

STRUCTURAL FRAMING OF LARGE FIELD HOUSES

by

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## STRUCTURAL FRAMING OF LARGE FIELD HOUSES

### INTRODUCTION

The increase in number of spectators and participants in sportings events and indoor exhibitions has created a demand for coliseums, gymnasiums, field houses, and show arenas of unprecedented size. The spectators must have a clear view and an unobstructed floor area is necessary for the particular activities in progress.

During the infancy of sporting events, when spectators were few in number, the necessary unobstructed area could be easily secured by a conventional column and roof truss type of construction. As the popularity of the events increased, so did the attendance, until today it is not uncommon for twelve to fifteen thousand people to view the different events. Of course each person insists upon and is entitled to a clear vision of the entire floor area.

These demands for large clear span structures were increased by participants in outdoor activities. The activity of these outdoor groups was often discontinued because of disagreeable weather. The logical solution to these weather uncertainties was to build a structure large enough to accommodate the various events. This building would need to be large enough to house a football field, baseball diamond, track, livestock show arenas and many other programs.

Determining the size of a field house is perhaps the most

arbitrary and intangible aspect of the design stage. Here an engineer must shed his technical robe and become an economist, compromiser and a diplomat. Each of the different interests in the project will want the field house built to fit his particular needs. The ideal thing would be to have a building large enough to allow simultaneous participation in all of the sports. This could not be economically accomplished because even if such a plan was possible engineering wise, the cost would be prohibitive. At this point compromises would have to be carried out in order to arrive at an arrangement which would prove to be economical and functional.

In the case of a field house for indoor practise, the size of a football field and length of track will pretty well set the standards for the length and width. The height would be determined by requirements of baseball. The majority of other outdoor sports could be fitted into these maximum dimensions.

The field houses or coliseums which house activities drawing gate receipts that support the structure, must undergo very close scrutiny to certify a proper balance between capacity and attendance. In this situation the cost per seat is usually a good index on which to base the expenditure. A large amount of seats could be installed, thereby increasing total cost but reducing cost per seat. If this large number of seats is seldom or never filled, the total cost must then be readjusted over the majority of filled seats. This readjustment would cause the previously sound venture to turn

into a white elephant.

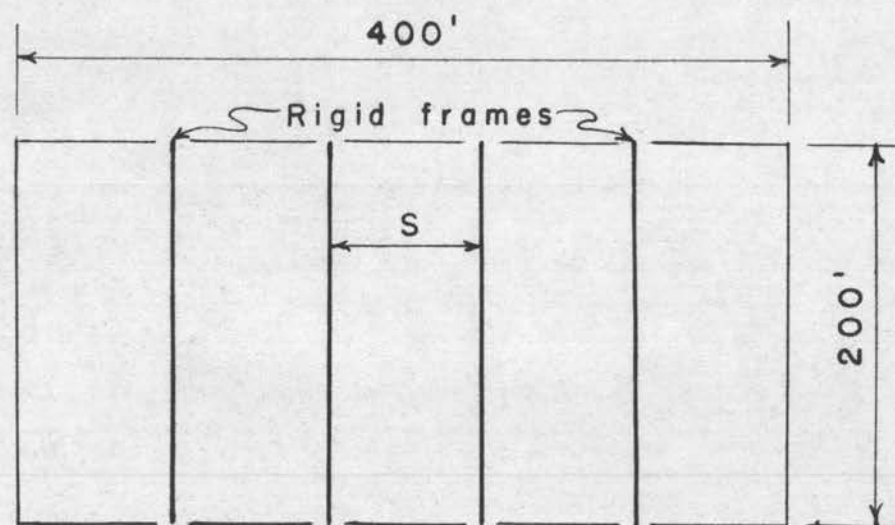
The field house containing the practice areas for outdoor sports has little income. The basis for determining the required size would be the maximum benefits derived by the most number of groups using the building.

The selection of the size of a typical structure mentioned in this paper is based on the above criteria. Aid in determining the necessary size was solicited from the Directors of Athletics at colleges having large field houses on their campuses. A comparison of their contributions indicated close agreement as to the necessary size. As the directors are very cost conscious, an economical or beneficial balance is assured. The size selected for this typical field house is shown in Figure 1.

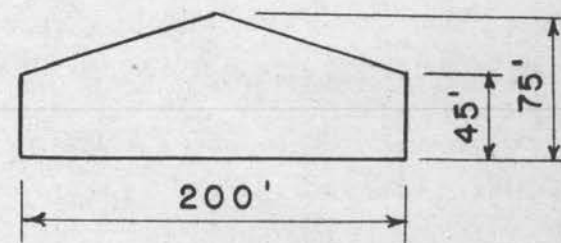
In most cases the dimensions of a field house require some type of clear span structure. The types of structures that offer this large clear span vary considerably. A list of the more prominent ones are concrete arches, thin-shell domes, tied two hinged rigid frames, fixed rigid frames, two and three hinged arches and fixed arches.

Although not predominating, the tied two hinge steel rigid frame or fixed steel rigid frame is very frequently used in the long clear span structures.

