115	d1	108	d1	167	a	450	e2	1.0	b	1.5	b	.80	b
CM	2.00	c	100	b	120	d1	202	a	225	a	1.3	e1	[a]1.3 [b]2.0	e1 b	.34	a
CN	2.95	a	135	d1	125	d1	180	e2	200	b	2.0	e1	2.0	b	.47	a
DK	1.62	a	20	d1	20	d1	207	e2	207	e2	0.8	e1	0.8	e1	.69	a
DL	2.16	a	40	b	70	b & d1	495	a	700	e2	3.5	b & e1	2.8	e1	[c].03 [d].16	c c
DQ	3.27	a	50	d1	45	d1	202	a	357	a	1.5	b	1.8	e1	.49	a
DR	6.61	a	55	d1	55	d1	900+	a	900+	a	4.5	e1	4.5	e1	4.20	c
QA	2.75	b	96	d1	26	b	100	b	900+	a	2.5	b	1.5	b	0.50	b
STEEL																
AH	5.75	a	166	a	166	a	520	e2	520	e2	2.0	b	2.0	b	0.94	a
AL	4.74	a	108	a	100	a	225	a	178	e2	3.6	d2 & e1	2.7	d2	.41	a
AP	—	—	71	a	82	a	130	a	720	a	0.4	d2	2.0	b	—	—
AV	11.30	a	200	d1	220	d1	138	a	850	e2	1.0	b	2.1	e1	[c].40 [d].81	c e2
AZ	6.47	a	126	a	110	b	295	a	760	e2	2.0	b	2.0	b	.60	c
CQ	2.30	c	120	b	110	d1	320	e2	220	e2	1.8	d2 & e1	1.7	e1	.30	b
CU	4.50	b	254	a	202	a	485	e2	512	b	6.5	e1	3.2	c	1.54	a
DG	3.58	a	60	d1	52	d1	87	a & e2	430	e2	2.0	b	2.5	e1	0.33	c
DH	3.42	a	69	d1	59	d1	86	a & e2	510	e2	2.0	b	2.8	e1	.25	b
REINFORCED BRICK AND CONCRETE																
AR	54.0	a	82	a	240	b	900+	a	900+	a	3.0	b & c	5.0	b	4.41+	a
AT	106.0	a	100	c	100	c	900+	a	900+	a	9.0	c	9.0	c	—	—
BV	18.0	b	289	a	170	b	900+	a	900+	a	4.0	b & c	3.0	b	3.00	b
MASONRY																
AA	99.6	a	46	a	46	a	900+	a	900+	a	4.7	c	4.7	c	2.50+	a
AB	21.0	a	15	a	15	a	900+	a	900+	a	1.9	c	1.9	c	2.50+	a
AC	36.2	a	32	a	32	a	900+	a	900+	a	3.0	c	3.0	c	2.50+	a
AD	18.0	a	14	a	14	a	900+	a	900+	a	0.6	c	0.6	c	1.42	a
AE	9.5	a	23	a	23	a	900+	a	900+	a	1.1	c	1.1	c	1.37	a
AF	15.3	a	12	a	12	a	900+	a	900+	a	0.5	c	0.5	c	1.20	a
AU	10.6	a	7	a	10	a	900+	a	900+	a	2.0	b	1.3	c	2.01	a
AX	14.4	a	19	a	19	a	900+	a	900+	a	1.5	b & c	1.5	b & c	2.36	a
BD	[e]24.7 [f]19.3	a a	9	a	9	a	581	a	581	a	1.5	b	1.5	b	1.98	a
BE	16.9	a	8	a	7	a	900+	a	900+	a	0.6	c	0.5	b	0.79	a
BF	13.6	a	18	a	16	a	900+	a	900+	a	1.5	c	1.5	c	2.08	a
BO	13.0	a	22	a	14	a	900+	a	900+	a	1.5	c	1.1	c	2.20	a
BP	7.8	a	3	a	3	a	900+	a	900+	a	0.5	b	0.5	b	0.64	a
BW	7.9	a	77	a	77	a	900+	a	900+	a	1.0	b	1.0	b	1.50	b
CF	15.6	a	10	a	10	a	900+	a	900+	a	0.8	c	0.8	c	0.72	a
CG	14.4	a	10	a	10	a	900+	a	900+	a	0.5	c	0.5	c	.80	a
CH	2.6	a	18	a	37	a	900+	a	540	e2	1.0	d2	1.5	d2	.23	b
CP	23.5	a	28	a	28	a	900+	a	900+	a	3.0	c	3.0	c	2.40+	a
CZ	43.6	a	101	a	101	a	900+	a	900+	a	9.0+	a	9.0+	a	2.50+	a
DA	59.6	a	86	a	86	a	900+	a	900+	a	9.0+	a	9.0+	a	2.50+	a
DB	5.4	a	23	a	23	a	900+	a	900+	a	1.5	b	1.5	b	1.00	a
DC	4.4	a	27	c	27	c	554	a	554	a	2.0	b	2.0	b	1.00	c
DD	38.8	a	26	a	26	a	900+	a	900+	a	1.0	b	1.0	b	2.50+	a
DE	43.2	a	40	a	40	a	900+	a	900+	a	2.0	b & c	2.0	b & c	1.92+	a
DF	4.9	a	20	c	20	c	900+	a	900+	a	1.5	b	1.5	b	0.65	a
DP	15.9	a	9	a	9	a	900+	a	900+	a	0.5	b	0.5	b	1.29	a

[a] Sandbag struck over a stud.
[b] Sandbag struck between studs.
[c] Without braces.
[d] With braces.
[e] Cavity wall, loaded on facing and backing.
[f] Cavity wall, load centered on backing only.

TABLE 21.—*Allowable loads on load-bearing partitions*

[Loads which may be applied to partitions over and above the weight of partition]

Construction symbol	Compressive load; height, 8 ft		Transverse load; span, 7 ft 6 in.				Concentrated load; diameter of disk, 1 in.				Impact load; span, 7 ft 6 in.; weight of sandbag, 60 lb				Racking load	
			Inside face		Outside face		Inside face		Outside face		Inside face		Outside face			
	Load	Criterion	Load	Criterion	Load	Criterion	Load	Criterion	Load	Criterion	Drop	Criterion	Drop	Criterion	Load	Criterion
WOOD																
	Kips/ft		*lb/ft²*		*lb/ft²*		*lb*		*lb*		*ft*		*ft*		*Kips/ft*	
CJ	1.50	c	84	d1	84	d1	158	a	158	a	1.5	b	1.5	b	0.48	a
CO	3.30	a	120	d1	120	d1	219	a	219	a	[a]1.0	b	[a]1.0	b	.47	a
											[b]1.5	b	[b]1.5	b		
QD	0.96	b	48	b	48	b	162	a	162	a	2.5	b	2.5	b	.38	b

[a] Sandbag struck over a stud.
[b] Sandbag struck between studs.

TABLE 22.—*Allowable loads on nonload-bearing partitions*

[Loads which may be applied to partitions over and above the weight of partition]

Construction symbol	Concentrated load; diameter of disk, 1 in.		Impact load; span, 7 ft 6 in.; weight of sandbag, 60 lb	
	Load	Criterion	Drop	Criterion
WOOD				
	lb		*ft*	
AM	130	a	1.0	b
BM	150	a	1.0	b
BN	134	a	1.2	e1
BR	171	a	0.5	b
BZ	166	a	1.0	b
CA	90	a	1.1	e1
CC	900+	a	0.5	e1
CD	330	e2	.2	e1
CK	165	a	1.1	e1
STEEL				
AI	98	a	0.8	e1
AQ	127	a	1.0	b
BA	280	a	1.0	b
CV	300	e2	2.5	b
DI	87	a	1.5	e1

TABLE 23.—*Allowable loads on floors*

[Loads which may be applied to floors over and above the weight of floor]

Construction symbol	Transverse load; span, 12 ft		Concentrated load; diameter of disk, 1 in.		Impact load; span, 12 ft; weight of sandbag, 60 lb	
	Load	Criterion	Load	Criterion	Drop	Criterion
WOOD						
	lb/ft²		*lb*		*ft*	
BS	35	b	900+	a	1.5	b
CE	124	c	900+	a	9.0+	a
CL	192	a	900+	a	9.0+	a
DM	97	a	900+	a	2.5	b
DS	100	b	900+	a	9.0+	a
QB	29	b	900+	a	4.0	b
STEEL						
AG	379	b	900+	a	9.0+	a
AJ	195	a	296	a	7.0	b
AN	141	a	900+	a	9.0+	a
AY	90	b	900+	a	4.0	b
BB	108	a	900+	a	9.0+	a
BC	80	b	900+	a	1.0	b
CR	282	a	900+	a	9.0+	a
CW	210	a	900+	a	9.0+	a
REINFORCED CONCRETE						
AS	60	b	900+	a	5.5	b
AW	326	a	900+	a	9.0+	a
CT	244	a	900+	a	9.0+	a
CY	246	c	900+	a	9.0+	a

TABLE 24.—*Allowable loads on roofs*

[Loads which may be applied to roofs over and above the weight of roof]

Construction symbol	Transverse load; span, 14 ft		Concentrated load; diameter of disk, 1 in.	
	Load	Criterion	Load	Criterion
WOOD				
	lb/ft²		*lb*	
BT	23	b	900+	a
DN	24	b	753	a
DO	67	b	750	b
DT	123	b	900+	a
DU	200	b	900+	a
QC	48	a	900+	a
STEEL				
AK	134	a	414	a
AO	42	a	382	a
CS	112	a	660	e2
CX	135	a & b	900	b
DJ	25	a	900+	a

5. DISTRIBUTION OF CRITERIA

The distribution of the criteria are given in table 25.

TABLE 25.—*Distribution of criteria for allowable loads*

Allowable load	Criterion				
	a	b	c	d	e
WALLS					
	Percent	*Percent*	*Percent*	*Percent*	*Percent*
Compressive:					
Wood	58	32	10		
Steel	75	12	12		
Reinforced brick and concrete	67	33			
Masonry	100				
Transverse:					
Wood; inside face		42		d1, 58	
Wood; outside face		39	61		
Steel; inside face	56	11		d1, 33	
Steel; outside face	44	11		d1, 44	
Reinforced brick and concrete; inside face	67		33		
Reinforced brick and concrete; outside face		67	33		
Masonry; inside face	92		8		
Masonry; outside face	92		8		
Concentrated:					
Wood; inside face	74	5			e2, 21
Wood; outside face	53	26			e2, 21
Steel; inside face	56				e2, 44
Steel; outside face	11	11			e2, 78
Reinforced brick and concrete; inside face	100				
Reinforced brick and concrete; outside face	100				
Masonry; inside face	100				
Masonry; outside face	96				e2, 4

TABLE 25—*Distribution of criteria for allowable loads*—Cont.

Allowable load	Criterion a	b	c	d	e
WALLS—Continued					
Impact:					
Wood; inside face	—	53	—	—	e1, 47
Wood; outside face	—	58	—	—	e1, 42
Steel; inside face	—	56	—	d2, 22	e1, 22
Steel; outside face	—	33	11	d2, 11	e1, 44
Reinforced brick and concrete; inside face	—	33	67	—	—
Reinforced brick and concrete; outside face	—	67	33	—	—
Masonry; inside face	8	38	50	d2, 4	—
Masonry; outside face	8	38	50	d2, 4	—
Racking:					
Wood	30	15	55	—	—
Steel	33	22	33	—	e2, 11
Reinforced brick and concrete	50	50	—	—	—
Masonry	88	8	4	—	—
LOAD-BEARING PARTITIONS					
Compressive:					
Wood	33	33	33	—	—
Transverse:					
Wood	—	33	—	d1, 67	—
Concentrated:					
Wood	100	—	—	—	—
Impact:					
Wood	—	100	—	—	—
Racking:					
Wood	67	33	—	—	—
NONLOAD-BEARING PARTITIONS					
Concentrated:					
Wood	89	—	—	—	e2, 11
Steel	80	—	—	—	e2, 20
Impact:					
Wood	—	44	—	—	e1, 56
Steel	—	60	—	—	e1, 40
FLOORS					
Transverse:					
Wood	33	50	17	—	—
Steel	62	38	—	—	—
Reinforced concrete	50	25	25	—	—
Concentrated:					
Wood	100	—	—	—	—
Steel	100	—	—	—	—
Reinforced concrete	100	—	—	—	—
Impact:					
Wood	50	33	17	—	—
Steel	62	38	—	—	—
Reinforced concrete	75	25	—	—	—
ROOFS					
Transverse:					
Wood	17	83	—	—	—
Steel	90	10	—	—	—
Concentrated:					
Wood	83	17	—	—	—
Steel	60	20	—	—	e2, 20
ALL CONSTRUCTIONS					
All loads:					
Wood	33	35	7	d1, 12	e1, 10 e2, 4
Steel	47	21	4	d1, 6 d2, 3	e1, 7 e2, 12
Reinforced brick and concrete	57	26	17	—	—
Masonry	73	10	15	d2, 1	e2, 1
Compressive:					
All materials	78	15	7	—	—
Transverse:					
All materials	49	23	5	d1, 23	—
Concentrated:					
All materials	80	6	—	—	e2, 14
Impact:					
All materials	10	48	19	d2, 3	e1, 20
Racking:					
All materials	58	15	25	—	e2, 2
All loads:					
All materials	51	23	10	d1, 6 d2, 1	e1, 5 e2, 4

Evidently many more allowable loads are fixed by some criteria than by others. For many constructions, particularly masonry, if there were no criterion **a** (whatever the percentage selected), the least maximum load would have to be taken as the allowable load because no other criterion fixes a smaller value. For all kinds of loading, this criterion sets the allowable load for three-fourths of the masonry constructions and for fewer of the others, down to a third of the wood constructions.

If it is conceded that the allowable load should not exceed the load causing damage, criterion **b** is necessary because it determines the allowable loads for a third of the wood constructions and for a smaller proportion of the others, down to a tenth of the masonry constructions.

Some kind of allowable load (or working stress), depending on the slope of the load-deformation curve, is generally accepted in engineering practice. For the house constructions tested, criterion **c**, (slope of curve) fixes less than a fifth of the allowable loads for the reinforced and masonry constructions, less than one-tenth for the wood and for a few of the steel constructions.

The strength deformation criteria **d1** (deflection) and **d2** (set) apply only to the lateral deflection of walls and load-bearing partitions as a safeguard against collapse under combined compressive and transverse loading. This criterion is unnecessary if the results obtained under combined loading are available. However for the present, criterion **d1** does serve to limit the allowable transverse load on many wood and steel walls also on load-bearing partitions. Criterion **d2** only limits the transverse loads and impact loads on a few steel walls and also a very few masonry walls.

The utility deformation criteria **e1** (deflection) and **e2** (set) also **e3** (shortening) and **e4** (set) are unnecessary to insure the structural safety of the house; they affect only the utility of the house to the occupants. Many engineers are convinced that the values for these criteria are too great. They can be decreased to any desired value, provided they are the controlling criteria, by increasing the cost of the house.

Of the loadings to which these criteria apply, the concentrated load is probably the least important, but criterion **e2** (identation 0.1 in.) fixes the allowable load for about half the steel walls, a quarter the wood walls, a few nonload-bearing partitions and steel roofs, and none of the load-bearing partitions and floors.

The reader is reminded that the allowable concentrated load is a matter of opinion and in any case depends upon the materials in the face of the construction and not at all upon the materials in the principal structural members.

Criteria **e3** and **e4** apply only to the shortening of walls and load-bearing partitions but no allowable compressive loads are fixed by these criteria because other criteria gave smaller loads.

If the setting of smaller values for criteria **e1** and **e2** is contemplated, consideration should be given to the fact that, although these criteria do not fix the allowable compressive or transverse loads for walls or load-bearing partitions and do determine the racking load on only a few steel walls, **e1** controls the allowable impact load for half the wood and steel walls and also for nonload-bearing partitions. For the constructions in this group, the lowest allowable height-of-drop (60-lb sandbag) for the wood walls, both inside and outside, is 0.8 ft and most of them are 2.5 ft or less. For the steel walls, the lowest value is 1.8 ft for loading on the inside, all the other values being above 3.5 ft. For loading on the outside, the lowest value is 1.7 ft. All the nonload-bearing partitions are much less stiff than the walls, the lowest value being 0.2 ft and the highest 1.2 ft for wood constructions. For steel constructions, the lowest value is 0.8 ft and the highest 1.5 ft.

Insofar as criterion **e1** is concerned, the allowable height-of-drop for the impact load depends upon two factors, the impact which is likely to be applied to the walls and the deflection which does not damage doors or windows. Records show that trucks and street cars have crashed into houses or buildings, but it is uneconomical to design houses to withstand such impacts. However, the impacts of smaller objects such as bricks and baseballs are likely to occur and should be considered in the design. Impacts likely to occur on the walls inside the rooms depend greatly upon the habits of the occupants.

6. Cracks in Plaster

If cracks in a plaster face are not considered damage, then the next greater critical load may be taken as the allowable load. Apparently the allowable load is determined more often by cracks in the plaster under transverse loading than under the other loadings. For walls, if the transverse load is applied to the outside face (plaster in tension) the cracks occur at somewhat smaller loads than if the load is applied to the inside face.

The behavior under transverse load of all constructions with plaster faces was studied. Each construction is listed in table 26 with the critical loads in order.

Critical loads are taken as the load determined by each of the criteria (if obtainable) and the least maximum load. There may be several kinds of damage, first in one portion of the construction and then in another portion at a greater load. The load for each kind of damage mentioned in the BMS reports is considered a critical load.

If the allowable load given previously for any constructions that have plaster faces is equal to or greater than the design load for the house, then to disregard the cracks in the plaster, and take the next greater critical load as the allowable load serves no useful purpose. Many walls that have plaster fulfill this requirement.

When wall *AZ*, which has a sheet-steel frame, is loaded on the inside face, the allowable load is not limited by cracks in the plaster, but, when loaded on the outside face, cracks do fix the load. If the cracks are disregarded, a slightly greater allowable load may be taken.

All the other walls have wood frames. If the cracks in wall *BJ*, which has stucco on metal lath in the outside face, are disregarded, the allowable load would be about twice as great as it is.

For wall *BK*, which has brick veneer outside, the allowable load is limited by cracks in the brick work for both inside and outside loading. Disregarding cracks in the veneer, the allowable loads could be about twice as great as they are because the loads causing cracks in the plaster are much greater than those causing cracks in the brick veneer.

Except, for wall *QA*, which has wood lath, all the wood walls have fiberboard lath. When loaded on the outside face, all the plaster on fiberboard lath cracked at much greater loads than did the plaster on the one wall having wood lath.

For the one load-bearing partition *QD*, provided cracks are disregarded, the allowable load could be twice as great, but there are no usually accepted design values for transverse load on partitions.

For all the floors, the allowable loads are limited by cracks in the plaster ceiling. The allowable load for floors *BS* and *QB* which have wood frames is less than the usually accepted design load for floors in houses (40 lb/ft^2) and at least twice as great for floors *AY* and *BC* with steel frames. Depending upon the kind of lath, the allowable loads increase in the order, wood lath (*QB*), fiberboard lath (*BS*), gypsum-board lath (*BC*), and expanded metal lath (*AY*).

For the floors with plaster ceiling, the least allowable load is for conventional wood floor *QB*. If cracks in the plaster are disregarded, the allowable load jumps from 29 to 145 lb/ft^2, a value greater than the design load for floors in houses.

The one roof *BT* has a wood frame and fiberboard lath. If cracks are disregarded, the allowable load jumps from 23 to 142 lb/ft^2.

Table 26.—*Critical transverse loads on constructions with plaster faces*

[lm =least maximum load]

Construction symbol	Inside face Load	Inside face Criterion	Outside face Load	Outside face Criterion
	WALLS			
	lb/ft²		*lb/ft²*	
AZ	126	a	110	b[1]
	136	d1	131	d1
	162	c	132	a
	168	lm	162	c
			176	lm
BG	75	b[1]	75	b[1]
	135	d1	100	d1
	180	e1	160	e1
	274	a	194	a
	305	lm	215	lm
	330	d2	320	d2
	385	c	370	e2
BJ	97	b[1]	70	b[1]
	145	d1	150	d1
	195	e1	210	e1
	205	d2	255	c, d2
	310	e2	258	a
	315	a	275	e2
	350	lm	287	lm
BK	50	b[2]	64	b[2]
	108	d1	100	b[1]
	150	b[1]	125	d1
	157	e1	170	e1
	215	d2	200	c, d2
	248	a	225	e2
	255	e2	265	a
	275	c	294	lm
	276	lm		
BL	75	b[1]	60	b[1]
	130	d1	120	d1
	195	e1	160	e1
	314	a	230	d2
	325	d2	232	a
	350	lm	258	lm
			260	e2
			275	c
BQ	90	b[1]	50	b[1]
	100	d1	92	d1
	135	b[3], e1	132	e1
	184	a	150	a
	205	lm	155	c
	220	d2	157	b[3]
	232	c	167	lm
	240	e2		
BX	90	b[1]	50	b[1]
	120	d1	110	d1
	175	e1	160	e1
	232	a	237	a
	250	c	263	lm
	258	lm	283	b[4]
	272	b[5]	350	d2
	325	d2		
QA	96	d1	26	b[1]
	150	e1	80	d1
	236	a	116	e1
	262	lm	194	a
			215	lm
	LOAD-BEARING PARTITIONS			
QD	48	b[1]		
	106	d1		
	216	a		
	240	lm		
	FLOORS			
AY	90	b[1]		
	250	a		
	280	c		
	320	b[6]		
	334	lm		
BC	80	b[1]		
	105	a		
	140	lm		
	151	c		
BS	35	b[1]		
	247	a		
	350	b[7]		
	375	e1		
	412	lm		
QB	29	b[1]		
	145	a		
	242	lm		

Table 26.—*Critical transverse loads on constructions with plaster faces—Continued*

Construction symbol	Inside face Load	Inside face Criterion	Outside face Load	Outside face Criterion
	ROOF			
BT	23	b[1]		
	142	a		
	204	c		
	210	b[8]		
	218	lm		

[1] Plaster cracked.
[2] Brick veneer cracked.
[3] One outer stud ruptured.
[4] One outer stud partially ruptured.
[5] One outer stud cracked.
[6] Rupture of weld holding the flange of a joist to the end member.
[7] One outer joist cracked.
[8] One outer rafter ruptured.

7. Graphs of Allowable Loads

The allowable loads for walls arranged in order, beginning with the greatest, are shown in figures 43 (compressive, transverse, and racking loads) and 44 (concentrated and impact loads). Those for partitions, floors, and roofs are shown in figure 45.

In designing a house, low weight is a desirable characteristic of the construction; therefore, the values in figure 42 (weight) are arranged in order, beginning with the lowest. On the other hand, an allowable load equal to the design load is necessary; therefore, in the graphs showing allowable load, the constructions are arranged in decreasing order beginning with the greatest. In selecting the construction for a particular house for which the design loads are known, start at the left of the graph and move to the right until a construction is found having the requisite allowable load. If it is unsuitable because of weight, price, or other characteristics, a construction with somewhat greater allowable load may be preferable.

8. Relation of Materials and Design to Allowable Load

(*a*) *Walls*

The allowable loads are shown on figures 43 and 44.

(*1*) *Wood*

Compressive load.—It is evident from figure 43 that, of the wood walls under compressive loading, wall *DR* is at the top and has the greatest allowable load. The other walls with unconventional frames are *CB* (planks) which is above the middle and *DL* (demountable) which is near the bottom of the group.

Two of the walls have light conventional frames, *DQ* which is above the middle and *DK* which is at the bottom of the group.