The choice between a tied two hinged rigid frame and a fixed rigid frame is largely dependent upon the foundation conditions. In the tied two hinged frame the horizontal thrust at the base is



PLAN



END ELEVATION

FIGURE 1. DIMENSIONS OF FIELD HOUSE



absorbed by a steel bar connecting the two bases. This leaves only vertical loads to be carried to the footing. A fixed rigid frame would require the footing and the soil to resist the vertical reaction, horizontal reaction and the moment that would be transferred to the footing by the high degree of fixity. Unless the soil has extraordinarily strong properties, the tied two hinged frame would be used.

The tied two hinged rigid frame was chosen as the basic structure because of its popularity in field house design and to allow a study of the effects of varied spacing between frames.

An investigation of numerous large clear span structures reveals a surprising relationship between the frames and the accompanying purlins. In many buildings, including Gill Coliseum, the huge rigid frames are spaced at twenty feet on centers. The purlins, ten inch wide flange beams, transferring the roof loads to the frames are dwarfed by the monstrous rigid frames.

Pictures of Gill Coliseum in Figures 2a, 2b, and 2c show the comparative sizes of the rigid frames and the wide flange purlins. These same pictures also indicate the comparatively short distance between the rigid frames.

A seemingly unmentioned question which arises from this observation is why not increase the depth and strength of the purlins and increase the spacing and strength of the rigid frames. This arrangement might eliminate some of the huge rigid frames.

The general opinion, without thought, would be that if the



Figure 2a. Rigid Frames and Purlins of Gill Coliseum.



Figure 2b. Knee Section of Rigid Frames of Gill Coliseum.



Figure 2c. Arrangement of Rigid Frames of Gill Coliseum.



spacing was increased, the loads increase and the dimensions and weight of the frame would increase accordingly. The net effect would show no savings in steel. In considering the question of frame spacing, it should be recalled that the weight of a beam increases with depth, but the resistive strength of a beam to moment increases approximately with the depth squared. The influence of the latter two factors has created a curiosity concerning the original question that seemed justifiable of an investigation.

An increase in spacing with the necessary increase in purlin framing would tend to approach, with limitations, the principles followed in bridge design which require a balance between substructure and superstructure.

There are many factors affecting the spacing of the rigid frame, some of which are foundation conditions, depth of frame, architectural requirements, materials available, and the engineer's pre-conceived ideas or convention. Should the excessive cost of an assumed spacing be due to an engineer's failure to properly investigate alternative spacings by merely adhering to convention, it is truly a setback for the profession.

Although recognizing the many facets of the spacing problem, this writer's purpose is to study the effects of greater spacing on rigid frames and to determine which spacing would involve the least weight of steel per structure. These results would then be modified to meet the various conditions imposed by the other

factors. The ideal spacing as controlled by the roof loading, perhaps can never be realized, but at least a compromise might be reached, whereby the spacing could be increased to a more economical value.

## UTILIZATION OF THE RIGID FRAME

An important advantage in employing the rigid frame in a field house structure is that the required height at the center and at the eaves can be obtained while the area for the base is held to a minimum. Other types of structures require much larger areas at the base to obtain the same height at the eaves.

This advantage gained by use of the rigid frame inadvertently causes a disadvantage. The rigid frame must take a shape, other than parabolic, which will cause the thrust line for approximately uniform load to produce moment.

The rigid frame which does not follow the line of thrust will be stressed in bending by its own dead load in addition to the loads applied to it by the roof system.

Some field houses, used mainly for basketball, have many seats permanently installed. To enable one to speak collectively of the basketball coliseum and an indoor practice field house, the seating would have to be framed independently of the rigid frames.

Upon observation of the interior of Gill Coliseum as shown in the picture in Figure 3, it will be noticed that 90% of the seats face transverse to the direction of the frames. These seats are supported by concrete framing which carries the load to concrete columns under the balcony and in the end wall. These columns transfer the load to the footings and at no time come in contact with the rigid frame. The remaining 10% of the seats are in the



Figure 3. Girder Section of Rigid Frames of Gill Coliseum.



end zone and are supported from below, independent of the rigid frames. The arrangement of the columns adjacent to the rigid frame as shown in Figure 4 is typical in this structure and indicates that the seat load is carried by these columns. The absence of a connection between the columns and rigid frame can be verified by placing a stick between the rigid frame and the wall. This type of support for the seating facilities is not typical, but is used considerably.

A structure of the height, length and width necessary to house a football field would offer a considerable resistance to wind. Because of the increase of 33% in allowable stresses in steel when resisting wind loads, the design of the rigid frame is still controlled by vertical loading.

The vertical section of the frame above the seat level must in most cases support the wall laterally spanning between the rigid frames. The spacing of the rigid frames, depending upon the material used to span this distance, may be limited to a smaller amount than recommended by the vertical loading. The scope of this thesis is to determine the most economical spacing as controlled by the vertical load. There are numerous other factors involved which would alter the recommended spacing. To attempt to include each of the factors in one paper would constitute a project beyond the intent of this thesis. It is felt that if the modern construction methods have provided means to allow increased spacing for vertical loading, these same means could be used to support the walls laterally against wind loads.



Figure 4. Column Section of Rigid Frame of Gill Coliseum.

## LOADS ON THE RIGID FRAMES

The loads carried by the rigid frame are those caused by the live load on the roof, the dead load of the roof system, the dead load of the purlins, and the dead load of the rigid frame.

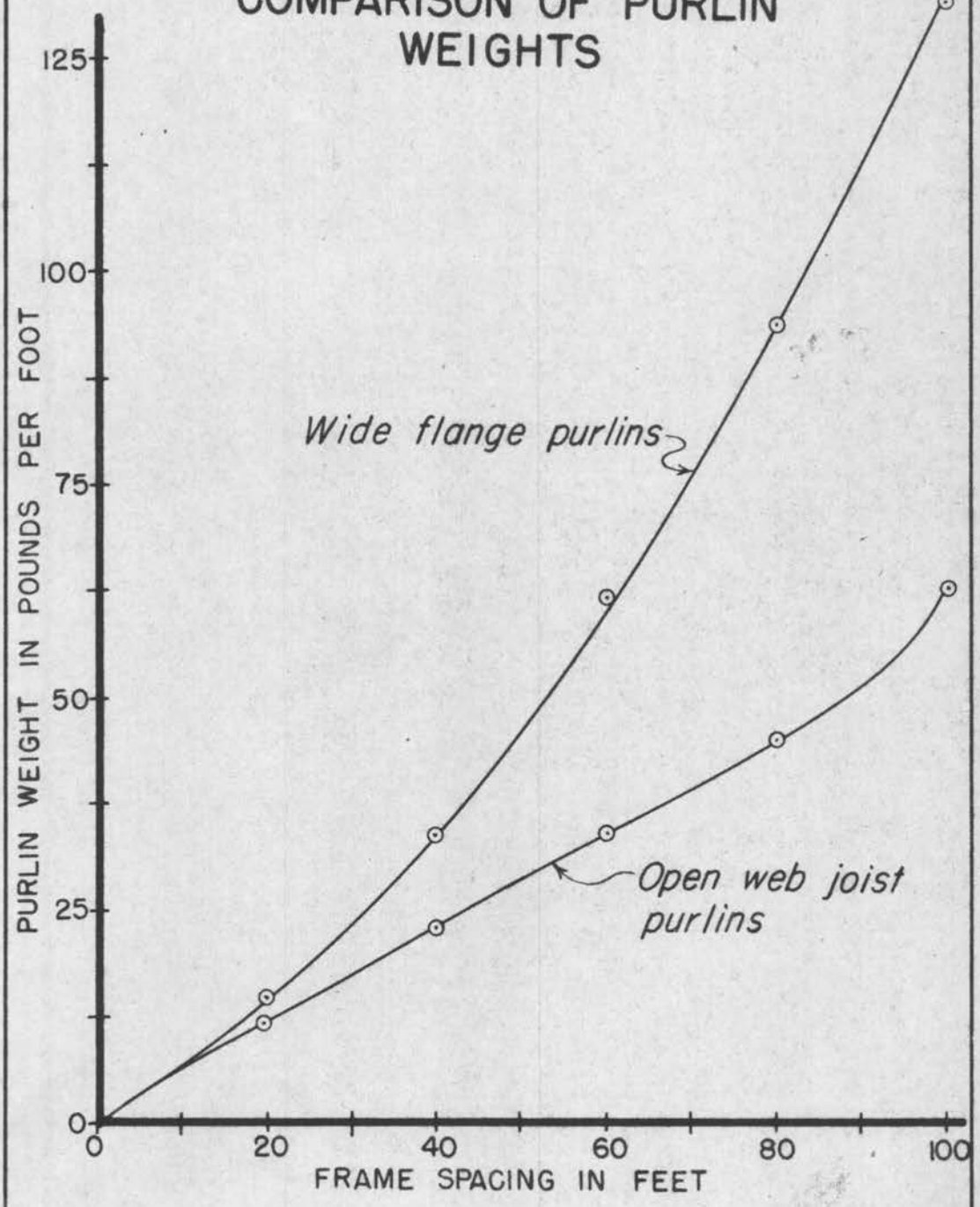
The first two mentioned loads can be considered constant for any one spacing. They vary directly with a change in spacing.

The latter two loads do not vary directly with the spacing and can be of different weights for any one spacing, dependent upon the design. The effect upon the purlins when the span is lengthened has probably greatly retarded the expansion of bay lengths. In studying the curves of Figure 5, it is seen that for any appreciable increase of the span of the wide flange purlin its weight becomes prohibitive. Thus if any weight of steel might be saved in the rigid frames by increased spacing, it probably would be lost in the purlins. The values for the wide flange purlins were obtained from the American Institute of Steel Construction's steel handbook. The loads used were those which would occur upon the typical field house used in this paper.