All the other walls have conventional frames and are scattered through the group from next

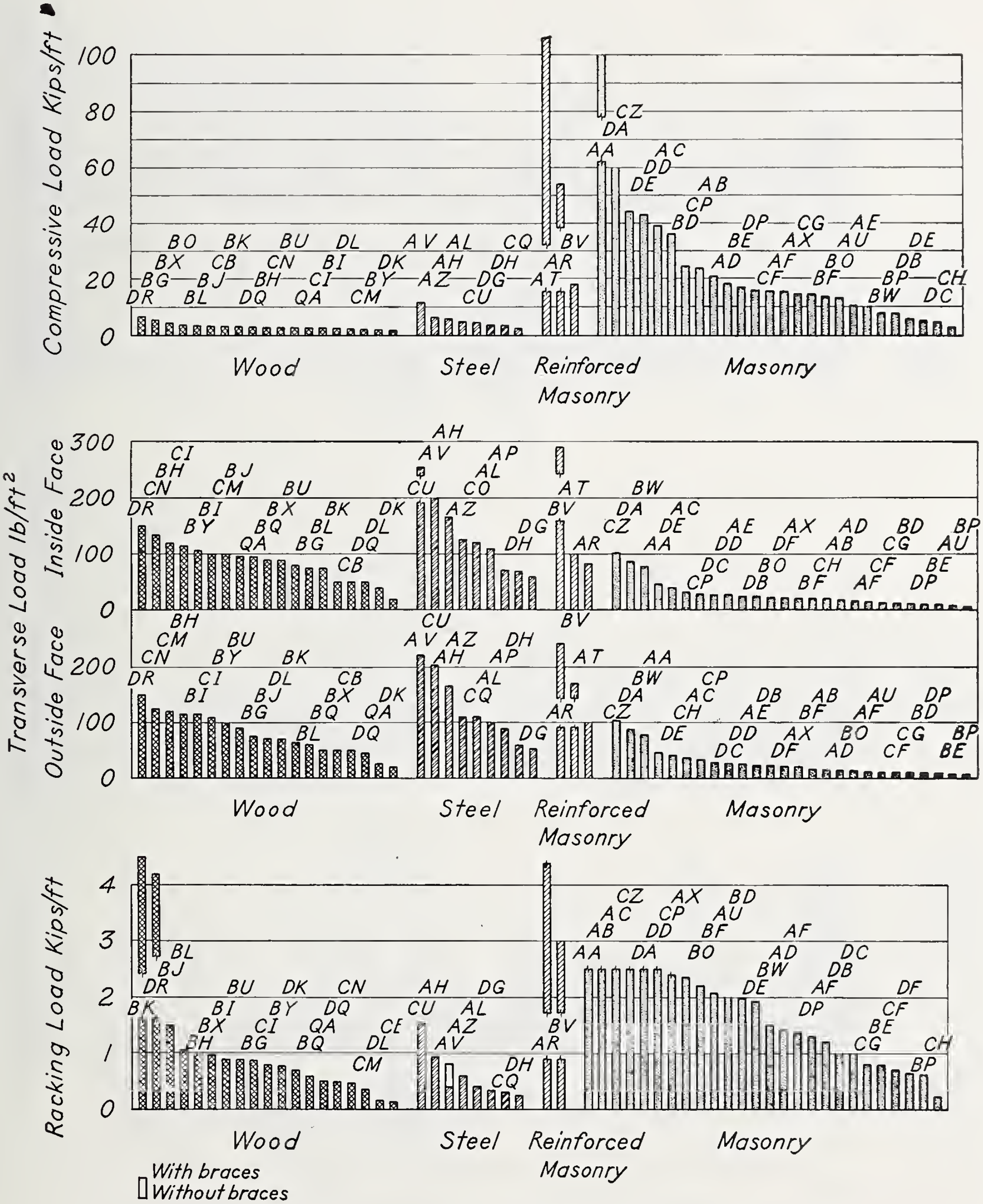

FIGURE 43.—*Allowable compressive, transverse, and racking loads for walls.*

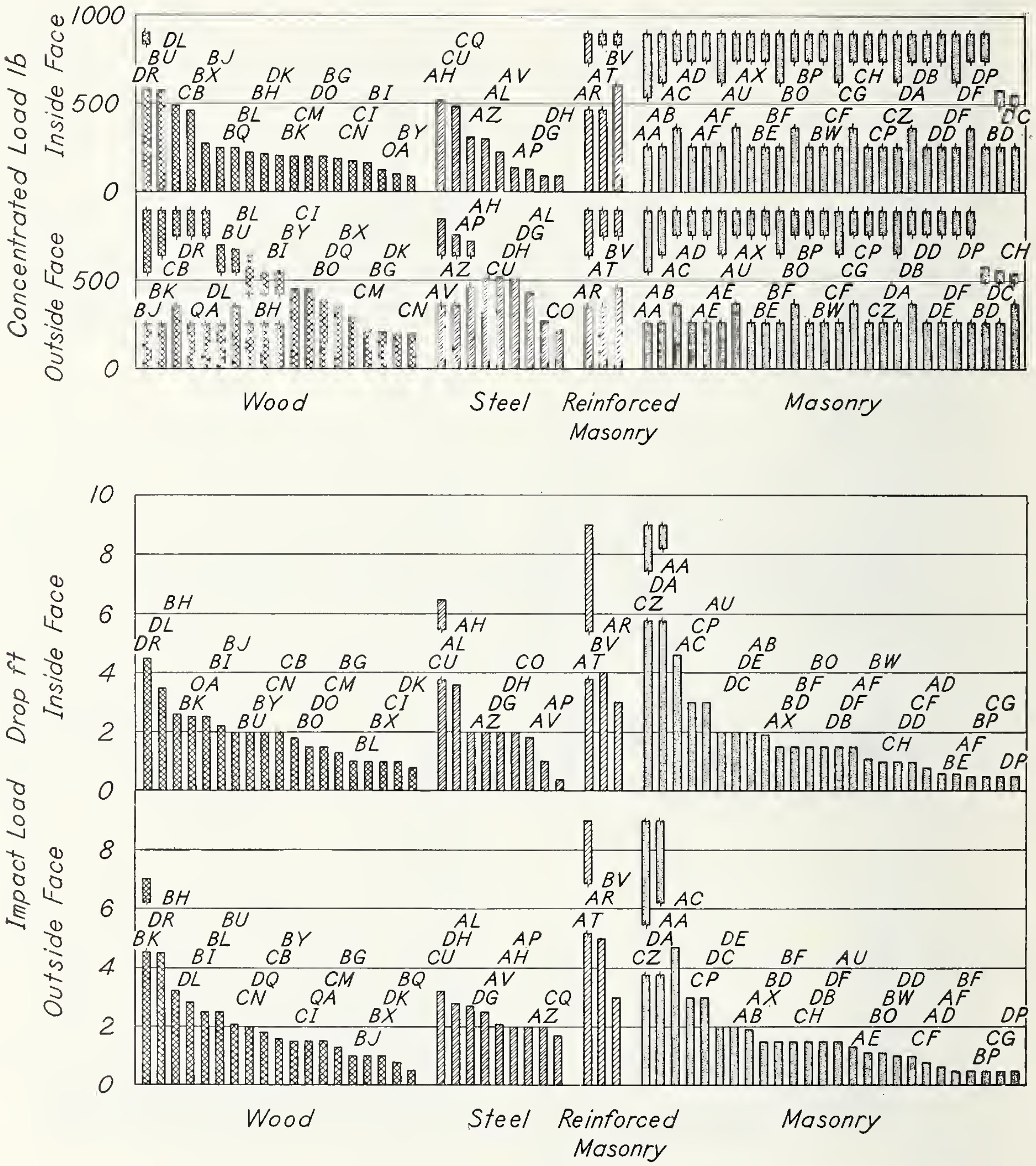

FIGURE 44.—*Allowable concentrated and impact loads for walls.*

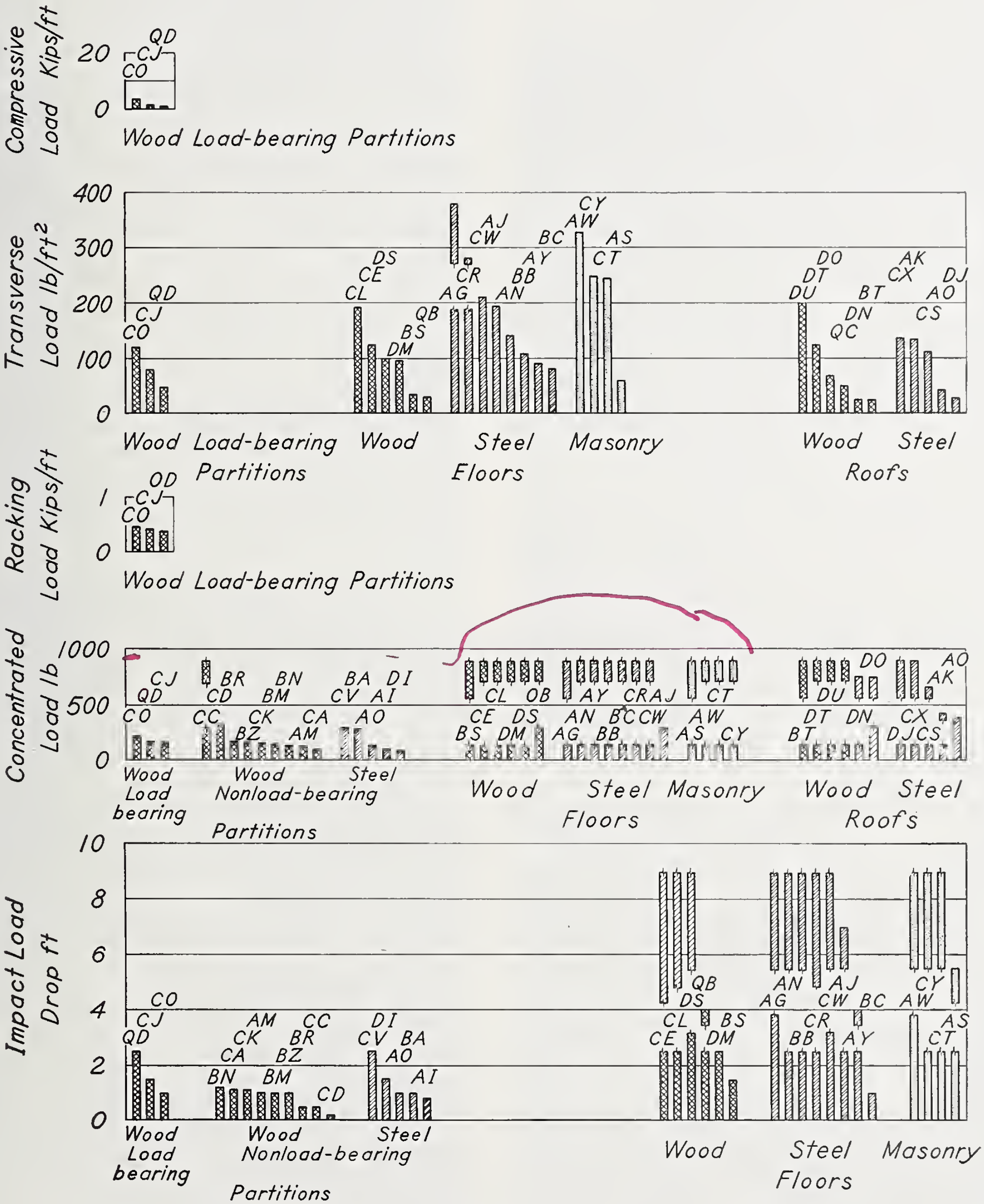

FIGURE 45.—*Allowable loads for partitions, floors, and roofs.*

to the top to next to the bottom. The studs are 2 by 4 in., spaced 1 ft 4 in., of Douglas fir or Norway pine with no indication that the kind of wood affects the allowable load.

Depending upon materials on the inside face, plywood is scattered through the group, but all walls with fiberboard lath and plaster are well above the middle, and are above those with fiberboard only. The walls with gypsum board, *CI*, and wood lath with plaster, *QA* (conventional), are below the middle and are lower than the three walls with fiberboard only.

Depending upon the materials in the outside face, walls of plywood and wood sheathing laid diagonally are scattered through the upper three-quarters of the group; fiberboard with brick veneer, *BK*, is above the middle; and fiberboard with stucco, *BJ*, is still higher. Fiberboard only or fiberboard overlaid with plywood, siding, or shingles is scattered from next to the top to the bottom of the group.

Transverse load, inside.—As for compressive loading, wall *DR* is at the top and has the greatest transverse allowable load.

The four walls not having 2- by 4-in. studs are grouped together at the bottom: *CB* (plank), *DQ* (2- by 3-in. studs), *DL* (demountable), and *DK* (2- by 2-in. studs). Conventional frame walls with 2- by 4-in. studs are intermediate with no indication that the kind of wood in the studs affects the allowable load.

Depending upon the materials on the inside face, walls of plywood are scattered through the group, gypsum-board walls are near the top, but walls having fiberboard lath with plaster are slightly below the middle, and walls of wood lath with plaster are next to the top of the plaster group.

Most of the fiberboard only are above the middle, but the two walls on light conventional frames are near the bottom of the group.

Depending upon the materials in the outside face, walls of plywood are scattered through the group, wood sheathing laid diagonally is at the middle, and fiberboard with stucco is higher. Fiberboard with brick veneer is well below the middle. Fiberboard only, *CM*, is above the middle, and *DK* is at the bottom. Walls of fiberboard overlaid with plywood, siding, or shingles are scattered through the group.

Transverse load, outside.—Again, wall *DR*, which has the greatest allowable load, is at the top. Of the four walls not having 2- by 4-in. studs, *DL* (demountable) is just below the middle, and the others, *CB*, *DQ*, and *DK* are near the bottom. Wall *QA* (conventional) is next to the bottom. The other walls are intermediate.

Depending upon the materials on the inside face, walls of plywood are scattered through the group, gypsum board is above the middle, and fiberboard lath with plaster is lower. Wood lath with plaster is next to the bottom. Most of the fiberboard only are above the middle but two walls on light conventional frames are near the bottom of the group.

Depending upon the materials on the outside face, walls of plywood are above the middle and wood sheathing laid diagonally is near the bottom. Fiberboard with brick veneer is higher, and fiberboard with stucco is still higher at the middle of the group. Fiberboard only is near the top, except the one wall on a light frame, which is at the bottom. Overlaid fiberboard walls are scattered through the group.

Racking load.—Except wall *DL* (demountable), the wood frame walls have no braces and the racking load depends more on the materials in the faces and the fastenings than on the frame.

Wall *BK*, having brick veneer is at the top; *DR* having plywood faces is next. Of the other four walls not having 2- by 4-in. studs, *DK* (2- by 2-in. studs) is just below the middle; *DQ* (2- by 3-in. studs) in considerably lower; *DL* (demountable, with braces) is next to the bottom; and *CB* (plank) is at the bottom of the group.

Depending upon the materials on the inside face, walls of fiberboard lath with plaster are scattered through the group, mostly above the middle, followed closely by walls of wood lath with plaster. Gypsum-board walls are at the middle of the group, and walls of fiberboard only are scattered from near the top to near the bottom. Plywood is scattered from next to the top to the bottom.

Depending upon the materials on the outside face, fiberboard with brick veneer is at the top and fiberboard with stucco is very near the top. Plywood is above the middle, but wood sheathing laid diagonally is considerably below the middle. Fiberboard only and fiberboard overlaid with siding or shingles is scattered from near the top to next to the bottom of the group.

Concentrated load.—The concentrated load is applied to the face at what is judged to be the weakest place—midway between studs. The allowable load depends to a great extent upon the materials in the face. The test was discontinued when the applied load was 1,000 lb.

Inside face.—As shown in figure 44, the four walls having plywood inside faces are at the top of the group. They are followed immediately by four walls having fiberboard lath with plaster and two others below the middle. Gypsum board is near the bottom and wood lath with plaster is next to the bottom. Fiberboard only ranges from the middle to the bottom of the group.

Outside face.—The concentrated load test was discontinued for brick veneer, plank, plywood, stucco, and one wall of wood sheathing with siding. All have an allowable load of 900+ lb. Plywood overlaid with shingles is above and below the middle of the group and one wall of wood sheathing with siding is below the middle. Fiberboard only and fiberboard overlaid with siding are scattered from above the middle to the bottom.

Impact load, inside.—Wall *DR* is at the top, having the greatest allowable load. Wall *DL* (demountable) is next. Of the other walls not having 2- by 4-in. studs, *CB* (plank) is just below the middle and *DQ* (2- by 3-in. studs) is still lower. Wall *DK* (2- by 2-in. studs) is at the bottom of the group. All the walls having 2- by 4-in. studs are scattered through the group.

Depending upon the materials on the inside face, plywood is scattered above the middle, fiberboard lath with plaster is scattered from near the top to near the bottom. Wood lath with plaster is next to the top of the plaster group and gypsum board is next to the bottom. Walls of fiberboard only are scattered through the entire group from near the top to the bottom.

Depending upon the materials in the outside face, plywood ranges from the top to next to the bottom. Fiberboard with brick veneer and fiberboard with stucco are well above the middle, while one wall of wood sheathing with siding is above the middle and one is below. Fiberboard only and fiberboard overlaid with siding or shingles are scattered from near the top to the bottom of the group.

Impact load, outside.—The brick veneer wall *BK* is at the top and *DR* is next. Of the other walls not having 2- by 4-in. studs, *DL* (demountable) is near the top, *DQ* (2- by 3-in. studs) and *CB* (plank) are at the middle, and *DK* (2- by 2-in. studs) is next to the bottom. The walls having 2- by 4-in. studs are scattered through the group.

Depending upon the materials on the inside face, one fiberboard lath with plaster is at the top, another is above the middle, and the rest are at or near the bottom. Gypsum board is below the middle followed by wood lath with plaster. Plywood walls are scattered above the middle and walls of fiberboard only range from near the top to next to the bottom.

Depending upon the materials in the outside face, brick veneer is at the top and stucco is near the bottom. One wall of wood sheathing with siding is below the middle and one is at the bottom.

Plywood walls are scattered, mostly above the middle, and plank is at the middle. Fiberboard only and fiberboard overlaid with siding or shingles are scattered from near the top to next to the bottom.

(2) *Steel*

Compressive load. — Wall *AV* is at the top and has the greatest allowable load in compression. Except that, for this wall, the yield strength and tensile strength of the sheet steel are appreciably greater than those for the other steel walls, there is no indication that the properties of the steel affect the allowable load. The width of the studs (perpendicular to the wall) ranges from 2 3/8 to 4 in., the spacing from 9 13/16 in. to 4 ft 0 in.; and the thickness of the sheet steel from No. 20 United States Standard Gage (0.0368 in.) to No. 11 BWG (0.120 in.), but apparently there is no relation between these variables and the allowable load. For some walls, a face and studs were formed from the sheet and for these the allowable loads are near the middle and at the bottom of the group. The studs in *DH* were No. 11 BWG (0.120 in.) but the allowable load is less than for *DG*. These two walls were identical except that the studs in wall *DG* were No. 14 BWG (0.083 in.); therefore, the greater strength for *DH* was not realized.

Depending upon the materials on the inside face, fiberboard only is at the top of the group. It is followed by gypsum-board lath and plaster; sheet steel and fiberboard then alternate.

Depending upon the materials in the outside face, fiberboard overlaid with sheet steel is at the top followed by fiberboard overlaid with plywood (1/2 in.) and shingles. Fiberboard overlaid with siding is at the middle of the group. All the others are sheet steel.

Transverse load, inside. —Wall *CU* is at the top, and the width of the studs is the greatest except one.

There is an indication that as the width of the studs decreases and the spacing increases the allowable load decreases.

Depending upon the materials on the inside face, sheet steel and fiberboard are scattered through the group; but, steel is higher; gypsum-board lath with plaster is about the middle; and gypsum board is lower.

Depending upon the materials in the outside face, fiberboard overlaid with siding is at the top. Plywood and fiberboard, either alone or overlaid with siding or shingles, are scattered through the group. Sheet steel only is near the middle and sheet steel box siding (0.0245 in.) is at the bottom.

Transverse load, outside.—Except that wall *AV* is at the top and *CU* is next, the order is the same for outside loading as for inside loading.

Racking load.—The braces in wall *AV* increase the allowable load. There is little relation between the design of the wall and the allowable load.

Concentrated load, inside.—Depending upon the material on the inside face, the walls above the middle of the group are sheet steel and gypsum-board lath with plaster, and those below are fiberboard only and gypsum board.

Concentrated load, outside.—Depending upon the material in the outside face, sheet steel is scattered through the group from top to bottom. Plywood only and plywood overlaid with shingles is above the middle, and fiberboard overlaid with siding is at the middle.

Impact load, inside.—Comparing the order of the walls for transverse load inside, three walls above the middle are in the same relative position for impact load inside. Of the others, two are considerably higher and three are considerably lower. Two of the walls having the steel sheet formed into a face with studs are at the top of the group.

Depending upon the material on the inside face, steel followed by gypsum-board lath with plaster are above the middle and fiberboard is below the middle. Gypsum board is at the bottom.

Depending upon the material in the outside face, there is no definite trend; but, apparently, the order is fiberboard, steel, and plywood.

Impact load, outside.—Comparing the order of the walls for transverse load applied to the outside face, most of the walls above the middle for transverse load are below the middle for impact load outside and in the same order; walls below the middle of the transverse group are above the middle of the impact group. Of the walls having a one-piece face with studs, one is at the top and one is at the bottom.

Depending upon the material on the inside face, steel is scattered from the top to the bottom of the group, fiberboard is above the middle and near the bottom, gypsum board followed by gypsum-board lath with plaster, are next to the bottom.

Depending upon the material in the outside face, steel is scattered through the group, the steel box siding (0.0245 in.) is above the middle, as is also fiberboard. Plywood is below the middle.

(3) *Reinforced Brick and Concrete*

Compressive load.—Wall *AT*, brick reinforced longitudinally and transversely, is at the top, having the greatest allowable load in compression. Next is a hollow concrete slab and at the bottom is a concrete slab having flanges.

Transverse load, inside.—The slab having flanges is at the top, the reinforced brick wall is next, and at the bottom is the hollow slab.

Transverse load, outside.—The hollow slab now is at the top, followed by the slab having flanges, and the reinforced brick wall which is at the bottom.

Racking load.—No racking load was applied to the reinforced brick wall. The hollow slab is at the top followed by the flanged slab.

Concentrated load.—Both inside and outside faces are either brick or concrete and for each of the walls the allowable load is 900+ lb.

Impact load, inside.—The reinforced brick wall is at the top. This is followed by the slab having flanges and then the slab enclosing air spaces.

Impact load, outside.—The reinforced brick wall again is at the top, followed by the flanged slab, and the slab enclosing air spaces.

As determined by the order in the group, the reinforced brick wall has a greater allowable load under impact than under transverse loading but both slabs have smaller allowable loads.

(4) *Masonry*

Compressive load.—Wall *AA*, high-strength brick, is at the top and has the greatest allowable load in compression. It is followed by plain concrete, terracrete (earth), and medium-strength brick, all near the top. Cavity walls of brick are high in the group and those of concrete blocks are near the middle. Tile on end is above the middle and tile on side is near the bottom. Near the middle are concrete slabs laid to leave air spaces, concrete blocks only, and concrete blocks with brick facing.

Near the bottom are solid concrete blocks, laid to leave air spaces; and the earth walls: adobe, rammed earth, and bitudobe.

There is no relation between the thickness of the wall and the allowable load. There is no very definite relation between the allowable load and the compressive strength of the materials such as masonry units and mortar.

Transverse load, inside.—Walls *CZ* and *DA*, plain concrete, are at the top, and have the greatest allowable load for transverse loading inside.

Wall *BW*, solid concrete units connected to vertical splines, follows plain concrete but *CH* is below the middle of the group.

Walls of brick are scattered from near the top, *AA* (high-strength brick), to near the bottom, *DP*.

All earth walls are above the middle, in order, terracrete, bitudobe, adobe, and rammed earth.

Structural clay tile on side, *AE*, is above the middle and on end, *AD* is below.

All concrete block walls scatter from just above the middle, *BO,* to the bottom, *BP.* Of the solid concrete block walls, those having brick facing are the higher. Of the cavity walls, *AX,* concrete block, is at the middle; *BD,* brick, is near the bottom; and *AU,* tile on side, is next to the bottom. The walls *CF* and *CG,* solid concrete blocks laid to leave air spaces, are midway between the middle and the bottom.

Except that the earth walls are much thicker than the other walls and have great allowable loads, there is no indication that the thickness of the walls is related to the load.

Transverse load, outside.—With a few exceptions, the order for outside loading is the same as for inside loading and the discussion is the same, Wall *CH* is much higher in the group, perhaps because the flanges extend inward, also *AU* is higher loaded on the brick facing than on the tile, but *BO* is lower in the group when loaded on the brick facing than on the concrete block.

Racking load.—The seven walls at the top, *AA* to *CP,* did not fail under a racking load of 50 kips, therefore the test was discontinued. Most of the brick walls are scattered above the middle. The earth walls are scattered from well above the middle to near the bottom in the order: terracrete, *DD,* and *DE*; adobe, *DB*; bitudobe, *DC*; and rammed earth, *DF.*

One cavity wall, *AX* (concrete block), is well above the middle and the others, *AU* (tile on side, brick facing) and *BD* (brick) are together just above the middle.

The concrete block walls *BO* and *BF,* having a brick facing, are well above the middle of the group and those without brick facing are from below the middle to next to the bottom.

Of the solid concrete units connected to vertical splices, *BW* is at the middle and *CH* is at the bottom of the group.

Structural clay tile on end and on side are together just below the middle and the solid concrete block *CG* and *CF,* laid to enclose air spaces, are near the bottom.

Concentrated load, inside and outside.—Except for three walls, the concentrated loading was discontinued and the allowable load is 900+ lb. The exceptions are *BD,* brick cavity wall; *DC,* bitudobe (cavity in the earth); and CH, concrete units flanged inward (unit cracked when loaded outside between flanges).

The least allowable load for any of the masonry walls is 540 lb.

Impact load, inside.—The plain concrete walls *CZ* and *DA* are at the top having the greatest allowable load under impact. The test was discontinued at a drop of 10 ft, therefore, the actual strength was not determined. High-strength brick wall *AA* follows. The other brick walls are all well above the middle of the group, except *DP* which is at the bottom.

The cavity walls all are above the middle, *AU,* tile and brick, are higher than *AX,* concrete block, and *BD,* brick.

The earth walls are scattered through the group. Well above the middle are *DC,* bitudobe, and *DE,* terracrete block; at the middle are *DB,* adobe, and *DF,* rammed earth; and lower is *DD,* monolithic terracrete.

Concrete block walls with brick facing are at the middle and walls of block only are near the bottom.

Structural clay tile on side, *AE,* is just below the middle and on end, *AD* is lower.

Walls *BW* and *CH,* concrete units connected to vertical splines, are below the middle. Still lower is *CF,* concrete blocks laid to enclose air spaces, and next to the bottom is *CG.*

Impact load, outside.—Except for a few walls, the order for outside loading is the same as for inside loading. *CH* is much higher in the outside group, perhaps, because the flanges extend inward. *AU* is much lower under impact when loaded on the brick facing, although it was higher under transverse loading. As for transverse loading, *BO* is lower in the group when loaded on the outside brick facing than when loaded on the concrete block.

(*b*) *Load-bearing Partitions*

The allowable loads are shown in figure 45.

(*1*) *Wood*

Compressive load.—The three load-bearing partitions all have 2- by 4-in. studs and single plates of Douglas fir, No. 1, common. All studs are spaced 1 ft 4 in. Evidently the differences in the allowable loads are not due to differences in the frames.

Depending upon the materials in the faces, partition *CO,* fiberboard glued to the frame, is at the top, followed by gypsum board, then by wood lath with plaster.

Transverse and racking loads.—For transverse and racking loads, the order is the same as for compressive load with *CO* at the top.

Concentrated load.—Depending upon the material in the faces, fiberboard is at the top followed by wood lath with plaster, then by gypsum board.

Impact load.—Depending upon the material in the faces, wood lath with plaster now is at the top, followed by gypsum board, then by fiberboard.

(*c*) *Nonload-bearing Partitions*

The allowable loads are shown on figure 45.

(*1*) *Wood*

Concentrated load.—The allowable load under concentrated loading depends principally upon the material in the faces and also upon the spacing of the structural members (studs).

In general, the greater the spacing (span) the less the allowable load. Depending on the spacing, partition *CC* (5 3/8 in.) is at the top having the greatest allowable load for concentrated loading and *CD* (1 ft 0 in.) next. For all the other partitions the spacing is 1 ft 4 in.

Depending upon the material in the faces, plywood (1/4 in.) is at the top followed by fiberboard lath with plaster, also, gypsum board and, at the bottom, fiberboard only (1/2 in.).

Impact load.—Partition *BN* is at the top, having the greatest allowable load under impact.

Most of the partitions have 2- by 4-in. studs, spaced 1 ft 4 in., but, in *CK* the studs are 2 by 3 in. The allowable load under impact for this partition is too great, because there are four studs in each specimen, not three, to represent portions of the partitions in a completed house. Estimating the probable allowable load of representative specimens is impracticable.

The two partitions at the bottom are *CC*, plank, and *CD*, 2- by 2-in. studs, spaced 1 ft 0 in. There is no indication that the allowable load depends on the kind of wood.

Depending upon the material in the faces, fiberboard (1/2 in.), *CK* (extra stud), and gypsum board (1/2 in.) are above the middle of the group; fiberboard lath with plaster is below the middle and plywood (1/4 in.) is at the bottom.

(*2*) *Steel*

Concentrated load.—Partition *CV* is at the top, having the greatest allowable load. For all the partitions, the spacing of the structural members did not vary greatly.

Depending upon the material in the faces, sheet steel is at the top, followed by gypsum-board lath with plaster and gypsum board only. Fiberboard is at the bottom.

Impact load.—Partition *CV* is at the top, having the greatest allowable load under impact. In general, the allowable load decreases as the width (perpendicular to faces) of the studs decreases.

Depending upon the material in the faces, sheet steel is at the top, then fiberboard, gypsum board, and gypsum-board lath with plaster.

(*d*) *Floors*

The allowable loads on floors are shown in figure 45.

(*1*) *Wood*

Transverse load.—Floor *CL* is at the top and has the greatest allowable load applied transversely. This floor and the two at the bottom of the group have 2- by 8-in. joists, spaced 1 ft 4 in., of Douglas fir and long-leaf southern pine. The other three floors have unconventional frames and are at or above the middle of the group. There is no indication that the allowable load depends upon the kind of wood.

Depending upon the material in the lower face (ceiling), the two floors that have no ceilings are at the top of the group. They are followed by two floors with plywood (3/8 in.) ceilings; at the bottom are the two floors that have ceilings of lath with plaster. Of these, *BS*, fiberboard lath with plaster, is above *QB* (conventional), which has wood lath with plaster, and the allowable load is taken as the load causing cracks in the plaster.

Depending upon the material in the upper face, *CL*, plywood with oak finish floor, is at the top. Floors *DS* and *DM*, plywood only, are at the middle and the others are scattered.

Concentrated load.—For all the wood floors, the concentrated loading was discontinued and the allowable load is 900+ lb.

Impact load.—For the floors above the middle, the impact loading was discontinued and the allowable load is 9.0+ ft drop.

Below the middle are *QB* (conventional) which has a plaster ceiling and *DM* (demountable) which has a plywood ceiling. *BS* with a plaster ceiling is at the bottom.

Under transverse loading, the load causing cracks in the plaster is nearly the same for wood lath and fiberboard lath, but, under impact loading, the plaster on wood lath cracked at nearly three times the height-of-drop of that for plaster on fiberboard lath.

(*2*) *Steel*

Transverse load.—Floor *AG*, cellular sheet-steel panel with concrete fill, is at the top and has the greatest allowable load. The next four floors have a lower face (ceiling) of sheet steel, one-piece deck with the joists. The two floors *BB* and *BC* at and near the bottom, have channel-shaped sheet-steel frames. There is no relation between the depth of the joists and the allowable load. There is a slight indication that as the thickness of the sheet steel increases the allowable load decreases.

Depending upon the material in the lower face (ceiling), all the floors at and above the middle have sheet-steel ceilings and *BB*, below the middle, has no ceiling. The two floors, *AY* and *BC*, at the bottom of the group have plaster ceilings but there is little difference in the allowable loads for metal lath and gypsum-board lath.

Depending upon the material in the upper face, concrete with composition finish flooring is at the top. For all the other floors, the finish flooring is either hard or soft wood on either wood subflooring or wood sleepers with no indication that the materials in the upper face have any effect on the allowable load.

Concentrated load.—The concentrated load was applied to the upper face only. For all the floors except *AJ*, the loading was discontinued and the allowable load is 900+ lb. For *AJ* the load is 296 lb.

Impact load.—For all the floors above the middle, the impact loading was discontinued and the allowable load is 900+ lb. Floors *AJ* and *AY*, below the middle, have a one-piece face with joists; and *BC*, at the bottom, has channel-shaped joists.

Depending upon the material in the lower face (ceiling), all except the two floors at the bottom have either sheet-steel ceilings or no ceiling. The two at the bottom of the group have plaster ceilings and the allowable load is taken as the load causing cracks in the plaster. For plaster on expanded metal lath (*AY*), the height-of-drop is four times that for plaster on gypsum-board lath (BC).

Depending upon the material in the upper face, concrete with composition flooring is at the top, whereas for the other floors there is no indication that the materials affect the allowable load.

Comparing the order of the floors under transverse loading with the order under impact loading, *AG* (concrete fill) is at the top for both, and *AY* and *BC* (plaster ceilings) are at the bottom for both. Under transverse loading, the three floors *CR*, *CW*, and *AJ*, above the middle, are each two places lower under impact. They have one-piece face and joists. The two floors *AN* and *BB*, below the middle under transverse loading, are each three places higher under impact. Floor *AN* has a one-piece face and joists which interlock and *BB* has channel-shaped sheet-steel joists.

(*3*) *Reinforced Concrete*

Transverse load.—Floor *AW* is at the top and has the greatest allowable load. It has steel joists, structural clay tile fillers on the lower flanges, and a concrete fill. Floor *CT*, below the middle, is similar except that the fillers are asbestos-cement, not clay tile. The steel joists in *CT* are slightly higher in carbon content, and the yield point and tensile strength are appreciably greater than the steel joists in *AW*. However, the weight per foot of the joists in *CT* is somewhat less than that of the joists in *AW*. It appears probable that there is no appreciable difference in the strength of the joists. In addition, the compressive strength of the concrete was the same in both floors; therefore, the lower allowable load for floor *CT* can only be ascribed to the greater spacing of the joists, 2 ft 0 in., not 1 ft 5 ½ in. It is unlikely that the substitution of asbestos-cement fillers for the tile fillers has any appreciable effect on the allowable load.

Floor *CY*, next the top, has precast reinforced concrete joists and nearly the same allowable load as *CT*.

Floor *AS*, at the bottom, is a hollow reinforced concrete slab.

Concentrated load.—For all the floors, the concentrated loading was discontinued and the allowable load is 900+ lb.

Impact load.—For the three floors *AW*, *CY*, and *CT*, at the top, the impact loading was discontinued and the allowable load is 9.0+ ft drop. For impact loading, the order is the same as for transverse loading and the discussion is much the same except that *AS* has one-fourth the transverse allowable load of *CT* but more than half the height-of-drop.

(*e*) *Roofs*

The allowable loads are shown on figure 45.

(*1*) *Wood*

Transverse load.—Roof *DU* is at the top, and has the greatest allowable load applied transversely. For all the roofs except *QC* (conventional), the allowable load was determined by damaged.

Roof *DT*, which has the upper rafters edgewise, should have a greater allowable load than *DU*, with the upper rafters flatwise, but it has only about half the load.

Roof *DN* is similar to *DO* except that the rafters are 6 in. deep, not 8 in. The transverse strength of 6-in. rafters is about half that of 8-in. and the allowable loads have about the same ratio.

Roof *QC* (conventional), at the middle of the group, has 2- by 6-in. rafters, spaced 2 ft 0 in., but *BT*, at the bottom, has 2- by 8-in. rafters, spaced 1 ft 4 in. Considering the rafters only, the transverse strength of roof *BT* should be two and a half times the strength of roof *QC* because longleaf pine has a slightly greater modulus of rupture than Douglas fir. However, the allowable load is less than half because the plaster ceiling cracked.

Depending upon the material in the lower face, plywood is at the top and fiberboard lath with plaster is at the bottom. The other roofs have no ceiling.

Depending upon the material in the upper face, 5/8-in. plywood is at the top and 3/8-in. plywood is lower. Wood sheathing overlaid with shingles or built-up roofing are below the middle of the group.

Concentrated load.—For the roofs *DT* and *DU* which have 5/8-in. plywood in the upper face, *QC*, which has wood sheathing with shingles, or *BT*, which has built-up roofing, the concentrated loading was discontinued and the allowable load is 900+ lb.

For roofs *DN* and *DO*, the upper faces are alike and both are 3/8-in. plywood. The allowable loads are nearly the same.

(2) Steel

Transverse load.—Roof *CX* is at the top, and has the greatest allowable load applied transversely. All the roofs, except *DJ*, at the bottom, have a one-piece face with rafters. *DJ* has channel-shaped rafters. There is an indication that the thinner the sheet steel and the greater the spacing of the rafters, the less the allowable load.

Depending upon the material in the lower face, sheet steel is high in the group and gypsum board is at the middle. Roof *DJ*, at the bottom, has no ceiling.

Depending upon the material in the upper face, all the roofs above the bottom have built-up roofing on wood sheathing or fiberboard. The roof at the bottom has sheet-steel box-rib roofing.

Concentrated load.—Roof *DJ*, which has box-rib roofing is at the top and is followed by built-up roofing over wood sheathing (next to the top) and fiberboard.

9. Relation of Materials and Design to Allowable Load-weight Ratio

For a house construction, efficient use of material requires a high ratio of allowable load to weight. Although this ratio is not of first importance in house design it should be considered—particularly for walls, load-bearing partitions, floors, and roofs above the first story—because if the ratio is low, the load on first-story walls and partitions may be greater than is necessary, and constructions having greater allowable loads are required for the first story.

To assist the architect when selecting a construction, the ratio of allowable load to weight was computed, for each construction. The values for walls are shown in figures 46 (compressive, transverse, and racking loads) and 47 (concentrated and impact loads). Those for partitions, floors, and roofs are shown in figure 48.

A study of these figures also should be helpful when developing new and unconventional constructions because they indicate the designs and materials having high load-weight ratios.

(a) Walls

The load-weight ratios for walls are shown in figures 46 and 47.

(1) Wood

Compressive load.—Wall *DR* is at the top, and has the greatest ratio of allowable load to weight. Experts on plywood house construction believe that some changes in the design would increase the allowable load and decrease the weight, thereby greatly increasing the load-weight ratio. Presumably these changes are economically practicable.

Wall *DR* may be considered a *"stressed-skin" construction* because the faces and frames are fastened by glue and, therefore, act together under load more completely than if fastened by nails only.

Apparently, gluing the faces to the structural members is conducive to high load-weight ratios because in walls *CN* and *CM* the fiberboard inside face is also glued to the studs and they are above the middle and higher than similar walls having the faces nailed to the studs.

Wall *CI*, above the middle, has gypsum-board inside face glued and nailed to the ribbon at midheight of the wall. Wall *DQ*, also above the middle, has fiberboard inside face glued to the light 2- by 3-in. studs. Wall *DK*, near the bottom, has fiberboard inside and outside faces glued to the light 2- by 2-in. studs. Wall *DL*, next below *DK* is demountable in pieces and both faces are removably attached to the studs by steel clips.

All the other walls have conventional frames except *CB*, plank vertical, above the middle.

Depending upon the material on the inside face, the load-weight ratio decreases in the order: glued plywood, glued fiberboard, nailed plywood, nailed fiberboard, and nailed fiberboard lath with plaster.

Depending upon the material in the outside face, nailed fiberboard either bare or overlaid

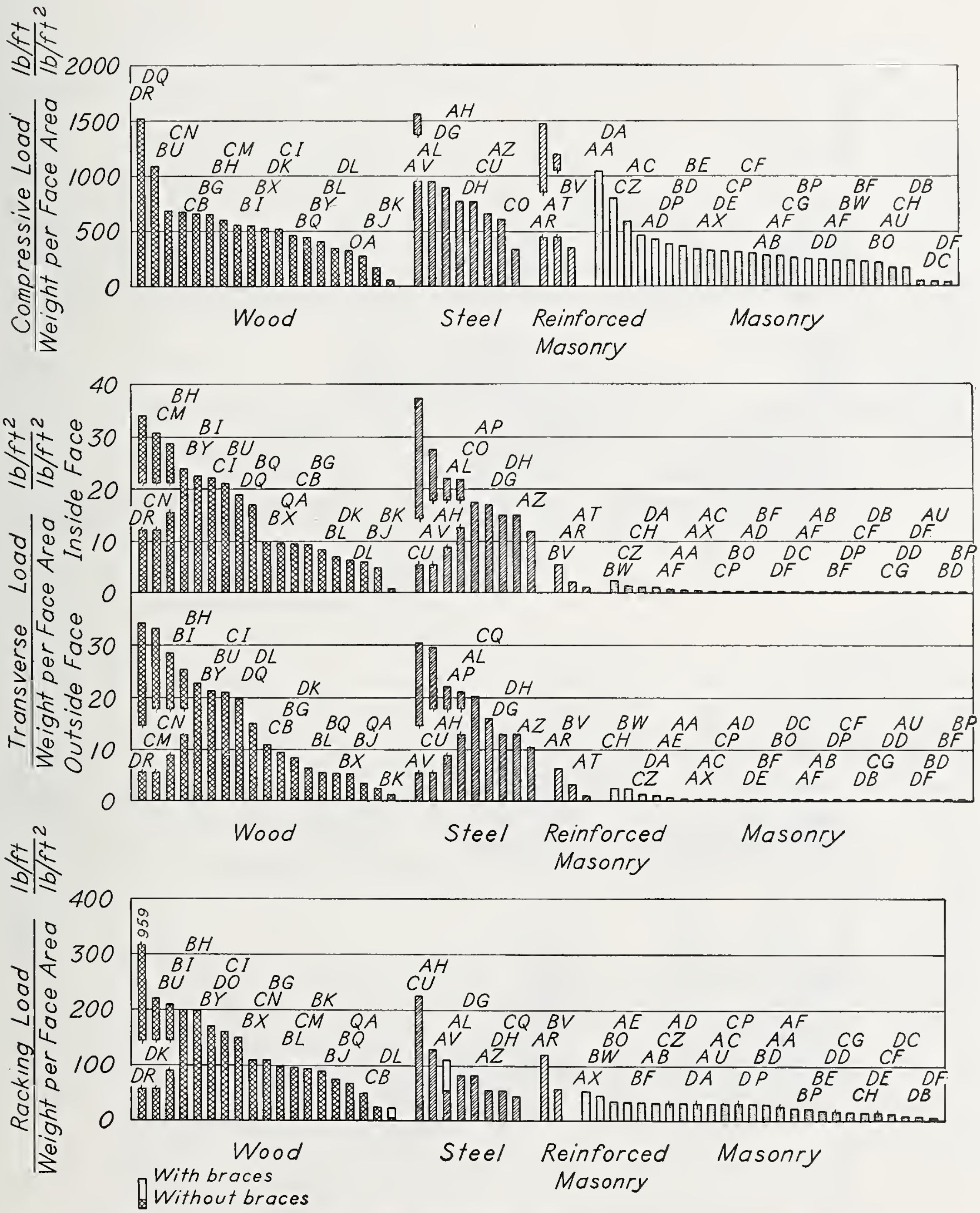

FIGURE 46.—*Allowable compressive, transverse, and racking load-weight ratios for walls.*

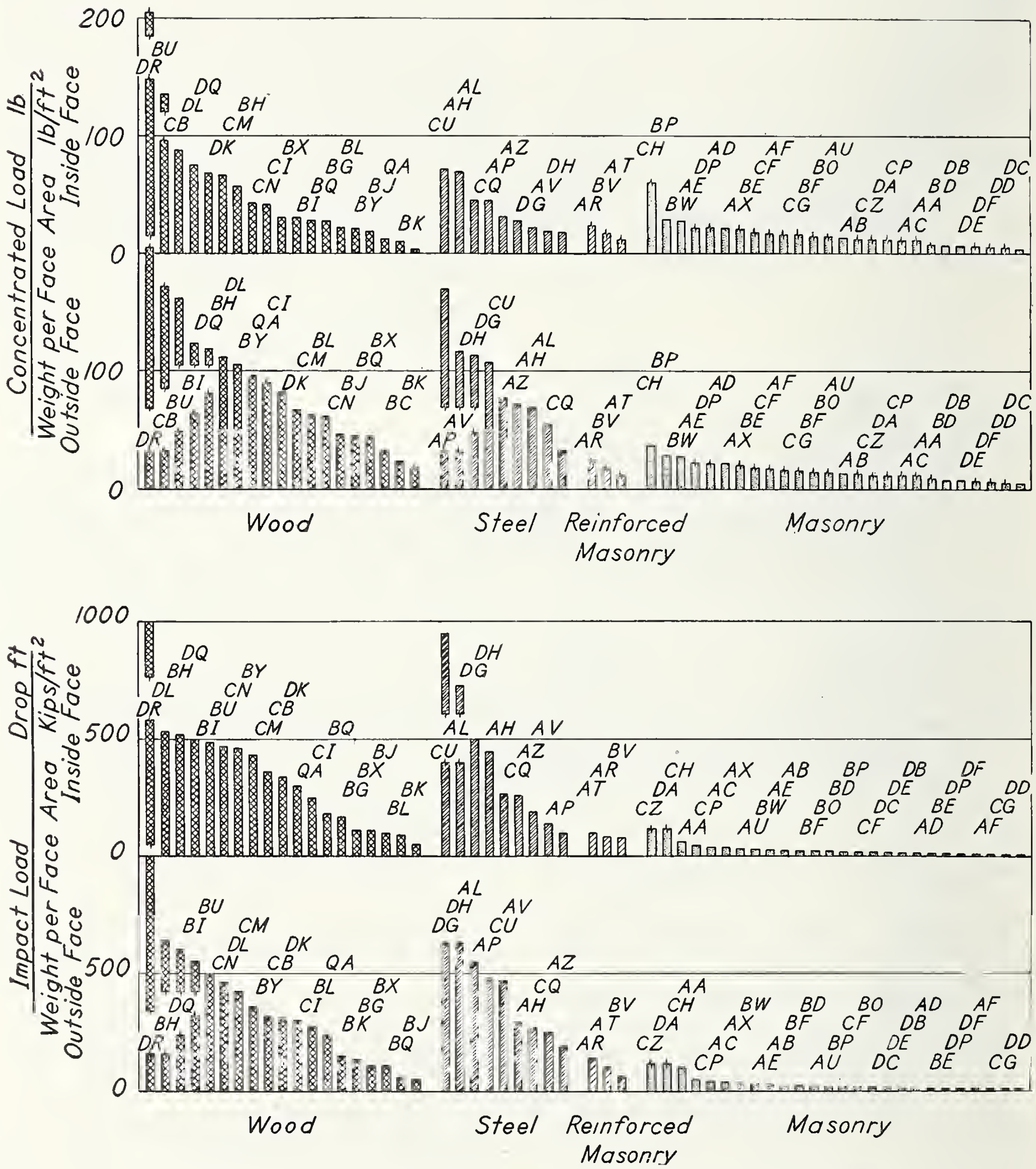

FIGURE 47.—*Allowable concentrated and impact load-weight ratios for walls.*

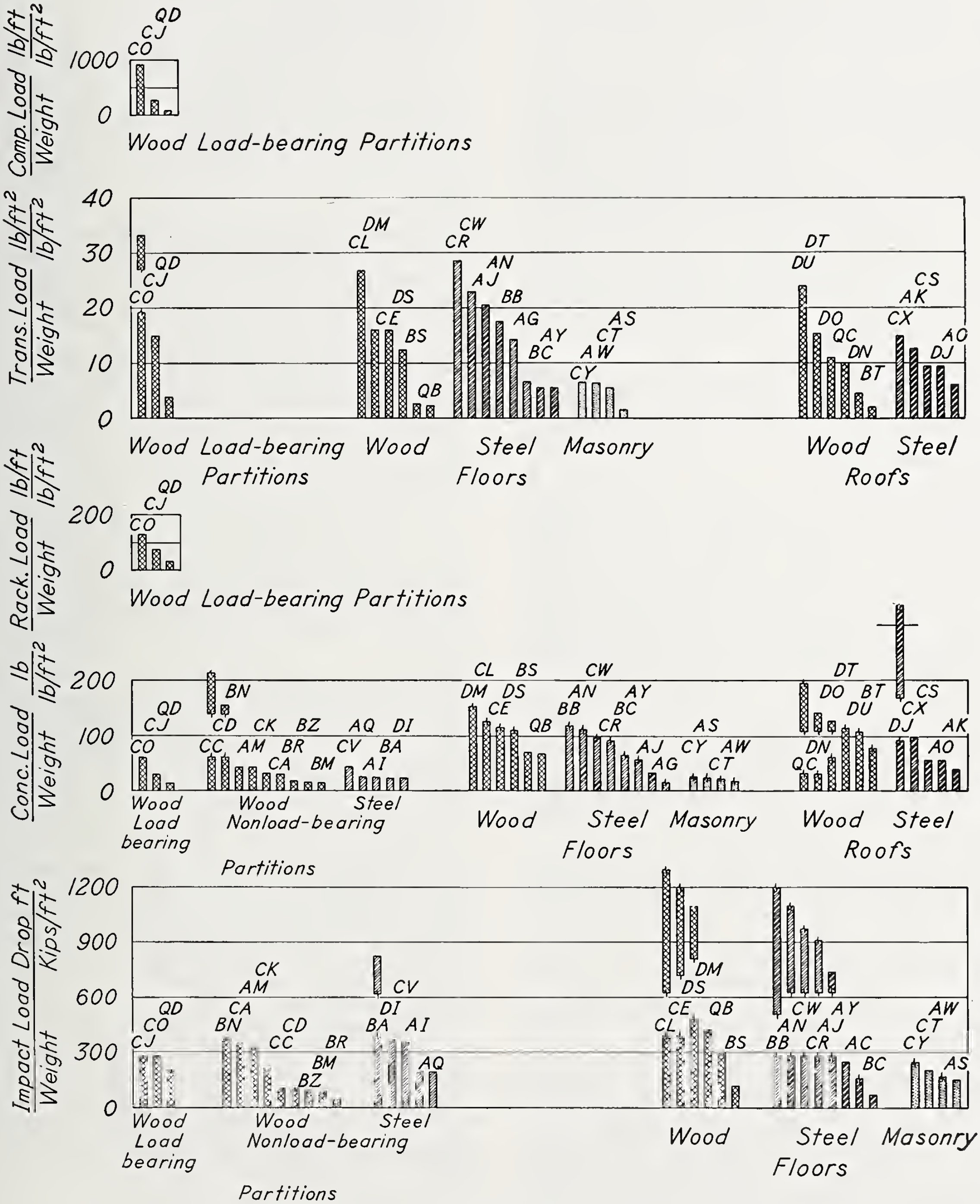

FIGURE 48.—*Allowable load-weight ratios for partitions, floors, and roofs.*

with siding or shingles is scattered throughout the group, glued plywood is at the top, nailed plywood, shingles, and siding, are above the middle, and nailed wood sheathing with siding is at the middle, and, finally, stucco and brick veneer are at the bottom.