Perhaps the occasion which has contributed much to solving the problems of increasing the bay lengths is the introduction and availability of welded prefabricated open web joists. These joists, which are prefabricated trusses, are very suitable replacements for the wide flange beams, because of their strength and light weight.

Further study of the curves shows that for an increase in span of open web joists their weights increase directly on a linear basis.

FIGURE 5  
COMPARISON OF PURLIN  
WEIGHTS





For the open web joist curve, it is noticed that the increase in weight deviates from the straight line for spans greater than 80 feet. Above 80 foot span lengths of the purlins or spacing of the rigid frames would not be recommended because of the increased weight of purlins.

The above discussion supports the use of open web joists and also indicates a reason for limiting the spacing of frames if designed when only wide flange beams were available.

The prefabricated joists will be used in the typical field house designed as a basis for comparison.

The improvement of construction practices in general and in welding methods specifically has reduced the dead load of the rigid frame. Because of the length of clear span of the frame a rolled section large enough could not be obtained. It then becomes necessary to use a built-up rigid frame. This frame can be fabricated by riveting or welding. The choice of a welded rigid frame will achieve a savings in weight of about 15%.

This reduction in weight has been acquired by the following changes:

1. The full gross area of the tension flange is available while rivet holes must be deducted from tension flange in a riveted structure.
2. One-sixth of the web may be considered as a part of the flange area in a welded structure, while one-eighth is allowed when using rivets.

3. Effective depth is greater in a welded girder than in a riveted girder.
4. Stiffener plates instead of angles are used.

Perhaps when writing of dead loads on the structure, the bracing should be discussed. True, if bracing were left on a structure its weight would have to be figured in the design load. However, as a rule the bracing in most conventional roof systems is for erection purposes. The bracing forms a measuring device in that if the bracing fits, the frame must be in position. After erection bracing remains in place as a matter of general principle and a large percentage of it in some completed structures takes very little load.

The design of the bracing is based upon an allowable  $l/r$  ratio and is so designed without attributing any known loads to the bracing.

On many of the welded jobs, bracing as necessary could be tack welded in place, then when erection is completed, it could be burned off and salvaged. Bolts could also be used to accomplish this temporary connection. This removal of bracing would lighten the dead load before the occurrence of much live load. If necessary to leave part of the bracing intact, the use of welded connection would decrease the weight to some extent.

Acknowledging the savings in dead weight of purlins and rigid frames, the proposal to lengthen the spacing seems closer to reality.

## DESIGN OF THE RIGID FRAME

The initial step in designing a rigid frame is to assume a frame of the proper proportions to support the vertical load. It should be pointed out that the wind force upon a frame is not critical because of an increase of allowable stress in steel of 33% when resisting wind loads.

The critical points of design in the rigid frame, based upon test data, are the points of tangency of a circular curved haunched knee. These critical points, F and G in Figure 6, actually divide the rigid frame into three separate parts. These parts are the girder or rafter section, the column section, and the knee.

The column and girder sections are checked by design procedures normally employed in the design of any built-up column or girder for combined stresses. The results of the check will determine the necessary adjustments of the assumed proportions.

Data from tests conducted upon knees of rigid frames indicate that the knee section may be designed in accordance with conventional practices, providing certain precautions regarding its shape are observed. Because of the preliminary nature of this design, it was deemed unnecessary to expend time computing the stresses in the knee. The critical points used to determine the stress in the column and girder sections are also the critical points in the design of the haunched knee. The size of the knee is decided by connecting the column section to the girder section with a circular arc. This shape of knee passes the specifications regarding



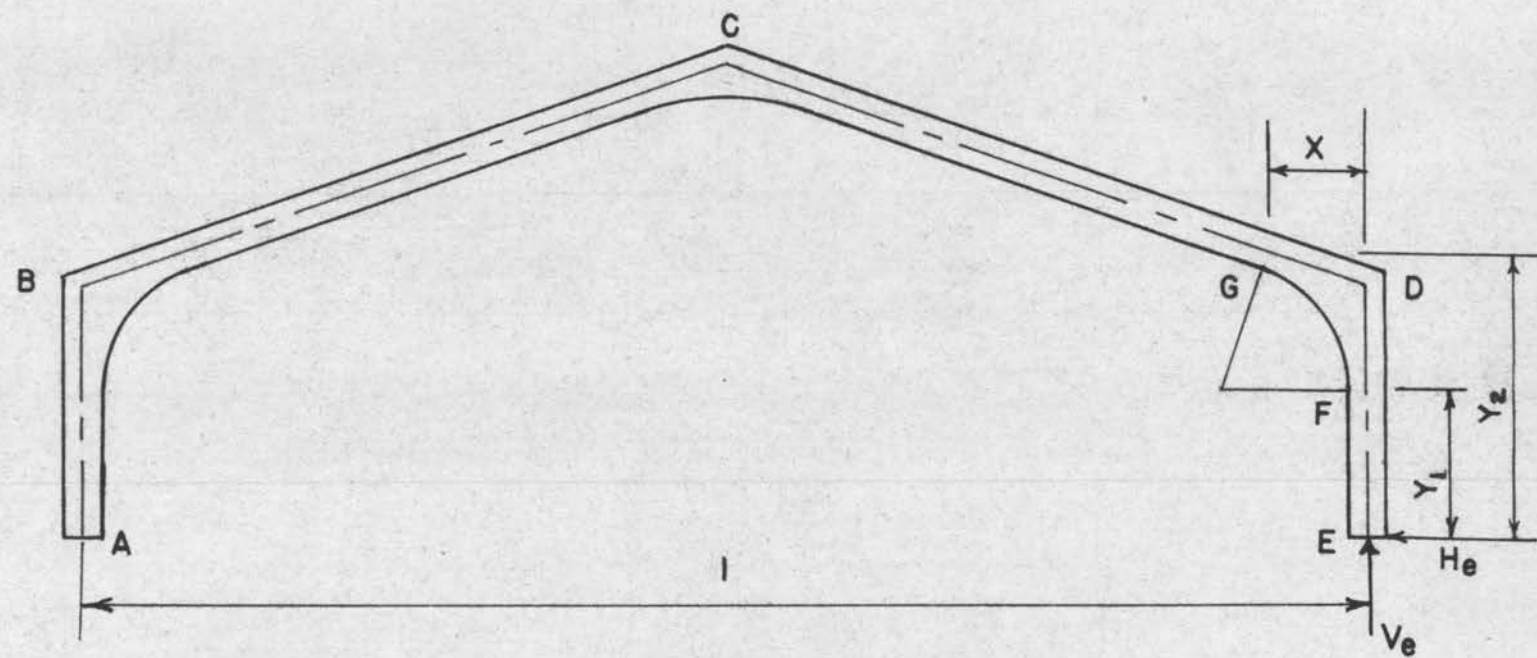


FIGURE 6. RIGID FRAME

relationships between the radius of the haunch, the depth, width and thickness of the column or girder section. "Single Span Rigid Frames in Steel" by John D. Griffiths (2, pp.10-11) outlines the recommended controls of the proportioning and design of the knee.

The bending moments at the assumed critical points can be determined in accordance with the principles of statics, if the horizontal thrust at the base is known. John D. Griffiths' "Single Span Rigid Frames in Steel" (2, pp.14-17) provides a group of charts which makes available a quick means of obtaining the horizontal reactions for a wide range of column heights, roof rise, span and loading conditions. The degree of accuracy of the horizontal reactions computed from the curves, as checked by an exact method, is adequate for design purposes. An example problem indicating the use of the charts is included in the appendix.

The exact values of the moments of inertia of the column and girder section can be determined only after the adequate sections have been established. Since a ratio of the moments of inertia is vital to the approximate design method, an assumption of 1 for this ratio will facilitate design procedures.

Assuming this ratio equal to 1 causes so little error, percentage wise, in the distribution of moments and the determination of horizontal reaction that it can be neglected. A table showing the amount of error involved is shown in the book, "Single Span Rigid Frames in Steel" (2, pp.6-7).

With the horizontal reactions known, the determination of the



stress at the critical points is merely a problem involving the principles of statics.

The moment at the critical point in the column, which produces the controlling stress, is equal to the reaction  $H$  times the lever arm  $Y_1$  as shown in Figure 6. The column must resist a combined stress, therefore the stress in the column is equal to  $\frac{Mc}{I} \pm \frac{P}{A}$ .

The moment in the girder section is equal to the horizontal reaction  $H$  times its lever arm  $Y_2$  minus the conventional beam moment,  $\frac{wlx}{2} - \frac{wx^2}{2}$ . Because the girder is inclined, it must resist a combined stress equal to  $\frac{Mc}{I} \pm \frac{P}{A}$ .

After determining the critical stresses, the column and girder can be proportioned by any normal procedure for design of built-up girders and columns subject to combined stresses.

The loads causing these stresses can be grouped into two categories: loads above the frame and those of the frame itself. Those loads above the frame, including the live load on roof and dead load of roof and purlins, vary practically on a linear basis with spacing as shown in Figure 7.

The dead weight of the frame varies on a non-linear basis as shown in Figure 8. Therefore the curve for total load falls below a linear variation with spacing and the weight of the rigid frame becomes the principal variable with spacing.

FIGURE 7

**RELATIVE ROOF LOADS**

(Load on frame at 20 ft. spacing assumed to be 100%)