Transverse load, inside.—The architect is reminded that, when selecting a wall for transverse (wind) load, it is necessary only that the allowable load be adequate for the particular house. Therefore, he may select the wall for compressive load and then check to make certain that the transverse load is adequate.

Wall *DR* is at the top, and has the greatest allowable load, *DQ* (light frame, glued inside face) is above the middle, *CB* (plank) is below the middle, *DK* (light frame, glued both faces) is near the bottom, and *DL* (demountable in pieces) is still lower.

Depending upon the materials on the inside face, glued plywood and fiberboard are at the top, nailed fiberboard is above the middle, and nailed wood and fiberboard lath with plaster are below the middle.

Depending upon the materials in the outside face, glued plywood is at the top and nailed plywood is above the middle but nailed wood sheathing with siding is just below the middle.

Fiberboard is scattered through the group but stucco and brick veneer are at the bottom.

Transverse load, outside.—As for inside loading, wall *DR* is at the top, *DQ* (light frame, glued inside face) just above the middle, and *CB* (plank) just below. However, *DL* (demountable in pieces) is now at the middle, probably because the pieces of the outside face are about 10 in. high, but the inside face is one piece from top to bottom of the wall. *DK* (light frame, glued both faces) is somewhat higher in the group.

Depending upon the materials in the faces, the order is the same as for inside transverse loading, except that wood sheathing with siding outside is next to the bottom.

Racking load.—Wall *DR* is at the top, having the greatest load-weight ratio.

Depending upon the materials in the faces, and disregarding overlays, outside, (siding, shingles) plywood glued is at the top, then fiberboard both faces glued (*DK* 2- by 2-in. studs, spaced 1 ft 0 in.), nailed plywood, nailed fiberboard, glued gypsum board and nailed plywood are all above the middle of the group. They are followed by fiberboard with both faces nailed and plaster inside; fiberboard on both faces, one glued, the other nailed, at the middle; fiberboard with both faces nailed, and nailed fiberboard with plaster inside; stucco or brick veneer are well below the middle; and lowest is nailed wood and nailed fiberboard lath with plaster inside and nailed wood sheathing outside.

The plank wall *CB* is next to the bottom and at the bottom is the demountable wall DL braced and unbraced.

Concentrated load, inside.—The allowable load-weight ratios are shown in figure 47.

Although the spacing of the structural members is not of great importance for the concentrated load, all the walls with less than 1 ft 4 in. spacing are well above the middle of the group, as are those with glued inside faces.

Depending upon the material on the inside face, all the plywood is at the top; fiberboard is next lower, most of which is above the middle. Gypsum board is at the middle, followed by all walls with plaster.

Concentrated load, outside.—Depending upon the material exposed on the outside face, plywood and plank are near the top, with wood siding well scattered through the group. Fiberboard only and wood shingles are near the middle but stucco is well below the middle and brick veneer is at the bottom.

Impact load, inside.—Wall *DR* is at the top, and has the greatest load-weight ratio. Next is *DL*, demountable in pieces, faces attached by steel clips. The plank wall, *CB*, is at the middle and the walls with fiberboard glued to light studs, *DQ* and *DK*, are above and at the middle.

Depending upon the material on the inside face, plywood, both glued or clipped to the studs, is at the top followed by fiberboard either glued or nailed and plywood nailed, most of which is above the middle. Nailed gypsum board, wood, and fiberboard lath with plaster, are all below the middle.

Depending upon the material in the outside face, plywood, both glued or clipped, is at the top. Sheathing of wood, fiberboard, and plywood, overlaid with wood siding or shingles, all nailed, are scattered from near the top to near the bottom. Fiberboard only, glued, is at the middle. Brick veneer and stucco are at and near the bottom.

Impact load, outside.—Wall *DR* is at the top and, with a few exceptions, the order is the same as for impact loading inside.

Wall *DL*, demountable, is nearer the middle, and with the exception of the comments on clipped faces, the discussion for impact inside applies to impact outside.

(2) *Steel*

The load-weight ratios are shown in figure 46.

Compressive load.—Wall AV is at the top, having the greatest load-weight ratio. Apparently there is no relation between the load-weight ratio and the thickness of the sheet-steel in the studs or the yield strength. The studs in *AV* are formed to a more intricate cross section than the others which may account in part for the high ratio.

Depending upon the material on the inside face, fiberboard and sheet steel alternate through the group, except that *AZ*, next the bottom, has gypsum-board lath with plaster on the inside face.

Depending upon the material in the outside face, sheet steel extends from the top to below the middle of the group. Fiberboard overlaid with siding or shingles is near the bottom.

Transverse load, inside and outside.—For both inside and outside transverse loading, the order is much the same. There is no very obvious explanation for the differences in load-weight ratios. Gypsum board only and gypsum-board lath with plaster are below the middle and at the bottom respectively.

Racking load.—There are many materials in the faces and many kinds of fastenings. Apparently there is no consistent relation between these factors and the load-weight ratios.

For wall *AV*, the ratio depends greatly upon whether there are diagonal braces extending from the top to the bottom of the wall.

Concentrated load, inside.—The load-weight ratios are shown in figure 47.

It is evident that the load-weight ratio depends on the material on the inside face. There is no relation between the ratio and the spacing of the structural members.

Sheet steel is above the middle of the group, gypsum board and gypsum-board lath with plaster are at the middle and fiberboard only is below the middle.

Concentrated load, outside.—Again there is no relation between the spacing of the structural members and the load-weight ratio.

Each of the materials in the outside face appear to be scattered through the group.

Impact load, inside.—Apparently there is no consistent relation between the design of the walls and the materials to the load-weight ratio.

Depending upon the material on the inside face, in general, the order is sheet steel, fiberboard only, gypsum-board lath with plaster, and gypsum board only.

Impact load, outside.—Depending upon the materials on the inside face, in general the order is fiberboard only, gypsum board, sheet steel and, at the bottom is gypsum-board lath with plaster.

Depending upon the materials in the outside face, in general the order is box-rib sheet-steel siding at the top, plywood next, sheet steel next lowest and, at the bottom, plywood with shingles.

(3) *Reinforced Brick and Concrete*

The load-weight ratios are shown in figure 46.

Compressive load.—Wall *AR*, which is at the top, has the greatest load-weight ratio. This wall, a reinforced hollow concrete slab, is followed by the reinforced brick wall, *AT*, and the reinforced slab having flanges, *BV*.

Transverse load, inside.—Wall *BV*, flanged slab, is at the top; it is followed by *AR*, hollow slab, and lowest is *AT*, solid wall.

Transverse load, outside.—Hollow slab *AR* is at the top; flanged slab *BV* is next; and lowest is solid wall *AT*.

Racking load.—Racking loads were not applied to wall *AT*, reinforced brick. Wall *AR*, hollow slab, is above *BV*, flanged slab.

Concentrated load, inside and outside.—The load-weight ratios are shown in figure 47.

For all these walls, the concentrated loading was applied to the concrete or brick face and discontinued at 1,000 lb. Although the allowable loads are the same, the weights are different.

Impact load, inside.—Wall *AT*, brick, is at the top; *AR*, hollow slab follows; and *BV*, flanged slab is lowest.

Impact load, outside.—Wall *AR*, hollow slab, is at the top; *AT*, brick follows; and lowest is *BV*, flanged slab.

(4) *Masonry*

Compressive load.—The load-weight ratios are shown in figure 46.

Wall *AA*, solid, high-strength brick, is at the top, and has the greatest load-weight ratio. It is followed by plain concrete *DA* and *CZ*.

Medium-strength brick is scattered above the middle.

Structural clay tile on end, *AD*, is above the middle of the group and tile on side, *AE*, is below the middle. Cavity walls, *BD*, brick; and *AX*, concrete block; are above the middle; but *AU*, brick with tile on side, is near the bottom. Walls of concrete block and solid concrete units laid to leave air spaces are scattered near the middle.

Of the earth walls, *DE*, terracrete block, is above the middle and *DD*, monolithic terracrete, is below. The others: adobe, *DB*, bitudobe, *DC*, and rammed earth, *DF*, are shown together at the bottom of the group.

Walls *BW* and *CH*, concrete slabs attached to vertical splines, are well below the middle, as are *BF* and *BO*, concrete block with brick facing.

Transverse load, inside.—Walls *BW* and *CH*, concrete slabs attached to vertical splines, which are at and near the top, have the greatest load-weight ratio. They are followed by *CZ* and *DA*, plain concrete.

Structural clay tile wall *AE*, on side, is near the top and *AD*, on end, is near the middle of the group.

Cavity wall, *AX*, concrete block, is well above the middle, but *AU*, tile on side with brick, and also *BD*, brick, are near the bottom.

AA, high-strength brick, is well above the middle and around the middle are the medium-strength brick walls.

Walls *BO* and *BF*, concrete block with brick facing, are just above the middle.

Two earth walls, *DE*, terracrete block, and DC, bitudobe, are near the middle but the others are much lower.

The concrete block walls and the concrete units laid to leave air spaces are all below the middle.

Transverse load, outside.—Walls *CH* and BW, concrete units attached to splines, are at the top. For outside loading, the order is about the same as for inside loading and the discussion is the same.

Racking load.—Cavity wall *AX*, concrete block, which is at the top, has the greatest load-weight ratio. The other cavity walls are near the middle of the group.

Wall *BW*, concrete units attached to vertical splines, is next to the top but similar wall, *CH*, is well below the middle.

Walls *BO* and *BF*, concrete block with brick facing, are near the top and lower is structural clay tile on side; still lower is clay tile on end.

Plain concrete is above the middle. Medium-strength brick is at or above the middle but high-strength brick is below the middle.

Concrete block is below the middle, concrete units laid to enclose air spaces are still lower.

Terracrete walls are near the bottom and the three other earth walls are at the bottom.

Concentrated load, inside and outside.—The load-weight ratios are shown in figure 47.

For the concentrated load, the order is the same for loading inside and outside. With one or two exceptions, the loading was discontinued, therefore the allowable loads are 900+ lb. The differences in the load-weight ratios are due to differences in the weights of the walls.

Almost all the walls above the middle of the group have air spaces in the wall and most of those below are solid walls. All the earth walls are at the bottom.

Impact load, inside and outside.—With the exception of walls *AU* and *BO* (different materials in the two faces) which are several places lower when loaded outside than when loaded inside, the order is the same for both loadings.

The plain concrete walls *CZ* and *DA* are at the top, having the greatest load-weight ratios.

Most of the brick walls are above the middle of the group as are the cavity walls *AX*, *BD*, and *AU*, also *CH* and *BW*, concrete units attached to vertical splines.

Structural clay tile on side, *AE*, is above the middle and the same tile laid on end, *AD*, is below the middle.

Concrete block with brick facing, *BF* and *BO*, are near the middle.

Concrete block is below the middle, as are concrete units laid to enclose air spaces.

All the earth walls are below the middle.

(*b*) *Load-bearing Partitions*

The allowable load-weight ratios are shown in figure 48.

(*1*) *Wood*

All the load-bearing partitions have conventional wood frames, therefore differences in the ratios are due to differences in the faces.

Compressive, transverse, racking, and concentrated load.—Partition *CO* is at the top, having the greatest load-weight ratio. The faces on this partition are fiberboard glued to the studs. Next is *CJ*, which has gypsum-board faces glued and nailed to the ribbon at mid-height and nailed to the other members of the frame. Last is *QD* (conventional) which has wood lath, nailed, with plaster faces.

Impact load.—Partition *CJ* is at the top, followed by *CO* and *QD* (conventional).

(*c*) *Nonload-bearing Partitions*

The allowable load-weight ratios are shown on figure 48.

(*1*) *Wood*

Concentrated load.—Partitions *CC* and *CD*, plywood, ¼ in., nailed, are at the top, and have the greatest load-weight ratios.

Fiberboard, ½ in., nailed, is about the middle of the group. At the middle is *CK*, gypsum board, ½ in., glued and nailed to the ribbon at midheight, and also nailed to the frame. The three partitions at the bottom have nailed fiberboard lath with plaster.

Impact load.—Partitions *BN*, *CA*, and *AM*, fiberboard, ½ in., nailed, are at the top, having the greater load-weight ratios. *CK*, gypsum board, ½ in., glued and nailed, is just above the middle and *CC*, plank, is at the middle.

Partition *CD*, studs 2 by 2 in., spaced 1 ft 0 in., plywood ¼ in., is just below the middle, followed by the three partitions having conventional frames and nailed fiberboard lath with plaster.

(2) *Steel*

Concentrated load.—For the steel nonload-bearing partitions, there is an indication that the load-weight ratio is less the greater the spacing of the structural members.

Partition *CV*, which is at the top, has the greatest load-weight ratio. On this partition the faces are sheet steel. Next are *AQ* and *AI*, gypsum board screwed, then *BA*, nailed gypsum-board lath with plaster and, finally, *DI*, fiberboard clipped to the frame.

Impact load.—Partition *BA*, which is at the top, has the greatest load-weight ratio. The faces are nailed gypsum-board lath, with plaster. The next, *DI*, has fiberboard fastened by clips.

At the middle of the group is *CV*, which has sheet-steel faces, followed by *AI* and *AQ* with gypsum board fastened by screws.

(*d*) *Floors*

The load-weight ratios are shown in figure 48.

(*1*) *Wood*

Transverse load.—Floor *CL*, conventional frame, which is at the top, has the greatest load-weight ratio. The other floors having conventional frames, *BS* and *QB*, are at the bottom. Those having unconventional frames are intermediate.

Depending upon the material in the lower face (ceiling), the floors having no ceiling, *CL* and *CE*, are at and near the top; plywood, *DM* and *DS*, follow.

Floor *BS*, fiberboard lath with plaster, and *QB* (conventional), wood lath with plaster, are at the bottom.

Depending upon the material in the upper face, most of the plywood is above the middle and all with wood subfloor and finish floor are at the bottom.

Concentrated load.—Floor *DM*, demountable in pieces, which is at the top, has the greatest load-weight ratio. Depending upon the material in the upper face, most of the plywood is above the middle and most of the wood subflooring is below.

Impact load.—Floor *CL*, which is at the top, has the greatest load-weight ratio. The frame is conventional, 2- by 8-in. joists, spaced 1 ft 4 in. The other floors *QB* and *BS*, having conventional frames are at the bottom. Those having unconventional frames are intermediate.

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Depending upon the material in the lower face (ceiling), the two floors *CL* and *CE*, which are at the top, have no ceiling; the next two, *DS* and *DM*, plywood ⅜ in. and the two at the bottom, *QB* and *BS*, have lath with plaster.

Depending upon the material in the upper face, nearly all the plywood is above the middle of the group and nearly all the wood subflooring is below.

(*2*) *Steel*

Transverse load.—Floor CR, which is at the top, has the greatest load-weight ratio. Floors *BB* and *BC*, having channel-shaped joists, are below the middle of the group.

Depending upon the material in the lower face (ceiling), sheet steel is above the middle, except for *AG*, concrete fill. The floor having no ceiling, *BB*, is at the middle. Floors *BC* and *AY*, which are at the bottom, have lath with plaster ceilings.

Depending upon the material in the upper face, all the floors have wood finish flooring over either wood subflooring or wood sleepers, except *AG*, which has a concrete fill and composition finish floor.

Concentrated load.—Floor *BB*, which is at the top, has the greatest load-weight ratio.

Depending upon the material in the upper face, all the floors have wood finish flooring over wood subfloor or wood sleepers, except *AG*, which is at the bottom and has composition finish flooring over concrete.

Impact load.—Floor *BB*, which is at the top, has the greatest load-weight ratio.

Depending upon the material in the lower face (ceiling), the floor having no ceiling is at the top, followed by sheet steel to well below the middle. *AY*, which has metal lath with plaster, is below the middle and *BC*, which is at the bottom, has gypsum-board lath with plaster.

(*3*) *Reinforced Concrete*

Transverse load.—Floor *CY* which is at the top, has the greatest load-weight ratio. The joists are precast reinforced concrete. Floors *AW* and *CT*, which are next, have expanded steel joists and floor *AS*, which is lowest, is hollow concrete slab.

Concentrated load.—For all reinforced floors, the concentrated allowable load is 900+ lb. Therefore, the differences in the load-weight ratios are due to differences in the weights of the floors.

Floor *CY* with precast joists, which is at the top, is followed by *AS*, hollow slab; below the middle of the group are *CT* and *AW* with expanded steel joists.

Impact load.—Floor *CY*, which is at the top, has the greatest load-weight ratio. This precast-joist floor is followed by the two floors having expanded steel joists, and the hollow slab.

(*e*) *Roofs*

The load-weight ratios are shown in figure 48.

(*1*) *Wood*

Transverse load.—The cellular plywood roofs *DU* and *DT*, which are at the top, have the greatest load-weight ratios.

Floor *DO*, demountable in pieces, with 8-in. joists, is just above the middle and *DN*, similar except for 6-in. joists is next to the bottom.

Roof *QC*, conventional, is just below the middle. The joists are 2 by 6 in., spaced 2 ft 0 in., and there is no ceiling. The other conventional frame roof, *BT*, which has joists 2 by 8 in., spaced 1 ft 4 in., and a plaster ceiling, is at the bottom.

Depending upon the material in the lower face (ceiling), the two roofs, *DU* and *DT*, which have plywood ceilings (stressed-skin), are at the top. The next three have no ceiling and the one at the bottom, *BT*, has a plaster ceiling.

Depending upon the material in the upper face, the two roofs, *DT* and *DU*, with 5/8-in. plywood (stressed-skin), are at the top and *DO* and *DN*, fiberboard overlaid with 3/8-in. plywood (demountable in pieces), are lower in the group.

Conventional roof *QC*, which has wood sheathing with wood shingles, is at the middle of the group and *BT*, which has sheathing with built-up roofing, is at the bottom.

Concentrated load.—Except for roofs *DN* and *DO*, upper face 3/8-in. plywood, the concentrated allowable load is 900+ lb. Therefore, the differences in the load-weight ratios, to a great extent, are due to differences in the weights.

Roof *QC* (conventional), which is at the top, has the greatest load-weight ratio. The upper face is wood sheathing overlaid with shingles. *DN* and *DO*, 3/8-in. plywood, are next to the top; *DT* and *DU*, 5/8-in. plywood, follow; and *BT*, wood sheathing overlaid with built-up roofing, is lowest in the group.

(*2*) *Steel*

Transverse load.—In general, the less the depth of the structural members the less the load-weight ratio.

Roof *CX*, which is at the top, has the greatest strength-weight ratio.

Depending upon the material in the lower face (ceiling), sheet steel is scattered from top to bottom of the group; CS, gypsum board, is at the middle; and *DJ*, which has no ceiling, is next to the bottom.

Depending upon the material in the upper face, all the roofs except *DJ* have built-up roofing over wood sheathing. *CX*, which is at the top, has fiberboard. DJ, which is next to the bottom, has box-rib sheet-steel roofing.

Concentrated load.—Roof *DJ*, which is at the top, has the greatest load-weight ratio. The upper face is 0.0245-in. sheet-steel roofing. Next is *CX*, which has built-up roofing over wood sheathing. All the other roofs have built-up roofing over fiberboard.

10. Allowable Loads for Constructions not Having Standard Height or Span

(*a*) *Compressive Load*

Walls and load-bearing partitions are the only elements of a house subjected to compressive loading.

In general, the higher the wall the less the allowable load and the greater the shortening and lateral deflection because many of the walls and partitions are "long columns."

A wall height of 10 ft may be taken as the greatest for the average house. A study of the walls and partitions indicated that, up to this height, the allowable load in this report is safe except for wood walls *DQ* and *DK*, which have light conventional frames.

For a wall or partition of any height, the allowable load, also, the shortening and lateral deflection, may be determined most satisfactorily by testing specimens having the height of the walls in the building.

(*b*) *Transverse Load*

Walls, load-bearing partitions, floors, and roofs are the elements subjected to transverse loading. For walls and partitions, the height determines the span.

In general, the greater the span the less the allowable load and the greater the deflection. If the span is less than the standard span, the load is greater than the allowable load for the standard span up to the load causing shear failure. It is believed that under the maximum transverse load, most of the constructions failed by rupture of members under tensile and compressive stresses. No tests of short specimens were made to determine the shearing strength, therefore, there is no way to compute the allowable load for spans less than the standard span from the data in the Building Materials and Structures reports. If it were not for the possibility of shear failure, the greater allowable load for short spans could be computed by the same method as for long spans.

Usually in a house, the span of the floor in a bath room or kitchen is much less than for the other rooms but the loads are much greater. If the floor construction is the same for all rooms, the price should be less than if a different construction is required for bath rooms and kitchens. Tests of short span floors probably are worth while.

For spans greater than standard, the allowable load should be computed by finding from the standard span location of loads and allowable load, the bending moment at midspan, then the load on the long span causing the same moment. The deflection, also, may be computed.

For a wall or partition, this load is the allowable load because the weight of the construction has no effect on either the transverse load or the deflection.

For a floor or roof, the weight should be added to the allowable load when computing the bending moment and subtracted from the computed load on the long span to obtain the allowable load which may be applied over the long span.

(*c*) *Concentrated Load*

Under concentrated loading, the allowable load as well as the indentation and set are independent of the height of a wall or partition or the span of a floor or roof.

(*d*) *Impact Load*

The allowable impact load for walls, partitions, and floors having heights or spans either greater or less than those tested, cannot be determined by computation.

For a house construction not having the standard height or span reported in the Building Materials and Structures series, specimens should be tested and the allowable height-of-drop determined by application of the criteria.

(*e*) *Racking Load*

Walls and load-bearing partitions are the only elements subjected to racking loads.

The allowable load (shear) is independent of the height of the wall, therefore the allowable load given in this report may be applied to a wall or partition of any height. However, the racking deformation is greater the greater the height. For any height, the deformation should be taken as directly proportional to the height.

XIX. Comparison of Allowable Loads with Design Loads for Typical Houses

1. Objective

How do design loads for a house compare with the allowable loads? It is evident that, if the recommended methods give design loads greater than the allowable loads for the usual house constructions, there is something seriously wrong with the design loads, with the allowable loads, or with both.

Most of the houses in this country are conventional wood-frame construction. They have successfully withstood the wind and snow loads in most locations. Therefore, conventional wood-frame houses were selected for this comparison. Comparisons were also made for other constructions.

If the allowable load for a construction greatly exceeds the design load, the construction is overdesigned for the particular house and location. Some other construction which has less material and will cost less is indicated.

2. Typical Houses

The Federal Housing Administration furnished architect's plans for House 22-D, one story, and House E, two story, which were believed to be typical of small houses which had found wide acceptance. Both houses are conventional wood-frame construction.

House 22-D.—Elevations of this one-story house are shown on figure 49 and floor plans on figures 50 and 51.

House E.—Elevations of this two-story house are shown on figure 52 and floor plans on figures 51 and 53.

The values for elevation at midheight of story, for House E, do not agree exactly with those in tables 3, 4, and 5 because those in the tables were scaled from the plans while those on figure 52 were computed from the story heights given on the figure.

(*a*) *Constructions*

The dimensions and spacing of the studs and joists on the plans are almost the same as those for the conventional wood-frame constructions for which the structural properties are reported in BMS25 and which are also described

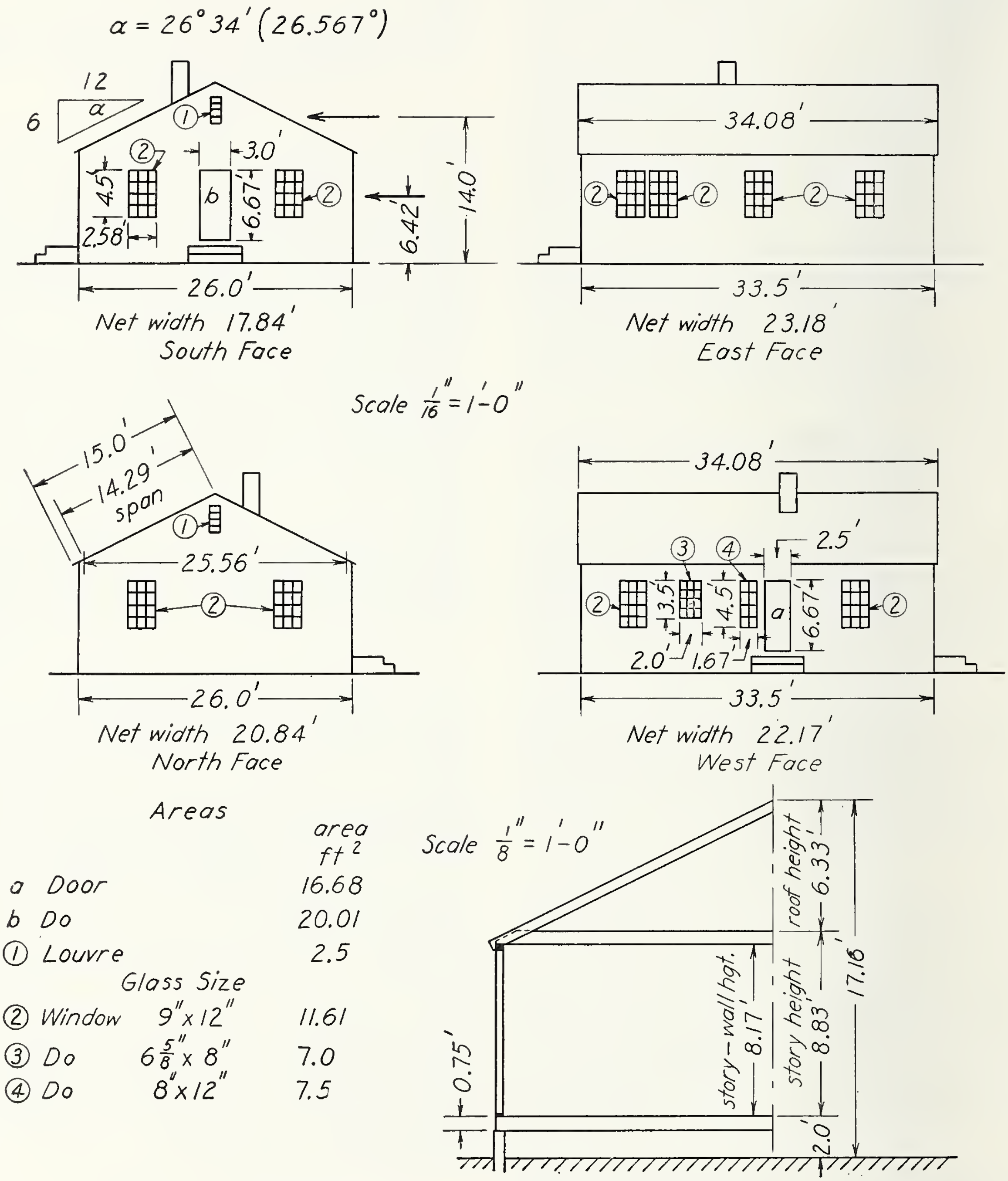

FIGURE 49.—*Elevations for one-story House 22-D.*

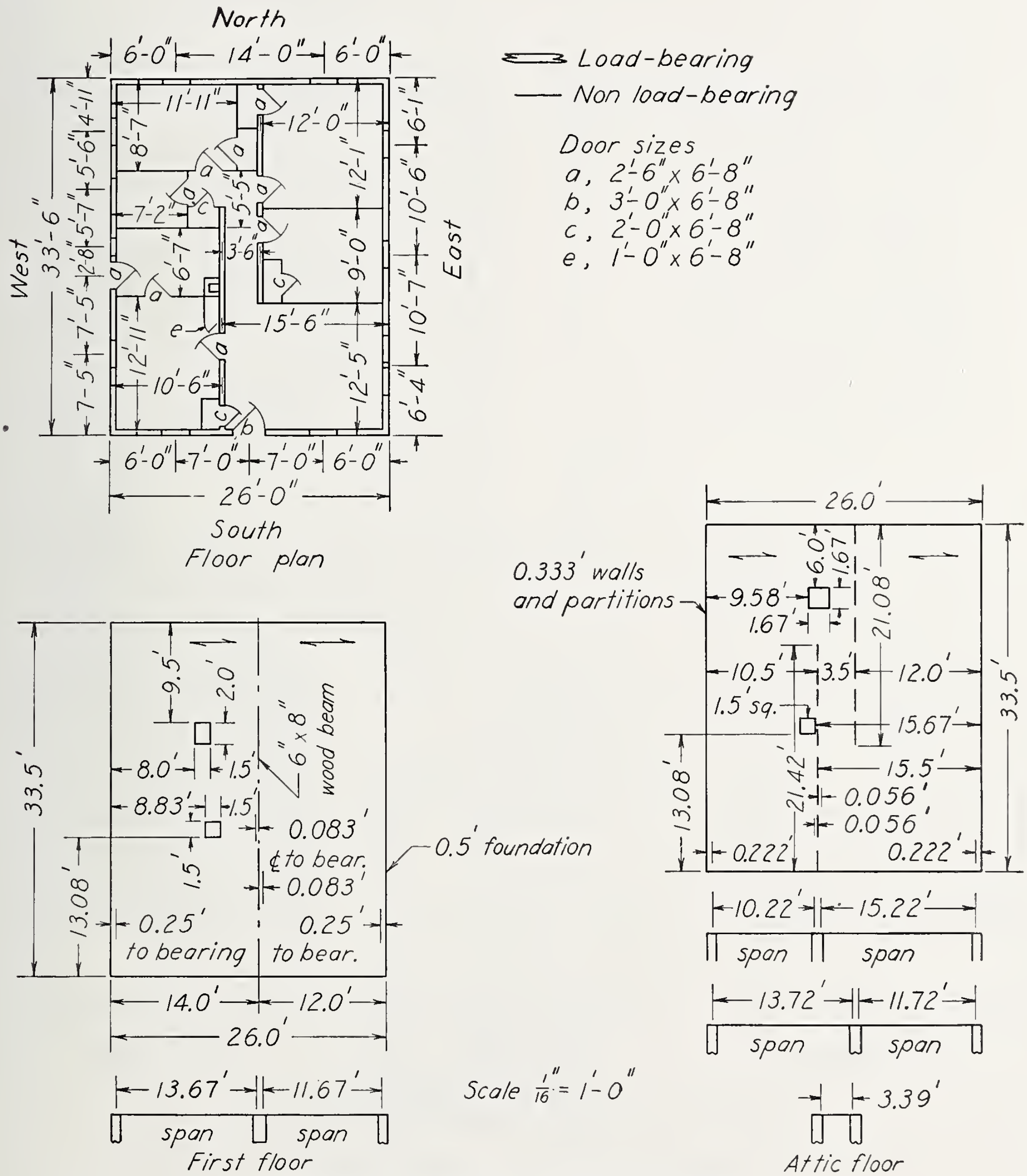

FIGURE 50.—*Floor plans for one-story House 22-D.*

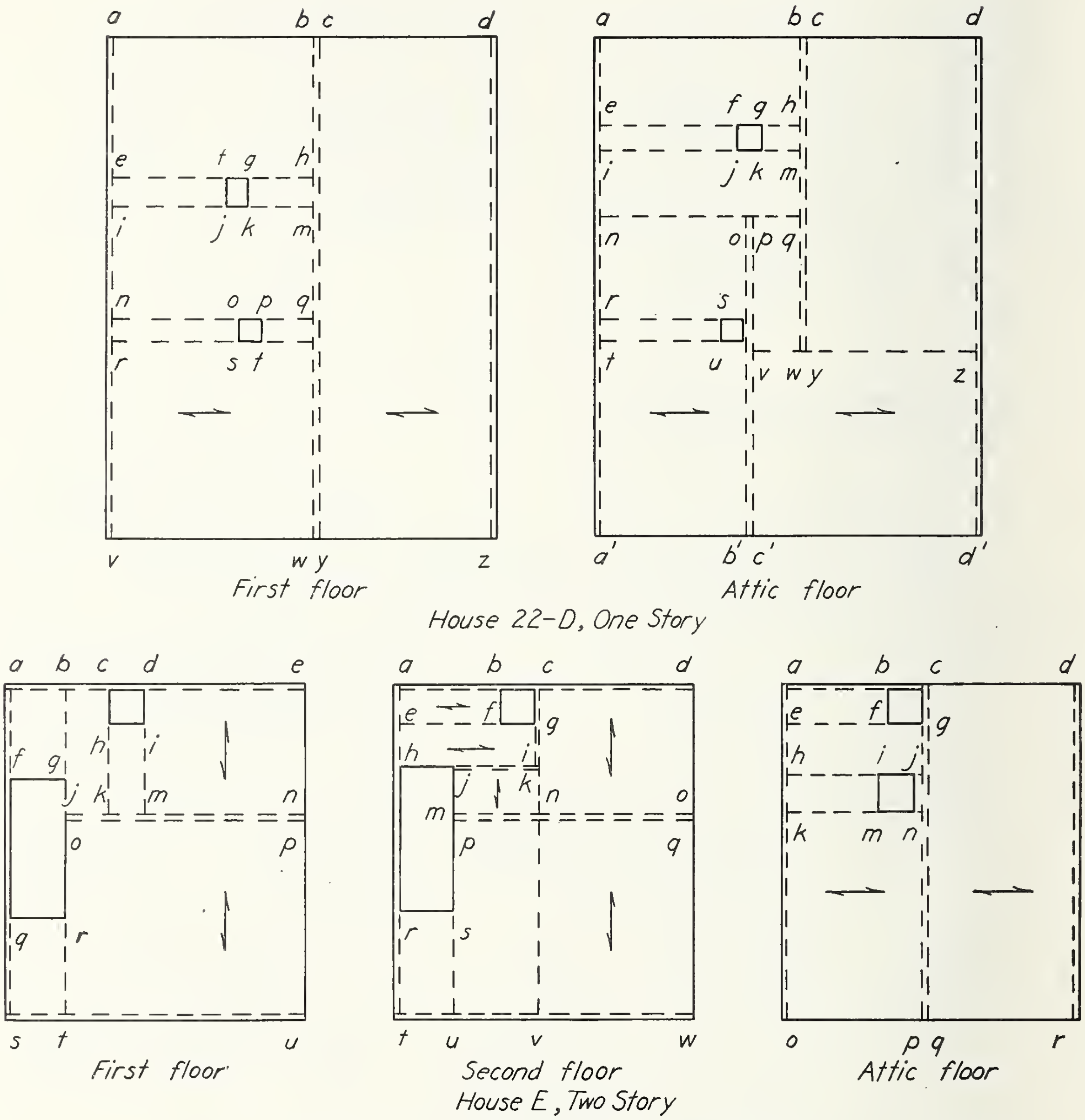

FIGURE 51.—*Floor plans for House 22-D and House E.*

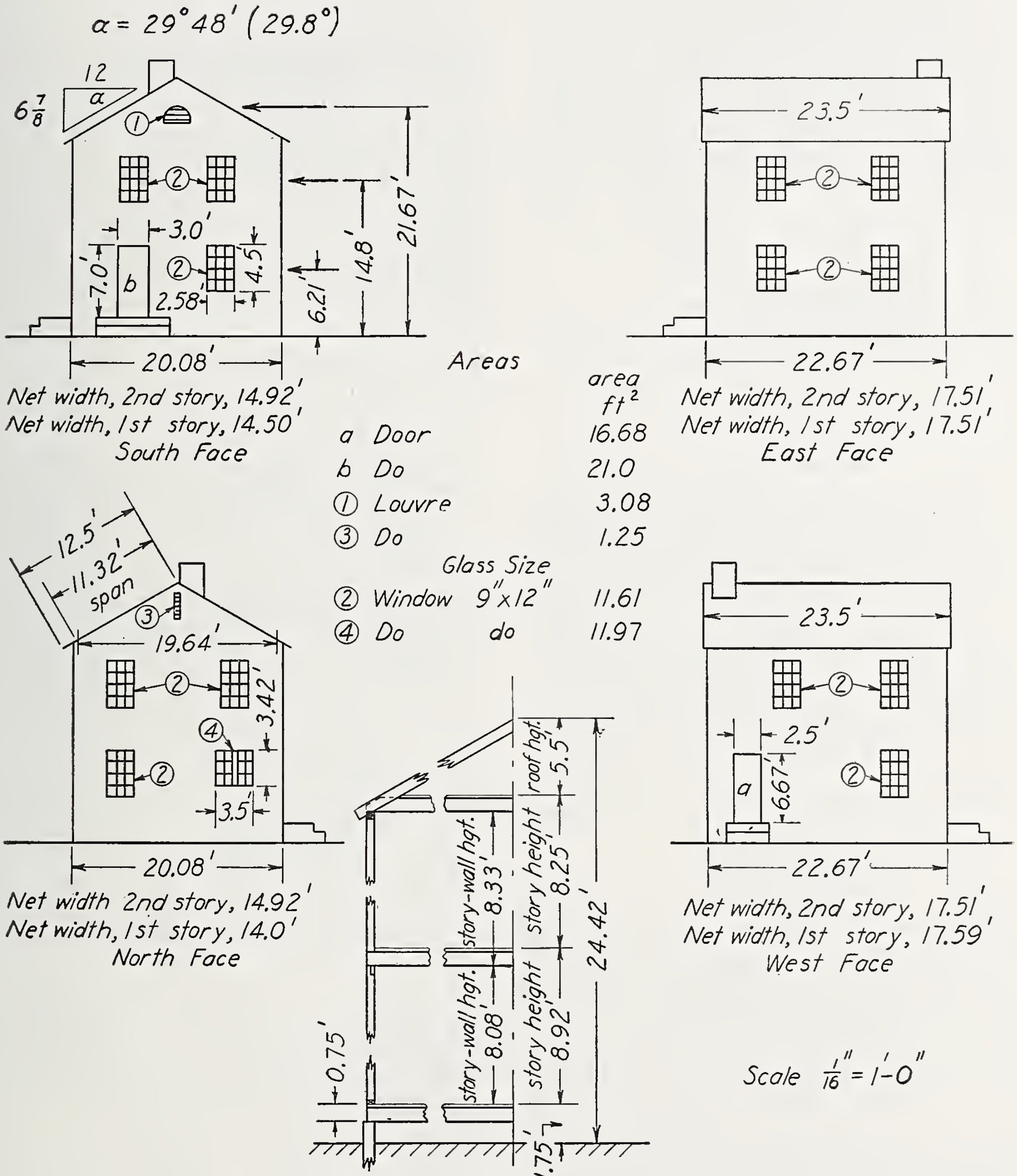

FIGURE 52.—*Elevations for two-story House E.*

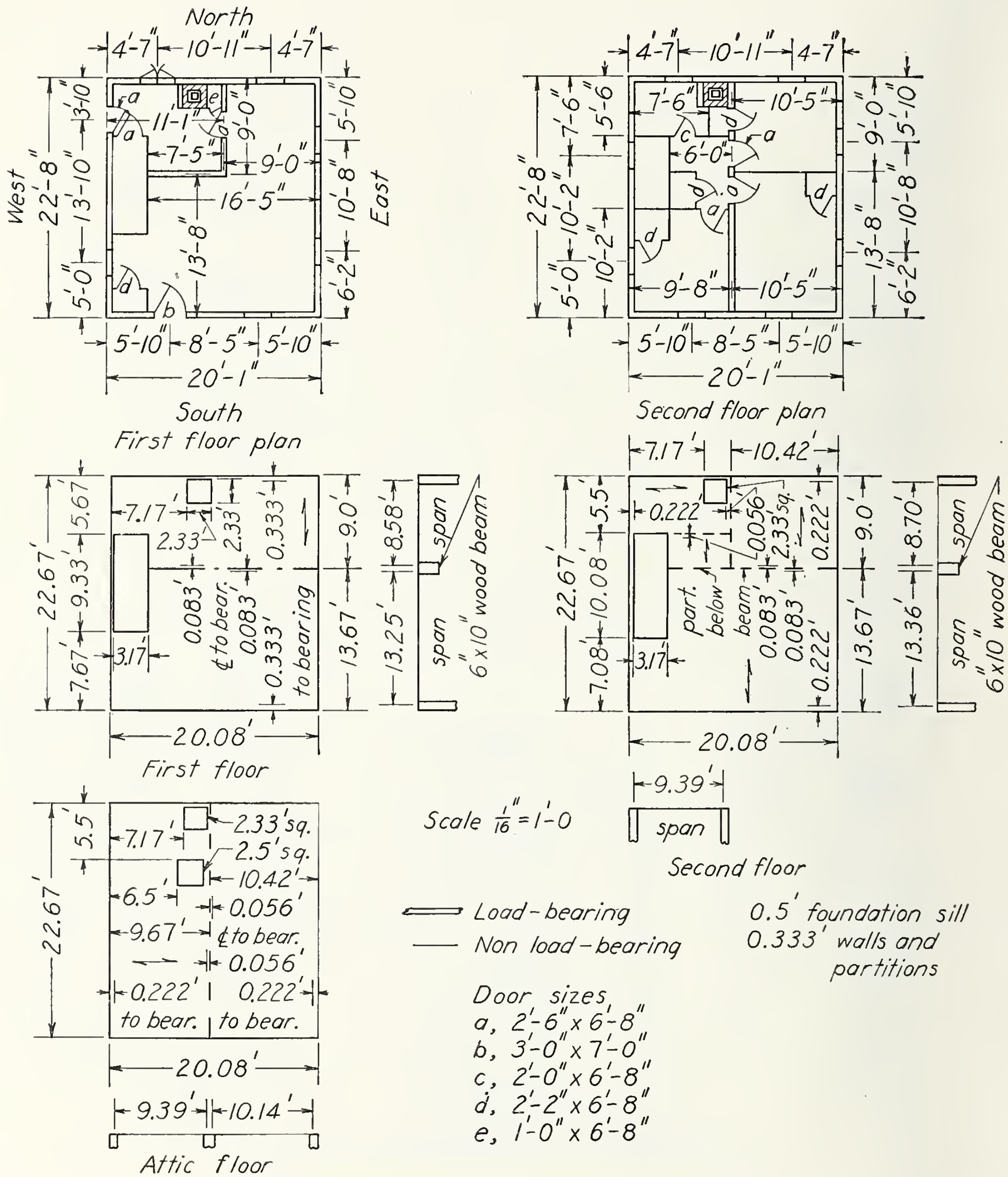

FIGURE 53.—*Floor plans for two-story House E.*

briefly in table 16. Therefore, it was assumed that these houses were the constructions in BMS25 for which the allowable loads have been determined.

Roofs.—Both roofs are taken as construction *QC*: 2- by 6-in. wood rafters spaced 24 in.; upper face, wood sheathing, paper, and wood shingles; lower face, none; weight 4.6 lb/ft^2.

Walls.—All the walls are taken as construction *QA*: 2- by 4-in. wood studs, spaced 16 in.; inside face, wood lath with plaster; outside face, wood sheathing, paper, and wood-bevel siding; weight 10.0 lb/ft^2.

Partitions.—All the partitions, both load-bearing and nonload-bearing, are taken as construction *QD*: 2- by 4-in. wood studs, spaced 16 in.; both faces, wood lath with plaster; weight 12.8 lb/ft^2.

Floors.—According to the plans, the attic floor (actually ceiling) in House 22-D is 2- by 6-in. wood joists, spaced 16 in. Presumably there is no upper face and the lower face is lath with plaster. No floor of this construction was included in the program, therefore no allowable load from laboratory data is available. For design, the weight is estimated as 8.0 lb/ft^2.

The attic floor in House E is also 2- by 6-in. wood joists, spaced 16 in. The upper face is matched flooring and the lower face is lath with plaster. No floor of this construction was included in the program, therefore no allowable load is available. For design, the weight is estimated as 10.6 lb/ft^2.

All the first- and second-story floors are taken as floor *QB*, which was constructed of 2- by 8-in. wood joists, spaced 16 in. The upper face consisted of wood subflooring, building paper, and wood finish floor; the lower face was wood lath with plaster; weight 13.2 lb/ft^2. Presumably the architect did not intend to have plaster on the lower face of the first floor, but no conventional wood-frame floor without plaster was included in the program.

(*b*) *Locations*

A few locations for the houses were selected from the Wind Map and the Snow Map where these loads are either about the greatest or least in the United States. The locations are

	Los Angeles Calif.	Miami Fla.	Portland Maine
	lb/ft^2	*lb/ft^2*	*lb/ft^2*
Basic velocity pressure	5.0	33.0	11.9
Basic snow load	0.0	0.0	28.0

3. Utility Factor

The ratio of the allowable load for the construction to the design load for the particular house and location is the *utility factor*. If it is less than one, the house is *unsafe*. If much greater than one, the construction is inefficient and other constructions having a smaller allowable load should be considered, taking into account the price, weight, durability, thermal conductivity, and fire resistance.

Because for economic reasons a given house construction must be suitable for a range of house design and locations, a construction may by considered satisfactory provided the utility factor is not less than one and does not exceed two.

4. Direction for Design Loads and Fastenings

If a direction is stated with the numerical value of a load, this direction is that in which the load or the reaction acts on the floor, wall, or roof under consideration.

5. Relation of Allowable Transverse Load to Span

An equivalent allowable transverse load for the span in the house was computed from the allowable loads in the tables which gave the same bending moment at midspan. These loads are inversely proportional to the squares of the spans.

For spans less than the test span, the Building Materials and Structures report on the construction was studied for indication of shear failure. If shear failure is reported, the computed load on the shorter span was decreased, if necessary, to a value for which the shear does not exceed the shear in the specimens tested.