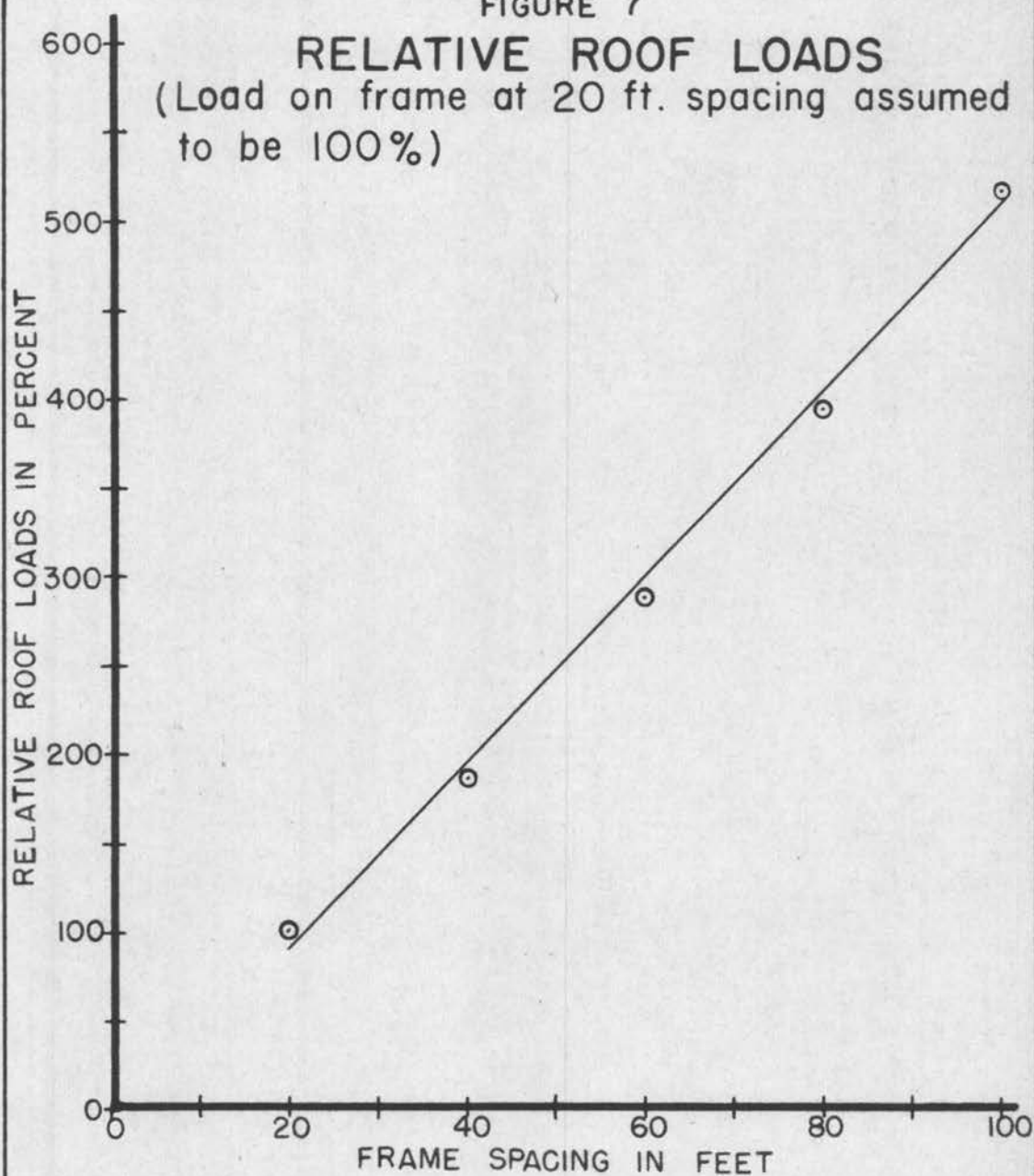
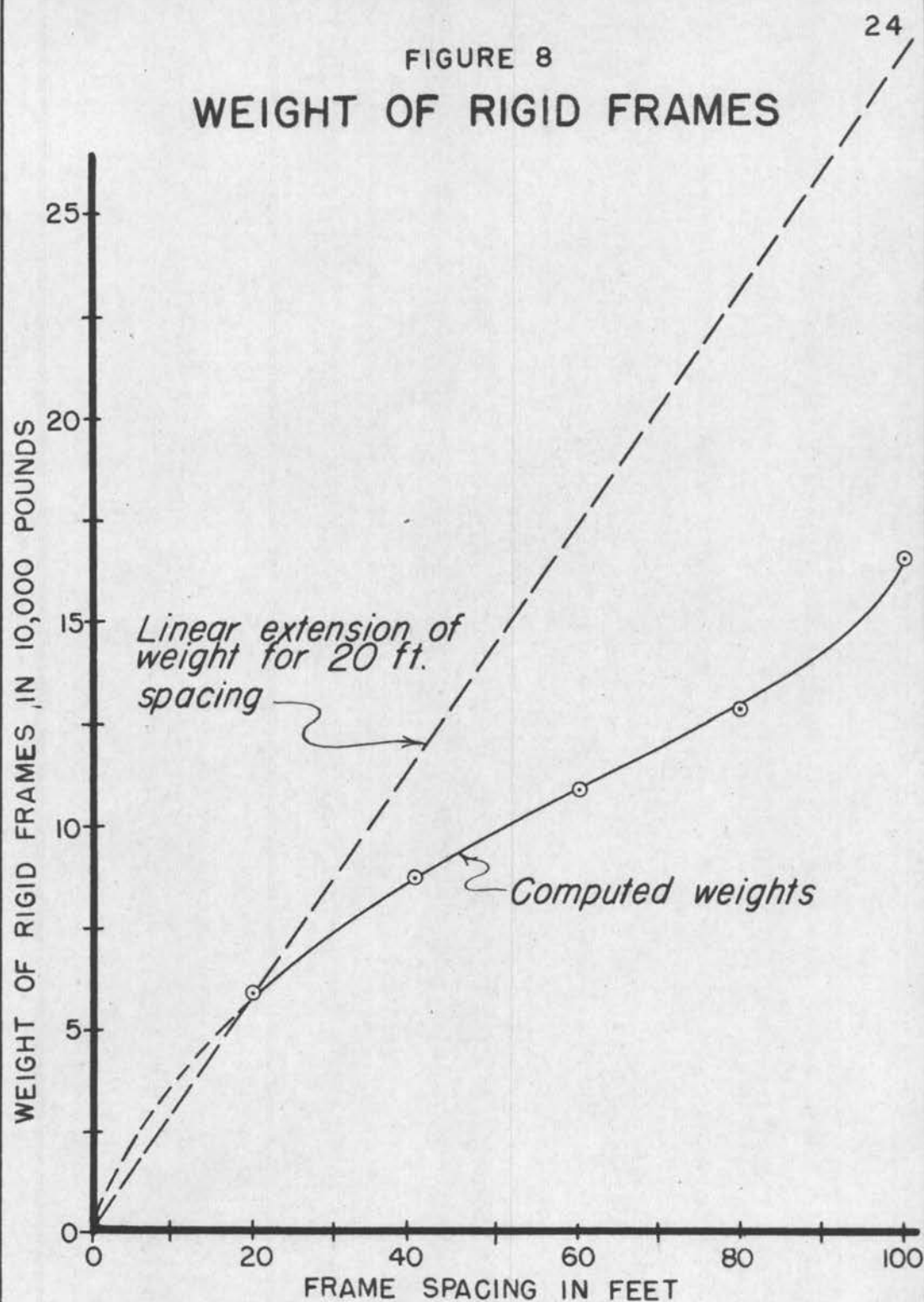


FIGURE 8  
WEIGHT OF RIGID FRAMES



## PRESENTATION OF RESULTS

The rigid frames were designed for 20 foot increments of spacing up to 100 feet. The 20 foot spacing intervals were selected to allow easy comparison with the controversial, conventional 20 foot spacing.

Once the frame has been designed, it might be well to check the moment of inertia ratio. The moment of inertia ratio using the figures determined by the design has a value of 0.85 for the first three spacings rather than the assumed value of 1. The ratio for the remaining two spacings was 0.70. This change in value of the moment of inertia ratio for all spacings causes an increase of only 1.5% in the magnitude of the reaction. For the purposes of a preliminary investigation and comparison the change in size of a rigid frame due to this increase can be disregarded.

The results of the preliminary design are tabulated in Tables 1, 2, and 3. Table 1 contains a list of the horizontal reactions and weights of each rigid frame at its respective spacing. The weights and horizontal reactions are also expressed as percentages of the frame spaced at 20 foot centers. This percentage rating will allow easier comparison in the discussion of the results.

Table 2 lists the depths and moments of inertia for the girder and column section. These values are also expressed as a percentage of the values for the 20 foot spacing. The depths tabulated in this table are those for the girder section only. The column depth is



about one inch greater, but the percentage would be the same as for the girder section.

Table 3 contains the pounds of steel in the rigid frame per square foot of floor area, covered by this structure.

The results tabulated in the tables are the quantities from which the curves contained in the discussion are plotted.



Table 1. Horizontal Reactions and Weight of Rigid Frames

Spacing	Weight of Rigid Frames lbs.	Weight of Rigid Frames %	Horizontal Reaction lbs.	Horizontal Reaction %
20	59,200	100	69,000	100
40	87,780	149	128,500	186
60	109,180	185	193,500	280
80	129,360	218	<sup>248,000</sup> 24,800	360
100	166,300	282	<sup>336,000</sup> 33,600	487

Table 2. Depths and Moments of Inertia.

Spacing	Depth ft.-in.	Depth %	Moment of Inertia			
			Girder		Column	
			in <sup>4</sup>	%	in <sup>4</sup>	%
20	4'-0 3/8"	100	25,200	100	30,740	100
40	5'-8 1/4"	142	68,980	274	82,280	272
60	7'-1 1/4"	178	130,600	520	154,045	518
80	7'-10 1/4"	196	170,170	675	232,670	780
100	8'-2 1/4"	205	206,332	820	319,472	1,060

Table 3. Weight of Frame per Square Foot of Floor Area.

Spacing	Weight/foot <sup>2</sup>	Weight as %
20	14.8	100
40	10.95	74
60	9.12	61.7
80	8.1	54.8
100	8.3	56.2

## DISCUSSION OF RESULTS

The results have been plotted against spacing so as to facilitate the interpretation of the effects of varied spacing. As an assistance in interpreting the curves, pertinent characteristics will be discussed.