6. Roof

(*a*) *Strength*

The transverse strength of roofs under loads applied to the lower face is not known because no roofs were tested under this loading. It is assumed that the transverse strength of roofs is the same, whether loaded on the upper face or the lower face.

In addition, the racking strength of roofs is unknown because racking loads were not applied to roofs.

To the allowable transverse loads for roofs given in table 24 was added the weight of the roof because this weight also is included in the design load.

The allowable transverse load for roof *QC*, adjusted for span, is House 22-D, 50.5 lb/ft^2 on span 14.29 ft; House E, 80.5 lb/ft^2 on span 11.32 ft. The design loads for roofs and fastenings, with utility factors, are given in table 27.

Table 27.—*Design loads for roof and fastenings*

[Loading condition A, south or north wind, windward openings, no snow; B, east or west wind, leeward openings, snow. U.F. =utility factor]

Design loads	House 22-D, one-story						House E, two-story						Loading conditions
	Los Angeles		Miami		Portland		Los Angeles		Miami		Portland		
LOADS ON ROOF													
Transverse:		*U.F.*		*U.F.*		*U.F.*		*U.F.*		*U.F.*		*U.F.*	
Out .... *lb/ft²*	0.9	-----	29.1	1.7	7.9	-----	1.6	-----	33.2	2.4	9.4	-----	A, with same load both slopes.
In .... *lb/ft²*	4.6	11.0	7.1	-----	23.9	2.1	5.7	14.2	15.1	-----	23.9	3.4	B, with load acting on windward slope. In Portland, snow on windward or on both slopes.
Span .... *ft*	14.29	-----	14.29	-----	14.29	-----	11.32	-----	11.32	-----	11.32	-----	
Parallel:													
Compression ... *lb/ft²*	92.3	-----	90.8	-----	513.2	-----	81.0	-----	130.8	-----	381.0	-----	B, with load acting on leeward slope adjacent to east or west wall.
Tension .... *lb/ft*	13.8	-----	436.3	-----	117.9	-----	18.0	-----	362.4	-----	102.8	-----	A, with load acting on both slopes adjacent to ridge.
LOADS ON FASTENINGS													
AT RIDGE													
Normal to ridge:													
Horizontal—	*lb/ft*		*lb/ft*		*lb/ft*		*lb/ft*		*lb/ft*		*lb/ft*		
Bearing	66.0	-----	49.5	-----	375.9	-----	55.1	-----	82.8	-----	262.1	-----	B, with snow on both slopes in Portland.
Anchor	15.4	10.2	487.8	0.3	131.8	1.2	20.7	7.6	417.6	0.4	118.5	1.3	A.
Vertical—													
Key	5.3	21.8	35.1	3.3	91.1	1.3	9.3	18.6	61.1	2.8	79.4	2.2	B.
Parallel to ridge:													
Key	0.0	-----	0.0	-----	0.0	-----	0.0	-----	0.0	-----	0.0	-----	Load always zero if symmetrical sloping roof.
AT WALL													
Normal to wall:													
Horizontal—													
Key, acts inward	71.5	5.6	85.3	4.7	388.9	1.0	60.7	6.6	119.4	3.4	275.3	1.5	B, with snow on both slopes in Portland. Load acts to windward on leeward slope.
Key, acts outward ...	0.0	-----	297.2	1.4	63.1	6.4	0.0	-----	227.7	1.8	50.1	8.0	A.
Vertical—													
Bearing, acts up	69.8	-----	74.4	-----	384.9	-----	66.4	-----	116.5	-----	308.1	-----	B, with snow on both slopes in Portland. Load acts on windward slope.
Anchor, acts down ...	0.9	386.7	392.5	0.9	97.4	3.6	7.2	48.3	369.6	0.9	96.5	3.6	A, with same load on both slopes.
Parallel to wall—													
Key	3.2	126.0	21.0	19.2	7.6	53.0	3.6	112.0	23.8	16.9	8.6	46.9	A, with same load on east and west walls, acting to windward on both slopes at wall.

Roof *QC* is safe for both houses in all three locations. For Los Angeles and Portland, however, the factor is so great that a less expensive roof should be developed.

(*b*) *Roof Fastenings*

Because the rafters in roof *QC* are spaced 24 in., the values for load parallel to slope and for fastenings must be doubled to obtain the load on each rafter. The parallel loads are much less than allowable tensile and compressive loads on wood loaded parallel to the grain. These 2- by 6-in. rafters have a nominal sectional area of 12 in^2.

In the past, many houses have been demolished by severe wind storms and hurricanes. The opinion has been expressed from time to time that many of these failures would not have occurred if the fastenings had been stronger, particularly the roof fastenings.

The design loads for fastenings appear great. Therefore, an attempt was made to determine just how rafters usually are nailed at plate and ridge in order that the strength of these fastenings could be computed. This detail of the construction is generally left to the contractor who follows trade practice.

(*1*) *Fastening A*

An unpublished survey of 100 houses in different parts of the country showed that for good practice (not the best practice) it is customary to toenail each rafter to the plate with four 20-penny spikes (two on each side) and toenail it to the ridge with two 10-penny spikes through the upper edge into the ridge board and opposite rafter.

(*2*) *Fastening B*

Another method of fastening is to toenail rafters to plate with two 16-penny common nails and two 10-penny nails, one large and one small nail on each side. The ridge board should be fastened to one rafter by two 8- or 10-penny nails through the board into the rafter. The other rafter should be toenailed with three 8- or 10-penny nails, one in the upper edge and one in each side.

(*3*) *Computed Strength of Fastenings*

For each of these fastenings withdrawal resistance and lateral resistance was computed in accordance with Design Specification for Stress-Grade Lumber [35]. Fastenings *A* are stronger than fastenings *B*, so the utility factors were based on the *B* fastenings. The strengths of the *B* fastenings are

	lb
At ridge, two-nail rafter:	
Withdrawal resistance (anchor)	312.9
Lateral resistance, acting either up or down (key)	231.2
At wall:	
Withdrawal resistance (anchor)	696.0
Lateral resistance, acting in, out, or parallel to wall (key)	806.1

In the specification, the allowable loads for nails are less than one-fifth the load which would be required to separate the pieces completely. This allowable load was doubled to obtain the design loads for these rafter fastenings.

In Miami, for both houses, the anchors at ridge have only about one-third the required strength. For fastenings *A*, also, the allowable load is less than half the design load for the fastening.

In Los Angeles, on the other hand, the fastenings are excessively strong and other fastening at less cost is indicated. Laboratory tests on specimens of a size simulating actual construction and loaded just the way they are in a house, should lead to the development of cheaper and better fastenings, not only for houses of wood but, also for those of other materials.

How thoroughly nailing is done in the field can only be ascertained by constant inspection.

(*c*) *Utility Factor For All Roofs*

For each of the roofs included in the program, the allowable transverse load was computed, as for roof *QC*, from the values in table 24, by including the weight and adjusting for the span in the houses. Also, the design transverse load for each house and location was computed. Finally, the utility factor for each roof was obtained. The values are given in table 28. Roof *QC* is included to facilitate comparison with the other roofs.

TABLE 28.—*Utility factors for roofs*

Symbol	Roof construction	Allowable load		House 22-D			House E		
		Span 14.29 ft	Span 11.32 ft	Los Angeles	Miami	Portland	Los Angeles	Miami	Portland
		lb/ft²	*lb/ft²*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*
BT	Wood frame with plaster	[a]33.1	52.8	3.1	1.4	1.1	4.5	1.9	1.8
	Do	[b]147.3	234.8	13.7	6.4	4.9	20.1	8.6	7.9
DN	Wood frame, no plaster	28.2	44.9	5.4	1.0	1.1	7.1	1.4	1.8
DO	Do	70.0	111.6	12.1	2.5	2.8	16.3	3.5	4.4
DT	Do	125.7	200.3	16.6	4.8	4.7	23.3	6.6	7.5
DU	Do	200.0	318.7	25.2	7.8	7.3	35.6	10.7	11.7
QC	Do	50.5	80.5	11.0	1.7	2.1	14.2	2.4	3.4
AK	Sheet steel, no plaster	138.9	221.4	13.8	5.9	4.7	20.1	7.9	7.6
AO	Do	47.0	74.9	7.0	1.7	1.8	9.7	2.4	2.9
CS	Do	118.9	189.5	10.7	5.3	3.9	15.8	7.1	6.3
CX	Do	138.3	220.4	16.1	5.5	5.0	23.1	7.5	7.9
DJ	Do	26.6	42.3	9.3	0.9	1.2	10.6	1.2	1.9

[a]Allowable load which does not crack the plaster on lower face. See table 26.
[b]Allowable load if cracks in plaster are acceptable.

7. GABLES

(*a*) *Strength*

The gables are wall *QA* although, presumably, it was not intended there should be plaster on the inside face. Taking the allowable transverse loads from table 26 if cracks in plaster are acceptable and adjusting for span (height of gable at ridge), the allowable transverse loads are:

	House 22-D	House E
	lb/ft²	*lb/ft²*
Loaded, inside face	206.6	266.2
Loaded, outside face	110.2	142.0

For louvre in south gable of House E, two studs are cut and the design wind load on this gable is doubled adjacent to louvre. The louvre in north gable of House E and both louvres of House 22-D are inserted between studs and the design wind load on these gables adjacent to louvre is not increased.

The design transverse loads on gables are given in table 29.

TABLE 29.—*Design transverse load on gables*

Gables	Design transverse load					
	Los Angeles		Miami		Portland	
HOUSE 22-D						
South and north:	*lb/ft²*	*U.F.*	*lb/ft²*	*U.F.*	*lb/ft²*	*U.F.*
Loaded, inside face	4.2	49.2	27.9	7.4	10.1	20.5
Loaded, outside face	4.7	23.4	31.3	3.5	11.3	9.8
HOUSE E						
South gable:						
Loaded, inside face	9.4	28.3	61.4	4.3	22.2	12.0
Loaded, outside face	10.8	13.1	71.0	2.0	25.6	5.5
North gable:						
Loaded, inside face	4.7	56.6	30.7	8.7	11.1	24.0
Loaded, outside face	5.4	26.3	35.5	4.0	12.8	11.1

Except for south gable of House E, the total transverse load decreases linearly from the value below ridge to zero at east and west walls. For constant transverse strength of gable from east wall to west wall, the spacing of studs should increase from ridge to wall. Midway between ridge and wall the spacing of studs should be four times the spacing at ridge (16 in.) or 5 ft 4 in.

Except for south gable of House E in Miami, the gables have excessive strength as shown by the utility factors which range up to 56.6 in Los Angeles.

There is no compressive load or racking load on gables.

The reaction upward on bottom of gable vertically below ridge is: House 22-D, 63.9 lb/ft; House E, 56.3 lb/ft. These reactions decrease linearly to zero at east and west walls.

(*b*) *Fastenings*

The design loads on fastenings for gables are given in table 30.

TABLE 30.—*Design loads on fastenings for gables*

[Design loads for fastenings vertically below ridge, which decrease to zero at wall]

Gables	Los Angeles, Acting up	Los Angeles, Acting inward	Los Angeles, Acting outward	Miami, Acting up	Miami, Acting inward	Miami, Acting outward	Portland, Acting up	Portland, Acting inward	Portland, Acting outward
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
HOUSE 22-D									
South and north gables:									
Bearing	63.9	----	----	63.9	----	----	63.9	----	----
Key, acting normal to gable:									
At top	----	12.0	13.4	----	79.7	89.4	----	28.9	32.3
At bottom	----	13.4	15.0	----	89.1	100.0	----	32.3	36.1
HOUSE E									
South and north gables:									
Bearing	56.3	----	----	56.3	----	----	56.3	----	----
Key, acting normal to gable:									
South gable, at top	----	23.0	26.4	----	150.0	173.4	----	54.2	62.5
South gable, at bottom	----	26.5	30.4	----	172.8	199.9	----	62.5	72.1
North gable, at top	----	11.5	13.2	----	75.0	86.7	----	27.1	31.3
North gable, at bottom	----	13.2	15.2	----	86.4	99.9	----	31.2	36.0

The louvre in south gable of House E is 2 ft 6 in. wide. According to the convention adopted for transverse load on walls adjacent to openings, the design transverse load and design load on fastenings are zero for the width of the louvre. The values for south gable apply from edge of louvre outward for 1 ft 3 in. For the remainder of the distance to wall, the design loads for north gable apply.

8. ATTIC FLOOR

(*a*) *House 22-D*

(*1*) *Vertical Loads*

For House 22-D, the attic floor (figs. 50 and 51) has no applied floor load, therefore the design loads for members and fastenings adjacent to scuttle, *jkgf*, and chimney, *us*, are not given here.

The reactions upward exerted by walls and partitions below, due to weight of floor, are

	lb/ft
East wall:	
d'z	60.9
zd	46.9
East partition:	
wq	60.4
qb	101.8
West partition:	
c'v	101.8
vp	54.4
West wall:	
a'n	40.9
na	54.9

These values, also, are the design loads for fastenings (bearing) acting vertically.

(*2*) *Horizontal Loads*

The design loads acting horizontally are

	Los Angeles	Miami	Portland
Tension:			
(East or west wind)	*lb/ft*	*lb/ft*	*lb/ft*
Parallel to joists	63.0	29.3	[a]368.6
Normal to joists			
At wall	15.4	101.7	36.7
Below ridge	28.9	190.9	68.8
Compression:			
(South or north wind)			
Parallel to joists	0.0	194.9	26.2
Beam load:			
(East or west wind)			
Along east or west wall	25.8	170.3	61.4
(South or north wind)			
Along south or north wall			
At wall	17.2	113.2	40.8
Below ridge	50.0	330.0	119.0

[a] Snow both slopes.

The design tensile and compressive loads parallel to joists, are also the design loads for fastenings. These fastenings are keys acting normal to both east and west walls. For tension, the keys act outward and for compression inward.

For any direction of wind, the attic floor is a deep beam under horizontal loads applied to the windward edge, the leeward edge, or to both edges. For east and west wind the floors in both houses are simple beams under uniformly distributed load supported by the racking resistance of south and north walls. For south and north wind, these floors are continuous beams over four supports in House 22-D and three supports in House E. The supports are the east and west walls and the partitions.

For south or north wind, the beam load is the same at east and west walls and increases linearly to a greater value vertically below ridge. If the roof is a symmetrically sloping roof, the greatest load is midway between walls.

Under beam load, the design loads for fastenings are those for roof and gable above and for walls below.

(*b*) *House E*

(*1*) *Vertical Loads*

For House E, the attic floor (figs. 51 and 53) has no applied load if the house is unoccupied (empty) and has a floor load of 20 lb/ft^2 if occupied.

The reactions upward exerted by walls and partitions below are

	Unoccupied	Occupied
	lb/ft	*lb/ft*
East wall, *rd*	53.7	155.1
Partition, *qc*	103.5	298.8
West wall, *oa*	49.8	143.7

The values if occupied are also the design loads for bearing.

(*2*) *Horizontal Loads*

The design loads acting horizontally are

	Los Angeles	Miami	Portland
Tension:			
(East or west wind)	*lb/ft*	*lb/ft*	*lb/ft*
Parallel to joists	44.7	14.0	[a]237.3
Normal to joists			
At wall	18.3	120.8	43.6
Below ridge	31.4	207.3	74.7
Compression:			
(South or north wind)			
Parallel to joists	0.0	104.0	5.5
Beam load:			
(East or west wind)			
Along east or west wall	38.8	255.8	92.3
(South or north wind)			
Along south or north wall			
At wall	22.2	146.6	52.9
Below ridge	54.8	361.4	130.3

[a] Snow both slopes.

(*3*) *Framing Around Openings*

If the house is occupied, there is an applied load of 20 lb/ft^2 on the attic floor and the design loads for the members and fastenings adjacent to chimney and scuttle openings are as follows:

		Design load	Span
Chimney:			*ft*
Floor, *efba*	*lb/ft*^{2}	20.0	6.95
Header, *fb*	*lb/ft*	106.3	2.33
Fastenings:			
Header to joist, *f* and *b*	*lb*	123.8	
Scuttle:			
Floor, *kmih*	*lb/ft*^{2}	20.0	6.28
Floor, *nj*	*lb/ft*^{2}	20.0	0.61
Header, *mi*	*lb/ft*	96.1	2.50
Header,, *nj*	*lb/ft*	9.3	2.50
Fastenings:			
Header to joist, *m* and *i*	*lb*	120.1	
Header to joist, *n* and *j*	*lb*	11.6	

Each of the members *ef* and *ba* has a point-load of 123.8 lb at a distance of 2.44 ft from bearing on partition; each of members *kn* and *jh* has two point-loads, one of 11.6 lb at a distance of 0.61 ft from bearing on partition and the other of 120.1 lb at a distance of 3.11 ft from bearing.

Customarily, in house construction, header *nj* would be the same size as header *mi*.

Probably that part of the point-loads carried by the west wall and by the partition would be distributed along the wall so that the effect of the openings on the wall and partition would be approximately the same as if there were no opening.

9. House 22-D, One Story

(*a*) *Wall*

The design loads at top of wall are obtained by combining the greatest reactions on roof, gable, and attic floor above. On the wall these loads act in the opposite direction.

(*1*) *Compressive Load*

East and west walls.—The design compressive load for east and west wall, House 22-D, also for top edge of floor under these walls, and top of east and west cellar walls are shown in table 31.

TABLE 31.—*Design compressive load for east and west walls, House 22-D*

Element	Los Angeles		Miami		Portland	
	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
East wall:	261.4	0.0	270.6	682.1	891.6	97.7
Top floor	208.2	.0	212.8	357.1	523.3	5.1
Top cellar wall	518.6	.0	523.2	294.7	833.7	0.0
West wall:	249.4	.0	258.6	691.5	879.6	108.8
Top floor	200.0	.0	204.6	360.7	515.1	11.6
Top cellar wall	593.5	.0	598.1	270.7	908.6	0.0

For walls, the values for loads acting downward are design loads in compression and for loads acting upward are design loads in tension.

For top of floor and top of cellar wall, the values for loads acting downward are design loads for bearing and for loads acting upward, they are design loads for anchors.

South and north walls.—The design compressive load for south and north walls, House 22-D, also for top edge of floor under these walls, and top of south and north cellar walls are shown in table 32.

TABLE 32.—*Design compressive load for south and north walls, House 22-D*

Element	Los Angeles		Miami		Portland	
	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
South wall:	113.0	0.0	113.0	0.0	113.0	0.0
Top floor	141.3	.0	277.4	122.6	141.3	.0
Top cellar wall	141.3	.0	309.5	154.7	141.3	.0
North wall:	71.7	.0	71.7	0.0	71.7	.0
Top floor	142.9	.0	292.4	134.4	142.9	.0
Top cellar wall	142.9	.0	325.8	167.8	142.9	.0

The design compressive load for south and north walls is based only on weight of gable above, therefore, it is the same for any location. There never is any design tensile load on south or north wall.

For each location, the design compressive load downward for wall is greatest for east wall.

The allowable compressive load for each wall construction was taken from table 20 and, making no adjustment for the actual story-wall height, the utility factor was computed for the greatest design load acting downward (east wall). The utility factor for each wall construction for *downward* compressive load is given in table 33.

For east and west walls, construction *QA* has sufficient compressive strength for the three locations, but is overdesigned, the factor being greater than ten for Los Angeles and Miami and greater than three for Portland.

All the other wall constructions have adequate compressive strength for House 22-D in the three locations. Except for wall *DK* in Portland, they all are overdesigned; the factor ranging from 2.1 for wall *BY* in Portland to 405.5 for wall *AT* in Los Angeles.

The design compressive loads on south and north walls are a small fraction of the loads on east and west walls. Therefore, they have adequate strength, but the utility factors are very much greater than the values in table 33. For wall *QA* in any location, the factor for south wall is 24.3 and for north wall 38.3.

Insofar as compressive load only is concerned, an economical construction for south and north walls should have a much smaller allowable compressive load than that for east and west walls.

Tensile load on wall.—There are no tensile design loads for south and north walls House 22-D. For Los Angeles, there are none for east and west walls. For Portland, they are about one-eighth the design compressive loads and for Miami, about two and one-half times the design compressive load.

Because no tensile load was applied to walls, the allowable tensile load is not known. It may be estimated by the usual engineering methods. For the wood and steel wall constructions, it seems probable that the allowable tensile load is not less than the allowable compressive load, but for masonry, the tensile properties of walls should be investigated.

Attachment of plate to studs, wall QA.—In wall *QA* (BMS25), two 16-penny common wire nails passed through the lower member of the double top plate into each stud and three 10-penny nails per stud space passed through the upper member of the plate into the lower member.

The sheathing, applied diagonally, was attached to the upper member of top plate by two 8-penny nails per board, 8 inches wide (2.12 nails per foot width of wall). Each board, also, was attached to lower member by one 8-penny nail.

All these nails are part of the wall construction and should not be confused with fastenings between roof and wall. The allowable loads on these attachments are

	lb/ft
(a) Lower member of top plate to studs	122.1
(b) Upper member to lower member	122.9
(c) Sheathing to upper member	135.8
(d) Sheathing to lower member	67.9
Total allowable load, sum (b) and (c)	258.7

The attachments in wall *QA* are ample for Portland, factor 2.4, but very inadequate for Miami, factor 0.4. Even if the allowable load is doubled, which sometimes is considered good practice, these attachments still are inadequate for Miami. Moreover, the load for (a) is computed for the nail penetrating the side grain, not the end grain as in this wall. The Design Specification [35] does not say that nails in the end grain offer no resistance to withdrawal, merely that, "800-D-1. The structural design shall be such that nails are not loaded in withdrawal from end grain of wood."

Presumably the allowable load on these attachments should be (c)+(d), sheathing to members of plate, 203.7 lb/ft, which is adequate for Portland, but not for Miami, factor 0.3.

There would be no difficulty if the plate could be attached to each stud by a four-foot strip of box strapping over the plate, down each edge of stud, and secured by a few strong nails.

(2) *Transverse Load*

The design transverse loads on wall House 22-D are

	Los Angeles	Miami	Portland
Acting on:	*lb/ft²*	*lb/ft²*	*lb/ft²*
Inside face	7.59	50.09	18.06
Outside face	8.22	54.27	19.57

The allowable transverse load for each wall construction was taken from table 20 and, after adjusting for the actual span (story-wall height), the utility factor was computed. The factors are given in table 33.

For load on inside face, wall *QA* has enough transverse strength for the three locations, but not enough for load on outside face in Miami, factor 0.4. For load on inside face, the utility factor for each location is about four times the factor for load on outside face, showing that wall *QA* is an unbalanced design. For Los Angeles and Portland, the factors for load on inside face are excessive.

All wood walls, including wall *QA*, have sufficient transverse strength for Los Angeles. All wood walls, with the exception of wall *DK*, have sufficient transverse strength for Portland. All wood walls, except *BK*, *BL*, *BQ*, *BX*, and *QA* which had plaster on the inside faces and walls *CB*, *DK*, *DL*, *DQ*, and *DR*, which had no plaster have the necessary transverse strength for Miami.

All the wood walls are greatly overdesigned for Los Angeles, wall *CN* having a factor of 15.0. Except for walls *DK*, *DL*, *DQ*, and *QA*, all the wood walls are somewhat overdesigned for Portland.

All the steel walls have enough transverse strength, except *DG* and *DH* for Miami. Some are overdesigned. The factor for wall *CU* for Los Angeles is 28.2.

All the reinforced brick and concrete walls have sufficient transverse strength for each location. The factor is 32.1 for wall *BV* in Los Angeles.

Masonry walls *BW*, *CZ*, and *DA* have adequate transverse strength for the three locations, but other masonry walls tested are deficient in this respect, particularly for Miami.

Some engineers who have had wide experience with small houses think many of these masonry walls have withstood satisfactorily the wind loads in most parts of this country. Several explanations have been suggested for this apparent discrepancy between the transverse strength found in the laboratory and that found in service.

First: The transverse strength of masonry walls may be greater if the load is uniformly distributed than if it is applied at the quarter points of the span.

Second: The transverse strength is greater, the greater the compressive load on wall. There was no compressive load on wall when the transverse load was applied in the laboratory.

For wood and steel walls, the transverse strength does not depend greatly upon the compressive load on wall.

Third: Due to the sheltering effect of surrounding fences, trees, and buildings, the velocity pressure of the wind is much less than the values for midheight of wall assumed in this report.

A satisfactory explanation of this question can only be determined by further investigation.

Transverse fastenings.—The design loads for fastenings are the same for top and bottom of wall. They are keys acting normal to wall. The design loads for transverse fastenings are

	Los Angeles	Miami	Portland
Fastenings:	*lb/ft*	*lb/ft*	*lb/ft*
Acting inward	15.5	102.3	36.9
Acting outward	16.8	110.8	40.0

(3) *Racking Load*

The design racking loads for House 22-D with wall *QA* and load-bearing partition *QD* are

	Los Angeles		Miami		Portland	
South wall:	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*
House wall	23.4	21.4	154.6	3.2	55.8	9.0
Cellar wall	29.2	...	192.4	...	69.4	...
East wall:						
House wall	13.3	37.6	87.9	5.7	31.7	15.8
Cellar wall	20.9	...	137.6	...	49.6	...
North wall:						
House wall	21.4	23.4	141.2	3.5	50.9	9.8
Cellar wall	30.2	...	199.5	...	71.9	...
West wall:						
House wall	13.2	37.9	87.1	5.7	31.4	15.9
Cellar wall	21.0	...	138.4	...	49.9	...

The design racking load for house wall is also the design load for keys at top and bottom of wall. The value for cellar wall is the design load for keys. These keys act along the wall in either direction.

The design racking load depends upon the racking modulus, not only of the wall considered, but also the modulus of each of the other walls and the load-bearing partitions.

The design racking loads for this house with walls of other constructions were not computed, therefore, the utility factors could not be obtained. It is thought that if the computations had been made for the other constructions the design racking load would not exceed 200 lb/ft.

TABLE 33.—*Utility factors for walls*

[L.A. = Los Angeles, Calif. M. = Miami, Fla. P. = Portland, Maine]

Construction symbol	House 22-D, one-story									House E, two-story																	
	First story									Second story									First story								
	Compressive load			Transverse load						Compressive load			Transverse load						Compressive load			Transverse load					
				Acting on inside face			Acting on outside face						Acting on inside face			Acting on outside face						Acting on inside face			Acting on outside face		
	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.	L.A.	M.	P.
													WOOD														
	U.F.	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*
BG	22.3	21.6	6.6	8.3	1.3	3.5	7.7	1.2	3.2	13.2	10.7	6.3	6.8	1.0	2.8	6.0	0.9	2.5	5.4	4.4	3.7	8.4	1.2	3.4	7.6	1.1	3.2
BH	11.5	11.1	3.4	13.3	2.0	5.6	11.8	1.8	5.0	6.8	5.5	3.2	10.8	1.6	4.5	9.3	1.4	3.9	2.8	2.3	1.9	13.4	2.0	5.5	11.6	1.8	4.9
BI	9.6	9.2	2.8	11.1	1.7	4.7	11.8	1.8	5.0	5.6	4.6	2.7	9.0	1.4	3.8	9.3	1.4	3.9	2.3	1.9	1.6	11.2	1.7	4.6	11.6	1.8	4.9
BJ	13.4	12.9	3.9	10.8	1.6	4.5	7.2	1.1	3.0	7.9	6.4	3.8	8.7	1.3	3.7	5.6	0.9	2.4	5.2	2.7	2.2	10.9	1.6	4.4	7.1	1.1	3.0
BK	12.6	12.2	3.7	5.6	0.8	2.3	6.6	1.0	2.8	7.4	6.1	3.6	4.5	0.7	1.9	5.2	.8	2.2	2.8	2.5	2.1	5.6	0.8	2.3	6.5	1.0	2.7
BL	14.2	13.7	4.2	8.3	1.3	3.5	6.1	0.9	2.6	8.4	6.8	4.0	6.8	1.0	2.8	4.8	.7	2.0	3.4	2.8	2.4	8.4	1.2	3.4	6.0	0.9	2.5
BQ	15.3	14.8	4.5	10.0	1.5	4.2	5.1	.8	2.2	9.0	7.4	4.3	8.1	1.2	3.4	4.0	.6	1.7	3.7	3.0	2.6	10.1	1.5	4.1	5.0	.8	2.1
BU	11.1	10.7	3.3	8.9	1.3	3.7	9.2	1.4	3.9	6.5	5.3	3.1	7.2	1.1	3.0	7.3	1.1	3.0	2.7	2.2	1.9	9.0	1.3	3.7	9.1	1.4	3.8
BX	18.2	17.6	5.3	10.0	1.5	4.2	5.1	0.8	2.2	10.8	8.8	5.1	8.1	1.2	3.4	4.0	0.6	1.7	4.4	3.6	3.1	10.1	1.5	4.1	5.0	0.8	2.1
BY	7.3	7.0	2.1	11.8	1.8	4.9	10.2	1.6	4.3	4.3	3.5	2.1	9.5	1.4	4.0	8.1	1.2	3.4	1.8	1.4	1.2	11.9	1.8	4.9	10.1	1.5	4.2
CB	13.4	12.9	3.9	5.6	0.8	2.3	5.1	0.8	2.2	7.9	6.4	3.8	4.5	0.7	1.9	4.0	0.6	1.7	3.2	2.7	2.2	5.6	0.8	2.3	5.0	0.8	2.1
CI	9.7	9.3	2.8	12.8	1.9	5.4	11.1	1.7	4.7	5.7	4.7	2.7	10.4	1.6	4.4	8.7	1.3	3.7	2.3	1.9	1.6	12.9	1.9	5.3	10.9	1.6	4.6
CM	7.7	7.4	2.2	11.1	1.7	4.7	12.3	1.9	5.2	4.5	3.7	2.2	9.0	1.4	3.8	9.7	1.5	4.1	1.9	1.5	1.3	11.2	1.7	4.6	12.1	1.8	5.1
CN	11.3	10.9	3.3	15.0	2.3	6.3	12.8	1.9	5.4	6.7	5.4	3.2	12.2	1.8	5.1	10.1	1.5	4.2	2.7	2.2	1.9	15.1	2.2	6.2	12.6	1.9	5.3
DK	6.2	6.0	1.8	2.2	0.3	0.9	2.0	0.3	0.9	3.7	3.0	1.7	1.8	0.3	0.8	1.6	0.2	0.7	1.5	1.2	1.0	2.2	0.3	0.9	2.0	0.3	0.8
DL	8.3	8.0	2.4	4.4	.7	1.9	7.1	1.1	3.0	4.9	4.0	2.3	3.6	.5	1.5	5.6	.9	2.4	2.0	1.6	1.4	4.5	.7	1.8	7.1	1.1	3.0
DQ	12.5	12.1	3.7	5.6	.8	2.3	4.6	0.7	1.9	7.4	6.0	3.5	4.5	.7	1.9	3.6	.5	1.5	3.0	2.5	2.1	5.6	.8	2.3	4.5	0.7	1.9
DR	25.3	24.4	7.4	6.1	.9	2.6	5.6	.9	2.4	14.9	12.2	7.1	5.0	.8	2.1	4.4	.7	1.7	6.1	5.0	4.2	6.2	.9	2.5	5.5	.8	2.3
QA	10.5	10.2	3.1	10.7	1.6	4.5	2.7	.4	1.1	6.2	5.1	3.0	8.6	1.3	3.6	2.1	.3	0.9	2.5	2.1	1.8	10.8	1.6	4.4	2.6	.4	1.1
													STEEL														
AH	22.0	21.2	6.4	18.4	2.8	7.7	17.0	2.6	7.1	13.0	10.6	6.2	14.9	2.3	6.3	13.4	2.0	5.6	5.3	4.4	3.7	18.6	2.7	7.6	16.7	2.5	7.0
AL	18.1	17.5	5.3	12.0	1.8	5.0	10.2	1.6	4.3	10.7	8.7	5.1	9.7	1.5	4.1	8.1	1.2	3.4	4.4	3.6	3.0	12.1	1.8	4.9	10.1	1.5	4.2
AP	----	----	----	7.9	1.2	3.3	8.4	1.3	3.5	----	----	----	6.4	1.0	2.7	6.6	1.0	2.8	----	----	----	8.0	1.2	3.3	8.3	1.3	3.5
AV	43.2	41.8	12.7	22.2	3.4	9.3	22.5	3.4	9.5	25.5	20.8	12.2	18.0	2.7	7.6	17.7	2.7	7.5	10.5	8.6	7.2	22.4	3.3	9.2	22.2	3.4	9.3
AZ	24.8	23.9	7.3	14.0	2.1	5.9	11.3	1.7	4.7	14.6	11.9	7.0	11.3	1.7	4.8	8.9	1.3	3.7	6.0	4.9	4.1	14.1	2.1	5.8	11.1	1.7	4.7
CQ	8.8	8.5	2.6	13.3	2.0	5.6	11.3	1.7	4.7	5.2	4.2	2.5	10.8	1.6	4.5	8.9	1.3	3.7	2.1	1.7	1.5	13.4	2.0	5.5	11.1	1.7	4.7
CU	17.2	16.6	5.0	28.2	4.3	11.9	20.7	3.1	8.7	10.2	8.3	4.9	22.9	3.5	9.6	16.3	2.5	6.8	4.2	3.4	2.9	28.5	4.2	11.6	20.4	3.1	8.6
DG	13.7	13.2	4.0	6.7	1.0	2.8	5.3	0.8	2.2	8.1	6.6	3.9	5.4	0.8	2.3	4.2	0.6	1.8	3.3	2.7	2.3	6.7	1.0	2.7	5.2	0.8	2.2
DH	13.1	12.6	3.8	7.7	1.2	3.2	6.0	.9	2.5	7.7	6.3	3.7	6.2	.9	2.6	4.8	.7	2.0	3.2	2.6	2.2	7.7	1.1	3.2	5.9	.9	2.5
													REINFORCED BRICK AND CONCRETE														
AR	206.6	199.6	60.6	9.1	1.4	3.8	24.6	3.7	10.3	121.9	99.4	58.3	7.4	1.1	3.1	19.3	2.9	8.1	50.0	41.1	34.5	9.2	1.4	3.8	24.2	3.7	10.2
AT	405.5	391.7	118.9	11.1	1.7	4.7	10.2	1.6	4.3	239.2	195.1	114.4	9.0	1.4	3.8	8.1	1.2	3.4	98.1	80.6	67.8	11.2	1.7	4.6	10.1	1.5	4.2
BV	68.9	66.5	20.2	32.1	4.9	13.5	17.4	2.6	7.3	40.6	33.1	19.4	26.0	3.9	10.9	13.7	2.1	5.8	16.7	13.7	11.5	32.4	4.8	13.2	17.1	2.6	7.2

TABLE 33.—*Utility factors for walls*—*Continued*

[L.A. = Los Angeles, Calif. M. = Miami, Fla. P. = Portland, Maine]

MASONRY

AA	381.0	368.1	111.7	5.1	0.8	2.1	4.7	0.7	2.0	224.8	183.3	107.5	4.1	0.6	1.7	3.7	0.6	1.6	92.2	75.5	63.7	5.2	0.8	2.1	4.6	0.7	1.9
AB	80.3	77.6	23.6	1.7	.3	0.7	1.5	.2	0.6	47.4	38.7	22.7	1.4	.2	0.6	1.2	.2	0.5	19.4	16.0	13.4	1.7	.2	0.7	1.5	.2	0.6
AC	138.5	133.8	40.6	3.6	.5	1.5	3.3	.5	1.4	81.7	66.6	39.1	2.9	.4	1.2	2.6	.4	1.1	33.5	27.5	23.2	3.6	.5	1.5	3.2	.5	1.4
AD	68.9	66.5	20.2	1.6	.2	0.7	1.4	.2	0.6	40.6	33.1	19.4	1.3	.2	0.5	1.1	.2	0.5	16.7	13.7	11.5	1.6	.2	0.6	1.4	.2	0.6
AE	36.3	35.1	10.7	2.6	.4	1.1	2.4	.4	1.0	21.4	17.5	10.3	2.1	.3	.9	1.9	.3	.8	8.8	7.2	6.1	2.6	.4	1.1	2.3	.4	1.0
AF	58.5	56.5	17.2	1.3	.2	0.6	1.2	.2	0.5	34.5	28.2	16.5	1.1	.2	.5	1.0	.1	.4	14.2	11.6	9.8	1.3	.2	0.5	1.2	.2	0.5
AU	40.6	39.2	11.9	0.8	.1	.3	1.0	.2	.4	23.9	19.5	11.4	0.6	.1	.3	0.8	.1	.3	9.8	8.1	6.8	0.8	.1	.3	1.0	.2	.4
AX	55.1	53.2	16.2	2.1	.3	.9	1.9	.3	.8	32.5	26.5	15.5	1.7	.3	.7	1.5	.2	.6	13.3	10.9	9.2	2.1	.3	.9	1.9	.3	.8
BD	a94.5 b73.8	91.3 71.3	27.7 21.6	1.0	.2	.4	0.9	.1	.4	a55.7 b43.6	45.5 35.5	26.7 20.8	0.8	.1	.3	0.7	.1	.3	a22.9 b17.9	18.8 14.7	15.8 12.3	1.0	.1	.4	0.9	.1	.4
BE	64.7	62.5	19.0	0.9	.1	.4	0.7	.1	.3	38.1	31.1	18.2	.7	.1	.3	.6	.1	.2	15.6	12.8	10.8	0.9	.1	.4	.7	.1	.3
BF	52.0	50.3	15.3	2.0	.3	.8	1.6	.2	.7	30.7	25.0	14.7	1.6	.2	.7	1.3	.2	.5	12.6	10.3	8.7	2.0	.3	.8	1.6	.2	.7
BO	49.7	48.0	14.6	2.4	.4	1.0	1.4	.2	.6	29.3	23.9	14.0	2.0	.3	.8	1.1	.2	.5	12.0	9.9	8.3	2.5	.4	1.0	1.4	.2	.6
BP	29.8	28.8	8.7	0.3	.1	0.1	0.3	.1	.1	17.6	14.4	8.4	0.3	.1	.1	0.2	.1	.1	7.2	5.9	5.0	0.3	.1	0.1	0.3	.1	.1
BW	30.2	29.2	8.9	8.5	1.3	3.6	7.9	1.2	3.3	17.8	14.5	8.5	6.9	1.1	2.9	6.2	.9	2.6	7.3	6.0	5.1	8.6	1.3	3.5	7.8	1.2	3.3
CF	59.7	57.6	17.5	1.1	0.2	0.5	1.0	0.2	0.4	35.2	28.7	16.8	0.9	0.1	0.4	0.8	.1	0.3	14.4	11.9	10.0	1.1	0.2	0.5	1.0	0.2	0.4
CG	55.1	53.2	16.2	1.1	.2	.5	1.0	.2	.4	32.5	26.5	15.5	.9	.1	.4	.8	.1	.3	13.3	10.9	9.2	1.1	.2	.5	1.0	.2	.4
CH	9.9	9.6	2.9	2.0	.3	.8	3.8	.6	1.6	5.9	4.8	2.8	1.6	.2	.7	3.0	.5	1.3	2.4	2.0	1.7	2.0	.3	.8	1.8	.3	.8
CP	89.9	86.8	26.4	3.1	.5	1.3	2.9	.4	1.2	53.0	43.3	25.4	2.5	0.4	1.1	2.3	.3	0.9	21.8	17.9	15.0	3.1	.5	1.3	2.8	.4	1.2
CZ	166.8	161.1	48.9	11.2	1.7	4.7	10.4	1.6	4.3	98.4	80.3	47.1	9.1	1.4	3.8	8.1	1.2	3.4	40.4	33.2	27.9	11.3	1.7	4.6	10.2	1.5	4.3
DA	228.0	220.3	66.8	9.5	1.4	4.0	8.8	1.3	3.7	134.5	109.7	64.3	7.7	1.2	3.3	6.9	1.1	2.9	55.2	45.3	38.1	9.6	1.4	3.9	8.7	1.3	3.6
DB	20.7	20.0	6.1	2.6	0.4	1.1	2.4	0.4	1.0	12.2	9.9	5.8	2.1	0.3	0.9	1.9	0.3	0.8	5.0	4.1	3.5	2.6	0.4	1.1	2.3	0.4	1.0
DC	16.8	16.3	4.9	3.0	.5	1.3	2.8	.4	1.2	9.9	8.1	4.7	2.4	.4	1.0	2.2	.3	.9	4.1	3.3	2.8	3.0	.4	1.2	2.7	.4	1.1
DD	148.4	143.4	43.5	2.9	.4	1.2	2.7	.4	1.1	87.6	71.4	41.9	2.3	.4	1.0	2.1	.3	.9	35.9	29.5	24.8	2.9	.4	1.2	2.6	.4	1.1
DE	165.3	159.6	48.5	4.4	.7	1.9	4.1	.6	1.7	97.5	79.5	46.6	3.6	.5	1.5	3.2	.5	1.4	40.0	32.8	27.6	4.5	.7	1.8	4.0	.6	1.7
DF	18.7	18.1	5.5	2.2	.3	0.9	2.0	.3	0.9	11.1	9.0	5.3	1.8	.3	0.8	1.6	.2	0.7	4.5	3.7	3.1	2.2	.3	0.9	2.0	.3	0.8
DP	60.8	58.8	17.8	1.0	.2	4	0.9	.1	.4	35.9	29.3	17.2	0.8	.1	.3	0.7	.1	.3	14.7	12.1	10.2	1.0	.1	.4	0.9	.1	.4

a Cavity wall, loaded on facing and backing.
b Cavity wall, load centered on backing only.

Except wood walls *CB* and *DL*, all the other wall constructions have this strength, some considerably more.

(*b*) *Partition*

(*1*) *Compressive Load*

For House 22-D in any location, the design compressive load for both load-bearing partitions is 203.5 lb/ft acting downward. There never is a tensile load on these partitions.

All the load-bearing partitions in the program are wood frame. The allowable compressive loads are given in table 21. The utility factors are, partition *CJ*, gypsum-board faces, 7.4; *CO*, fiberboard faces, 16.2; *QD*, plaster faces, 4.7. All these partitions have ample strength but are overdesigned.

The design compressive loads (bearing) exerted by the two load-bearing partitions are

Partition:	Los Angeles	Miami	Portland
	lb/ft	*lb/ft*	*lb/ft*
East	198.8	273.3	217.1
West	210.2	300.4	232.4

These partitions never exert upward loads.

The east partition exerts these loads on south-north wood beam and the west partition exerts them on floor below.

The east-west nonload-bearing partitions exert design compressive loads on wood beam and on east and west cellar walls. These are point loads and are the same for any location.

On top of wood beam and east cellar wall there are two point loads, each 650.1 lb, one at 12 ft 6 in. from south wall and the other at 21 ft 5 in.

On top of wood beam and west cellar wall there are three point loads: 635.2 lb at 8 ft 7 in. from north wall, 713.0 lb at 14 ft 0 in., and 621. 2 lb at 20 ft 7 in.

(*2*) *Racking Load*

The design racking loads and utility factors for partition *QD* in House 22-D (one-story) are

Partition:	Los Angeles		Miami		Portland	
	lb/ft	*U.F.*	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*
East	8.9	42.7	58.6	6.5	21.1	18.0
West	8.9	42.7	58.9	6.5	21.2	17.9

Partition *QD* has ample racking strength for House 22-D in any location but it wastes material.

The allowable racking loads for the other partitions are *CJ*, 480.0 and *CO*, 470.0 lb/ft. As for the walls, it would be necessary to compute the design racking loads to obtain the utility factors. Probably they have sufficient racking strength but are overdesigned.

Racking fastenings.—The values for design racking load of partition are also the design loads for fastenings at top and bottom of partition.

(*c*) *Floor*

If required, the horizontal loads in the plane of floor may be computed as illustrated for attic floor. These loads are not given here.

(*1*) *Transverse Load*

When determining the transverse load on floor, no account was taken of the weight of the additional nonload-bearing partitions in walls of closets.

The weight of east-west nonload-bearing partitions is transferred to east and west cellar walls and to south-north wood beam. These partitions are not loads on floor because they are parallel to the joists in floor.

If the weight of equipment in kitchen, utility room, and bath exceeds 40 lb/ft^2, the excess is disregarded. In designing a house, these weights should be included when computing the design transverse load on floor.

There are two south-north load-bearing partitions. The east partition is supported directly by the south-north wood beam. The west partition exerts a line load on west floor normal to joists in addition to the surface load. The design transverse loads on portions of floor shown in figure 51 are

Portion of floor	Load	Span
	lb/ft^2	*ft*
yzdc	40.0	11.67
imba	40.0	13.67
rumi	49.81	13.67
vwur	53.64	13.67

The design transverse load on span 12 ft for all floor constructions was obtained from table 23 and the allowable load for the two spans in this house computed.

Four of the floors have plaster on the lower face; *AY* and *BC* are sheet steel, and *BS* and *QB* are wood frame. For these floors, the allowable load is taken from table 26 as value if cracks are permissible. The utility factors may be taken from table 34.

Floor *QB* has ample strength for each of the floors but is stronger than necessary for the east floor, *yzdc*.

If floor *QB* is satisfactory for west floor, *vwur*, with the partition and its load from attic floor, the question arises why a floor construction requiring less material would not be acceptable for east floor, *yzdc*, having no partition load and on a shorter span? Certainly the cost of the floor is a considerable portion of the cost of a structure such as this house.

TABLE 34.—*Utility factors for floors*

[The letters for portions of floor are shown on figure 51 for both houses]

	House 22-D, one-story				House E, two-story									
	First floor				Second floor				First floor					
Portion of floor	*yzdc*, east floor	*imba*	*rumi*	*vwur*, west floor	*mnkj*	*tusr*	*nodc*	*hica*	*vwqn*	*uvnm*	*fgba*	*strq*	*jneb*	*tupo*
Span (ft)	11.67	13.67	13.67	13.67	3.36	6.86	8.70	9.39	13.36	13.36	5.34	7.34	8.58	13.25
Construction symbol	WOOD													
	U.F.	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*	*U.F.*
BS	6.5	4.7	3.8	3.5	15.0	3.3	2.0	1.6	0.6	0.6	32.5	17.0	12.4	5.0
CE	3.3	2.3	1.9	1.7	41.8	9.9	6.1	5.2	2.5	2.2	16.4	8.6	6.3	2.5
CL	5.1	3.7	2.9	2.7	63.3	15.1	9.3	8.0	3.8	3.4	25.0	13.1	9.6	3.9
DM	2.6	1.8	1.5	1.4	32.7	7.7	4.7	4.1	1.9	1.7	12.8	6.7	4.9	2.0
DS	2.7	1.9	1.5	1.4	34.3	8.1	4.9	4.2	2.0	1.8	13.4	7.0	5.1	2.0
QB	3.9	2.7	2.2	2.0	13.1	2.9	1.7	1.4	0.5	0.5	19.6	10.2	7.4	2.9
	STEEL													
AG	10.1	7.0	5.6	5.2	137.2	31.9	19.3	16.4	7.4	6.6	53.5	27.7	19.9	7.5
AJ	5.2	3.7	3.0	2.8	65.0	15.4	9.5	8.1	3.9	3.5	25.6	13.4	9.8	4.0
AN	3.7	2.7	2.1	2.0	47.3	11.2	6.9	5.9	2.8	2.5	18.6	9.8	7.1	2.9
AY	6.6	4.7	3.8	3.5	33.4	7.7	4.6	3.9	1.7	1.5	33.2	17.4	12.6	5.1
BB	2.9	2.0	1.6	1.5	36.7	8.7	5.3	4.5	2.1	1.9	14.4	7.5	5.5	2.2
BC	2.8	1.9	1.6	1.4	29.7	6.8	4.1	3.5	1.5	1.4	14.7	7.6	5.5	2.1
CR	7.5	5.4	4.3	4.0	92.8	22.1	13.6	11.7	5.6	5.0	36.6	19.3	14.0	5.7
CW	5.6	4.0	3.2	3.0	69.7	16.5	10.2	8.7	4.2	3.7	27.4	14.4	10.5	4.3
	REINFORCED CONCRETE													
AS	1.6	0.9	0.8	0.7	30.2	6.5	3.7	3.0	1.0	0.9	11.4	5.6	3.8	1.1
AW	8.7	6.0	4.8	4.5	119.1	27.6	16.7	14.1	6.3	5.6	46.4	23.9	17.2	6.4
CT	6.5	4.4	3.6	3.3	90.8	20.9	12.6	10.7	4.7	4.2	35.3	18.2	13.0	4.8
CY	6.6	4.5	3.6	3.4	89.2	20.7	12.5	10.6	4.8	4.3	34.8	18.0	12.9	4.9

The architect made some provision for this condition by specifying 2- by 8-in. joists for both floors but spacing them 16 inches in east floor and 12 inches in west floor.

Of the other wood floors, all will safely support the occupants, but floor *BS* and *CL* are somewhat overdesigned for west floor. All the wood floors have excessive strength for east floor, the factor ranging from 2.6 for *DM* to 6.5 for *BS*.

The steel floors all have strength enough and floors *AN*, *BB*, and *BC* have strength to spare. For east floor, the factor ranges from 2.8 for *BC* to 10.1 for *AG*.

Reinforced concrete floor, *AS*, is the only floor that lacks the strength required for all floors. It has the strength for the east floor but not for the other floors. The greatest factor is 8.7 for east floor, *AW*.

In the ground floor of this house there are two openings, as shown on figure 51, "First floor," one for chimney (*stpo*) and the other for access to cellar (*jkgf*). The design loads for members adjacent to these openings are

Floor ijfe.—Load along *ei* and *fj*, 206.2 lb/ft.
Header *fj*, load 206.2 lb/ft, span 2.0 ft.
Fastenings at *f* and *j*, 206.2 lb.

Floor kmhg.—Load along *gk* and *hm*, 117.6 lb/ft.
Header *gk*, load 117.6 lb/ft, span 2.0 ft.
Fastenings at *g* and *k*, 117.6 lb.

Beams eh and im.—Two point loads, 206.2 lb at 8.0 ft. from west wall and 117.6 lb at 9.5 ft, span 13.67 ft.

Floor rson.—Load along *nr* and *os*, 228.2 lb/ft.
Header *os*, load 228.2 lb/ft, span 1.5 ft.
Fastenings at *o* and *s*, 171.2 lb.

Floor tuqp.—Load along *pt* and *qu*, 95.5 lb/ft.
Header *pt*, load 95.5 lb/ft from floor and 175.7 lb/ft from partition, total 271.2 lb/ft, span 1.5 ft.
Fastenings at *p* and *t*, 203.4 lb.

Beams nq and ru.—Two point loads, 171.2 lb at 8.83 ft from west wall and 203.4 lb at 10.33 ft, span 13.67 ft.

Where these members bear on west cellar wall and on wood beam they exert point loads acting downward.

Opposite openings, the load exerted by floor on wall and beam are redistributed locally between uniform and point loads without changing the magnitude appreciably. It does not seem necessary to consider the point loads due to openings when designing cellar walls and beams.

The south-north wood beam, 12 ft 0 in. from east wall, is supported (as four simple beams) on south and north cellar walls and on three posts in cellar. Post *A* is 7 ft 9 in. from south wall, post *B* is 16 ft 9 in., and post *C* is 25 ft 9 in. The design compressive (point) loads on these walls and posts are

	Los Angeles	Miami	Portland
	lb	*lb*	*lb*
South wall	3206.3	3423.7	3259.9
Post A	7371.2	7591.4	7425.4
Post B	9031.7	9062.3	9039.2
Post C	8146.5	8376.5	8203.2
North wall	3347.0	3585.1	3405.7

There never are loads acting upward on these supports. Neither is there any racking load.

This ends the evaluation and discussion of design loads on the elements of House 22-D in Los Angeles, Miami, and Portland.

10. House E, Second Story

Much of the explanation and discussion for House 22-D also applies to House E. This information is not repeated here.

(*a*) *Wall*

(*1*) *Compressive Load*

East and west walls.—The design compressive load for east and west walls, second story of House E are

	Los Angeles		Miami		Portland	
	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
East wall	443.1	0.0	543.3	612.8	926.5	78.7
West wall	420.1	.0	520.3	624.2	903.5	87.9

The design reactions at bottom of east and west walls are

	Los Angeles		Miami		Portland	
	Acting upward (compression)	Acting downward (tension)	Acting upward (compression)	Acting downward (tension)	Acting upward (compression)	Acting downward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
East wall	301.8	0.0	351.9	427.9	543.5	31.9
West wall	290.3	.0	340.4	429.9	532.0	18.1

These values are not the design loads for fastenings because the wall is continuous past the second floor. They are the design compressive and tensile loads in wall, particularly the studs, just above the ribbon supporting the second floor.

South and north walls.—The design compressive loads for south and north walls are

	Los Angeles		Miami		Portland	
	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
South wall	80.0	0.0	80.0	0.0	80.0	0.0
North wall	65.8	.0	65.8	.0	65.8	.0

The design reactions at bottom of south and north walls are

	Los Angeles		Miami		Portland	
	Acting upward (compression)	Acting downward (tension)	Acting upward (compression)	Acting downward (tension)	Acting upward (compression)	Acting downward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
South wall	136.1	0.0	439.3	279.6	209.5	49.8
North wall	136.1	.0	439.3	279.6	209.5	49.8

These are design compressive and tensile loads in wall above floor bearing.

The design compressive load on south and north walls is the same for any location. It is about one-sixth the design compressive load for east and west walls in Los Angeles and only one-twelfth the value in Portland. Certainly, a less expensive construction for south and north walls is indicated.

The east wall has the greatest design compressive load.

The utility factors for the greatest load acting *downward* are presented in table 33.

For wall *QA*, the factor ranges from 3.0 in Portland to 6.2 in Los Angeles showing the construction is safe but much stronger than is desirable.

All the walls of other constructions will carry the compressive load but many of them have much greater compressive strength than is required for the second story of this house. The factors show the trend:

Type of wall	Smallest utility factor	Wall	Largest utility factor	Wall
Wood	1.7	*DK*	14.9	*DR*
Sheet steel	2.5	*CQ*	25.5	*AV*
Reinforced brick and concrete	19.4	*BV*	239.2	*AT*
Masonry	2.8	*CH*	224.8	*AA*

All the smallest factors are for Portland and the largest for Los Angeles.

(*2*) *Transverse Load*

For the second story walls of House E, the design transverse loads are

	Los Angeles	Miami	Portland
Acting on:	lb/ft^2	lb/ft^2	lb/ft^2
Inside face	9.0	59.4	21.4
Outside face	10.1	66.4	23.9

The utility factors for transverse load may be taken from table 33.

Wall *QA* has the necessary transverse strength for each location if loaded on the inside face. If loaded on the outside face, the strength is enough for Los Angeles but not enough for Portland. For Miami, the strength is about one-third the required value.

Of the 19 wood walls, six do not have the necessary transverse strength if loaded on the inside face and twelve if loaded on the outside face.

The greatest factor is for wall *CN*, value 12.2.

All the steel walls have enough transverse strength except walls *DG* and *DH*. For wall *CU*, the factor is 22.9.

None of the reinforced brick and concrete walls are deficient in transverse strength. Wall *BV* has the greatest factor, 26.0.

Walls *CZ* and *DA* are the only masonry constructions complying with the requirements for transverse strength.

Transverse fastenings.—The design loads for transverse fastenings are

	Los Angeles	Miami	Portland
Fastenings:	*lb/ft*	*lb/ft*	*lb/ft*
Acting inward	18.7	123.7	44.6
Acting outward	20.9	138.2	49.9

(3) *Racking Load*

For second story of House E, with wall *QA* and partition *QD*, the design racking loads for walls are

	Los Angeles		Miami		Portland	
House wall:	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*
South wall	29.4	17.0	194.4	2.6	70.1	7.1
East wall	17.1	29.2	112.9	4.4	40.7	12.3
North wall	29.4	17.0	194.4	2.6	70.1	7.1
West wall	16.9	29.6	111.7	4.5	40.3	12.4

These values are also design loads for keys.

If the design racking loads for the other walls were computed and the greatest value does not exceed 300 lb/ft, all the walls have sufficient racking strength except wood walls *CB* and *DL*, steel wall *DH*, and masonry wall *CH*.

(b) *Partition*

(1) *Compressive Load*

For second story of House E in any location, the design compressive load for the one load-bearing partition is 597.6 lb/ft acting downward. The utility factors are, partition *CJ* 2.5, *CO* 5.5, and *QD* 1.6.

Partition *QD* has the necessary strength and is economical for House E. The other partitions are also adequate as to strength but are uneconomical.

To determine the vertical loads at bottom of partition, the south-north load-bearing partition in second story is taken as two independent partitions. The south portion of the partition extends from south wall of house to east-west wood beam 13 ft 8 in. from south wall. The north portion of the partition extends from wood beam to north wall of house.

Both partitions exert point loads acting downward on house walls and wood beam. These loads on first story are:

	Los Angeles	Miami	Portland
South partition:			
South wall, 9 ft 8 in. from west edge	*lb* 2801.9	*lb* 3236.4	*lb* 2909.0
Wood beam	2801.9	3236.4	2909.0
North partition:			
North wall, 10 ft 5 in. from east edge	3009.0	3264.8	3072.0
Wood beam	3009.0	3264.8	3072.0

(2) *Racking Load*

The design racking loads and utility factors for second-story partition *QD* of House E are

	Los Angeles		Miami		Portland	
	lb/ft	*U.F.*	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*
Partition	11.4	33.3	75.2	5.1	27.1	14.0

(c) *Floor*

(1) *Transverse Load*

The design transverse loads on portions of the second floor of House E shown in figure 51 are

Portion of floor	Load	Span
	lb/ft²	*ft*
mnkj	40.0	3.36
tusr	40.0	6.86
nodc	40.0	8.70
hica	40.0	9.39
vwqn	40.0	13.36
uvnm	44.98	13.36

The portion *uvnm* has a line load exerted by nonload-bearing partition normal to floor joists in addition to the surface load.

For the floor constructions having plaster on the lower face, cracks are not permissible, therefore, the allowable loads on a span of 12 ft are taken from table 23. For each floor construction, the allowable load was computed for each of the five spans in this second floor.

If the floors were tested on the 3.36- and 6.86-ft spans, it appears probable the floors would fail by shear under transverse loads less than these computed values. However, no results of tests are available.

The utility factors are available in table 34.

Floor *QB* has half the transverse strength required for portion *vwqn* and *uvnm* which have the longest span. Also, floor *BS* has about half the necessary strength.

All the other wood floors meet the strength requirements. Floors *DM* and *DS* are economical in the use of material but for floor *CL*, the factor is 3.8.

On the shorter spans, all the wood floors have excessive strength, the factor for floor *CL* being 63.3.

All the steel floors have enough transverse strength. Floors *AY*, *BB*, and *BC* are economical for the longest spans. On the shortest span, floor *AG* has a factor of 137.2.

Except reinforced concrete floor *AS*, all the floors in this group have sufficient transverse strength but are uneconomical on the shortest span. The factor for floor *AW* is 119.1.

11. House E, First Story

(*a*) *Wall*

(*1*) *Compressive Load*

The design compressive loads for first-story walls, also for top of wall, top edge of first-story floor under walls, and top of cellar walls are given in table 35.

The values for top of first-story wall are the design compressive and tensile loads in the wall just below the ribbon supporting the second floor.

The utility factors for each wall construction appear in table 33.

Wall *QA* has all the compressive strength required for the first story and is economically designed for Miami and Portland.

Table 35.—*Design compressive load for first-story walls of House E*

Element	Los Angeles		Miami		Portland	
	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)	Acting downward (compression)	Acting upward (tension)
	lb/ft	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*	*lb/ft*
South wall:	1015.4	0.0	1315.2	234.3	1099.0	4.5
Top wall	507.7	.0	794.7	234.3	564.8	4.5
Top floor	654.9	.0	1477.6	771.7	857.6	151.7
Top cellar wall	1018.2	.0	1901.8	795.0	1236.0	129.2
East wall:	603.6	.0	703.7	682.9	1086.9	31.9
Top wall	301.8	.0	351.9	427.9	543.5	31.9
Top floor	379.5	.0	508.1	776.7	621.2	90.9
Top cellar wall	379.5	.0	558.2	862.8	621.2	109.0
North wall:	732.0	.0	1142.7	325.8	833.7	49.8
Top wall	367.6	.0	670.7	279.6	440.9	49.8
Top floor	530.6	.0	1326.2	780.6	726.7	181.0
Top cellar wall	769.2	.0	1623.2	814.1	979.7	170.6
West wall:	1080.1	.0	1180.3	682.9	1563.5	18.1
Top wall	540.1	.0	590.2	429.9	781.8	18.1
Top floor	614.9	.0	665.0	787.9	856.6	116.4
Top cellar wall	614.9	.0	665.0	838.6	856.6	134.7

All the other wood walls have adequate compressive strength. In general, the wood walls are not very greatly overdesigned. The greatest factor is 6.1 for wall *DR* in Los Angeles.

Steel walls have all the compressive strength desired. Like the wood walls, they are not greatly overdesigned. The factor for wall *AV* is 10.5 in Los Angeles.

Reinforced brick and concrete walls, although adequate as to compressive strength, tend to be overdesigned. Wall *AT* has a factor of 98.1.

The masonry walls have ample compressive strength. The factors range from 1.7 for wall *CH* in Portland to 92.2 for *AA* in Los Angeles. Wall *CH* consists of flanged, reinforced-concrete slabs fastened to vertical steel plates with steel bolts. There is an air space in the wall. Wall *AA* is high-strength brick laid in cement mortar with excellent workmanship. It is a solid wall.

In addition to the distributed loads, there are point loads on top of first-story walls exerted by partitions and floor joists in second story.

On south wall at 9 ft 8 in. from west edge, there is a design point load of 2801.9 lb for Los Angeles, 3236.4 lb for Miami, and 2909.0 lb for Portland, all acting downward.

On north wall at 10 ft 5 in. from east edge, there is a design point load of 3009.0 lb for Los Angeles, 3264.8 lb for Miami, and 3072.0 lb for Portland, all acting downward.

Other point loads which are the same for any location are: 423.7 lb on south wall at 3 ft 8 in. from west edge, 3186.0 lb on east wall at 13 ft 8 in. from south edge, 752.2 lb on west wall at 5 ft 6 in. from north edge, and also 472.5 lb on the same wall at 15 ft 7 in. from north edge. All these loads act downward.

Under all these point loads, there are additional compressive members (posts) in wall extending from top of first-story wall through first floor to top of cellar wall.

(*2*) *Transverse Load*

The design transverse loads on first-story walls of House E are

	Los Angeles	Miami	Portland
Acting on:	*lb/ft²*	*lb/ft²*	*lb/ft²*
Inside face	7.9	52.2	18.8
Outside face	8.5	56.4	20.3

The utility factor for each wall construction is listed in table 33.

Wall *QA* is strong enough under transverse loading except in Miami when loaded on outside face.

Of the other wood walls *BK*, *BL*, *BQ*, *BX*, *CB*, *DK*, *DL*, *DQ*, and *DR* lack the strength required for Miami. Wall *DK* is the only wall lacking enough transverse strength for Portland. The factor is 13.4 for wall *BH* in Los Angeles.

Each of the steel walls complies with the transverse strength requirements except wall

DG (factor 0.8) and wall *DH* (factor 0.9) when loaded on outside face in Miami.

The reinforced brick and concrete walls have ample transverse strength, the factors ranging from 1.4 for *AR* to 32.4 for *BV*.

Walls *BW*, *CZ*, and *DA* are the only masonry walls which have the necessary transverse strength and they are well designed from the standpoint of efficient use of material because the factor for each is less than two.

The design loads for transverse fastenings are

	Los Angeles	Miami	Portland
Fastenings:	*lb/ft*	*lb/ft*	*lb/ft*
Acting inward	16.0	105.4	38.0
Acting outward	17.3	114.0	41.1

(3) *Racking Load*

For the first story of House E, with wall *QA* and load-bearing partition *QD*, the design racking loads are:

	Los Angeles		Miami		Portland	
	lb/ft	*U.F.*	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*
South wall:						
House wall	53.0	9.4	349.9	1.4	126.2	4.0
Cellar wall	54.5	...	360.0	...	129.8	...
East wall:						
House wall	39.1	12.8	258.2	1.9	93.1	5.4
Cellar wall	43.1	...	284.4	...	102.6	...
North wall:						
House wall	52.0	9.6	343.5	1.5	123.9	4.0
Cellar wall	55.2	...	364.4	...	131.4	...
West wall:						
House wall	39.5	12.7	260.9	1.9	94.1	5.3
Cellar wall	42.8	...	282.2	...	101.8	...

For Miami, wall *QA* is adequately and economically constructed for the first-story walls of House E. For Portland, the racking strength is five times the necessary strength, and for Los Angeles, ten times.

(b) *Partition*

(1) *Compressive Load*

For the first story of House E, the design compressive load on east-west load-bearing partition is 1386.8 lb/ft acting downward.

The allowable compressive load on partition *QD* is 960 lb/ft. The utility factor is 0.7 and this partition is not strong enough.

For partition *CJ*, the factor is 1.1 and for *CO*, 2.4, which indicates that these partitions will carry the compressive load safely. In partition *CJ*, the material is efficiently used and there is not much excess material in *CO*.

On the east-west load-bearing partition at 10 ft 5 in. from east wall, there is a design point load on the top of partition of 5443.5 lb for Los Angeles, 5622.1 lb for Miami, and 5487.5 lb for Portland. At the intersection of east-west and south-north partitions 9 ft 0 in. from east wall, there is a design point load of 3186.0 lb and at the edge of stair opening 16 ft 5 in. from east wall, the design point load is 811.4 lb. These point loads are the same for any location.

Under all these point loads there are additional compressive members (posts) in the partition extending from top of east-west partition to wood beam below.

On the south-north load-bearing partition, first story of House E, there is no design compressive load.

(2) *Racking Load*

In House E, which has walls *QA* and load-bearing partitions *QD*, the design racking loads for load-bearing partitions in first story are

	Los Angeles		Miami		Portland	
Partition:	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*	*lb/ft*	*U.F.*
East-west	35.1	10.8	231.8	1.6	83.6	4.5
South-north	26.3	14.4	173.8	2.2	62.7	6.1

In Miami, partition *QD* has ample racking strength for both east-west and south-north first-story partitions. For House E in Portland, the strength is about five times the required racking strength and in Los Angeles it is twelve times.

(c) *Floor*

(1) *Transverse Load*

For the first floor of House E in any location, the design transverse loads on portions of floor shown in figure 51 are

Portion of floor	Load	Span
	lb/ft²	*ft*
fgba	40.0	5.34
strq	40.0	7.34
jneb	40.0	8.58
tupo	40.0	13.25

For the utility factors, turn to table 34.

All the floor constructions have the transverse strength for the first floor of House E. The smallest factor is 1.1 for floor *AS*, (reinforced concrete) on 13.25-ft span and the largest is 53.5 for *AG* (steel) on 5.34-ft span.

Floor *QB* has twice the required transverse strength on the longest span and 19.6 times on the shortest span. Factors for other wood floors run from 2.0 to 32.5, indicating excessive material, particularly on the shorter spans.

For the steel and concrete floors, the overdesign is more than for wood floors. Factors are 2.1 to 53.5 for steel and 1.1 to 46.4 for concrete.

On top of cellar wall, in addition to the point loads from top of first-story walls, there are point loads from partitions and floor joists in first story.

Point loads which are the same for any location are 476.3 lb on south cellar wall 3 ft 8 in. from west edge, 2678.7 lb on east wall 13 ft 8 in. from south edge, 384.8 lb on north wall at 16 ft 5 in. from east edge, and 820.4 lb and 364.4 lb on west wall at 5 ft 8 in. and 15 ft 0 in. from north edge. These loads all act downward.

On north wall (cellar), there is a design point load at 9 ft 0 in. from east edge, the magnitude of which depends upon the location. The values are

	Los Angeles	Miami	Portland
	lb	*lb*	*lb*
Acting downward (bearing)	577.4	1517.7	809.2
Acting upward (anchor)	0.0	698.7	0.0

The east-west wood beam under first floor 13 ft 8 in. from south wall is supported as two simple beams on east cellar wall and on two posts in cellar. Post *D* is 9 ft 0 in. from east wall directly below intersection of the partitions in first story. Post *E* is 16 ft 4 in. from east wall at edge of stair opening.

The load on east cellar wall is given above. It is the same value whatever the location of the house.

The design point loads on posts are

	Los Angeles	Miami	Portland
	lb	*lb*	*lb*
Post *D*	15520.5	16605.2	15787.8
Post *E*	6298.8	6333.0	6307.2

This ends the evaluation and discussion of design loads on elements of House E in Los Angeles, Calif.; Miami, Fla.; and Portland, Maine.

XX. CONCLUSIONS

1. Wind Map

The wind map, showing the greatest velocity pressure at each location, is an engineering evaluation of Weather Bureau data. Further study may show that changes are desirable but, if the velocity pressures are increased greatly for regions subject to severe storms, the strength and consequently the cost of conventional houses will have to be increased over present practice.

2. Velocity Pressure vs. Wind Load

The relation between velocity pressure and wind load on roof and walls of a house are taken directly from the Final Report of Subcommittee 31, American Society of Civil Engineers [7]. This report gives the most reliable data available at present.

There is need for research on the relation of observed velocity pressure to measured wind load on walls and roof of typical houses.

3. Snow Map

The snow map, showing the greatest weight of snow at each location, is an engineering deduction from Weather Bureau data.

Changes may be desirable but, if the values for regions of heavy snowfall are increased greatly, the strength of the usual house must be increased. Experience seems to indicate that many houses of conventional construction have enough strength for the actual snow loads.

Research is needed to determine with reasonable accuracy the relation between the observed depth of snow on the ground and the measured snow load on roofs of different slopes on both heated and unheated houses.

4. Engineering Principles

The assumptions on how load is transmitted from one element of a house to another element follow closely those for designing large steel-frame buildings. In the future, perhaps these assumptions will be replaced by others more acceptable to engineers and building officials.

The application of an assumption is in accordance with the well-established principles of engineering mechanics, especially the essential principle of static equilibrium.

5. Design Load

Design loads for each element of a house are very close estimates of the loads which it is probable will be applied to the element during the life of the house. The design loads for each element of the two typical houses for which plans are given can be compared with the loads computed by the usual method. The design loads given in this report are believed to be less than the loads obtained by the usual methods.

6. Typical Design Load

The design load of each element of a typical one-story and a typical two-story house are given for three locations selected as representative of extreme wind and snow loads in the United States.

7. Fastenings

The design loads include the design loads for fastenings which in many houses appear far from adequate. If more secure fastenings were customary (at little increase in cost) fewer houses would be damaged or wrecked by severe storms.

8. Tests for Strength

Methods for applying loads in the laboratory to house elements, simulating service loads, are described in BMS2. Tests on 100 house constructions have been reported in the Building Materials and Structures series. The American Society for Testing Materials is preparing standards for testing building assemblies including the elements of houses. In future, we may expect these methods to be used by the housing industry when developing constructions to have adequate strength at reasonable cost. Undoubtedly, building officials will require tests of new and unusual constructions.

9. Allowable Load

A very important question is the service load which should be permitted on a given construction. At present, it is customary to compute the strength of the principal load-carrying members in the construction by the accepted methods for structural design. This approach is not entirely satisfactory for house constructions because tests show that the strength of the completed wall or roof is at times much greater than the strength of the structural members.

For many of the newer constructions, such as those of sheet metal and plywood, there are no generally accepted methods for computing the strength. There is, however, no difficulty in loading them in the laboratory and determining the strength.

10. Loads for 100 Constructions

Allowable load for 100 house constructions are given in this report. Experience may indicate that some should be decreased and others increased.

If the proposed allowable loads for conventional constructions are arbitrarily decreased very much, the strength of the house, particularly in regions subject to severe storms, would appear to be inadequate. However, many conventional constructions have withstood successfully the loads in these regions.

11. Allowable Load vs. Design Load

The allowable load for each construction is compared with the design loads for the two houses in the three locations. For some constructions, the allowable load is much less than the design load, showing that the house is unsafe; for other constructions, it is far too great, indicating an unnecessary waste of material and useless additional expense.

12. New Approach to House Design

General adoption of the approach to the design of houses for strength discussed in this publication, with such modifications as experience shows are necessary, should result not only in greater safety for occupants but through economical use of materials should also result in savings in cost of construction.

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