The curves of Figure 8 can be interpreted in terms of slope which indicates weight per foot of spacing. The slope of the linear extension curve of the frame weight at 20 foot spacing is constant, thereby indicating a consistent gain of weight per foot increase of spacing. A curve having a slope greater or less than the linear extension curve will have a gain or loss respectively in weight.

The computed curve of Figure 8 has a slope flatter than the linear curve for a range of spacing from 20 to 80 feet. The flattest portion being between 60 and 80 feet. Beyond 80 feet there is a marked increase in slope rendering this range of spacing ineffective.

The curve in Figure 9 gives a range of depths within the limitations of built-up sections. Although a spacing beyond 80 feet appears favorable with regard to depth, the excessive weight beyond an 80 foot spacing still controls.

A unique combination desired in a structure is minimum weight and a maximum moment of inertia. Although an ideal relationship is seldom attained, the structure exhibiting the proper proportions of each characteristic is more suitable in the economical aspect.

FIGURE 9  
RELATIVE DEPTHS  
OF  
RIGID FRAMES

(Depth at 20 foot spacing  
assumed to be 100%)

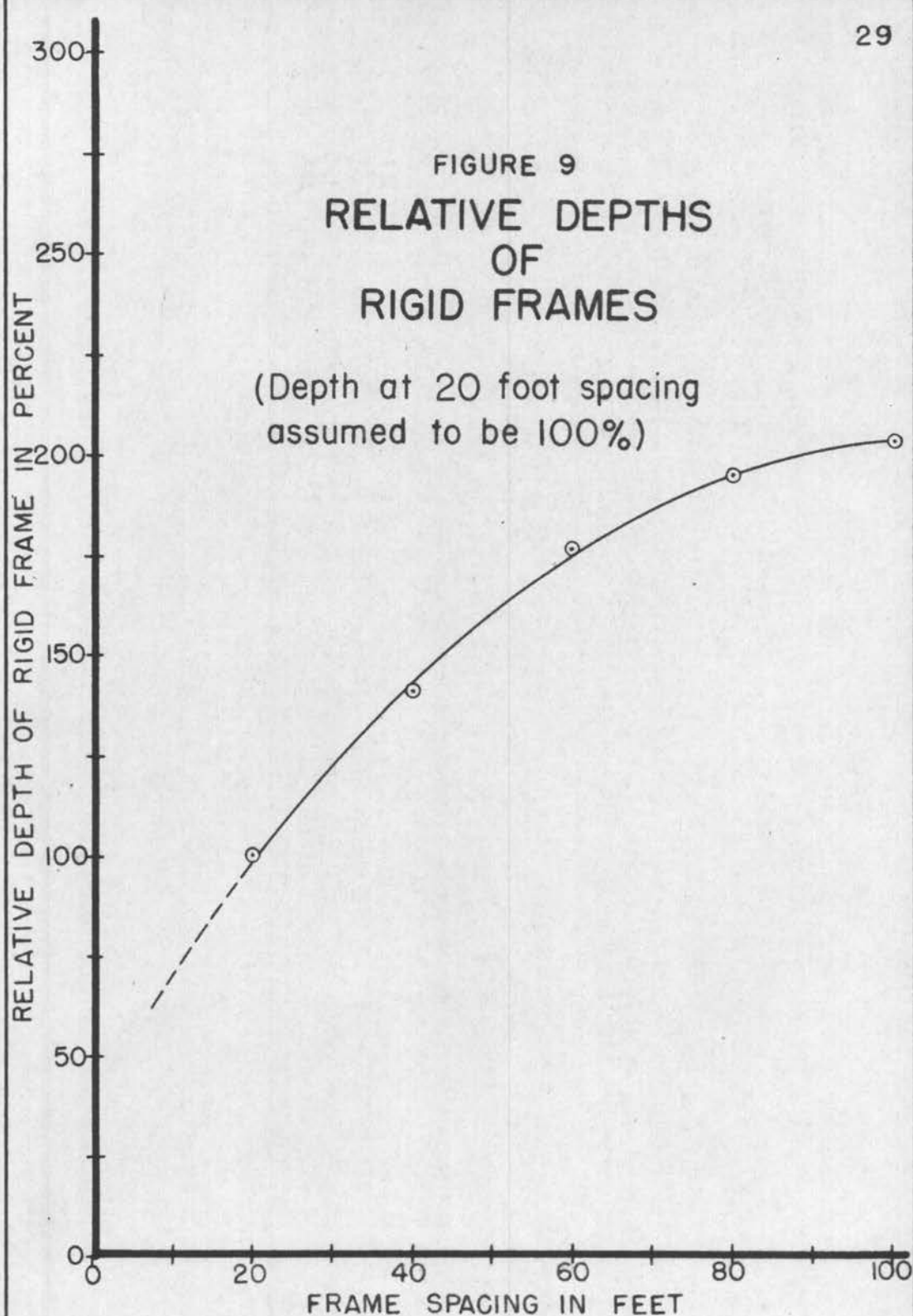
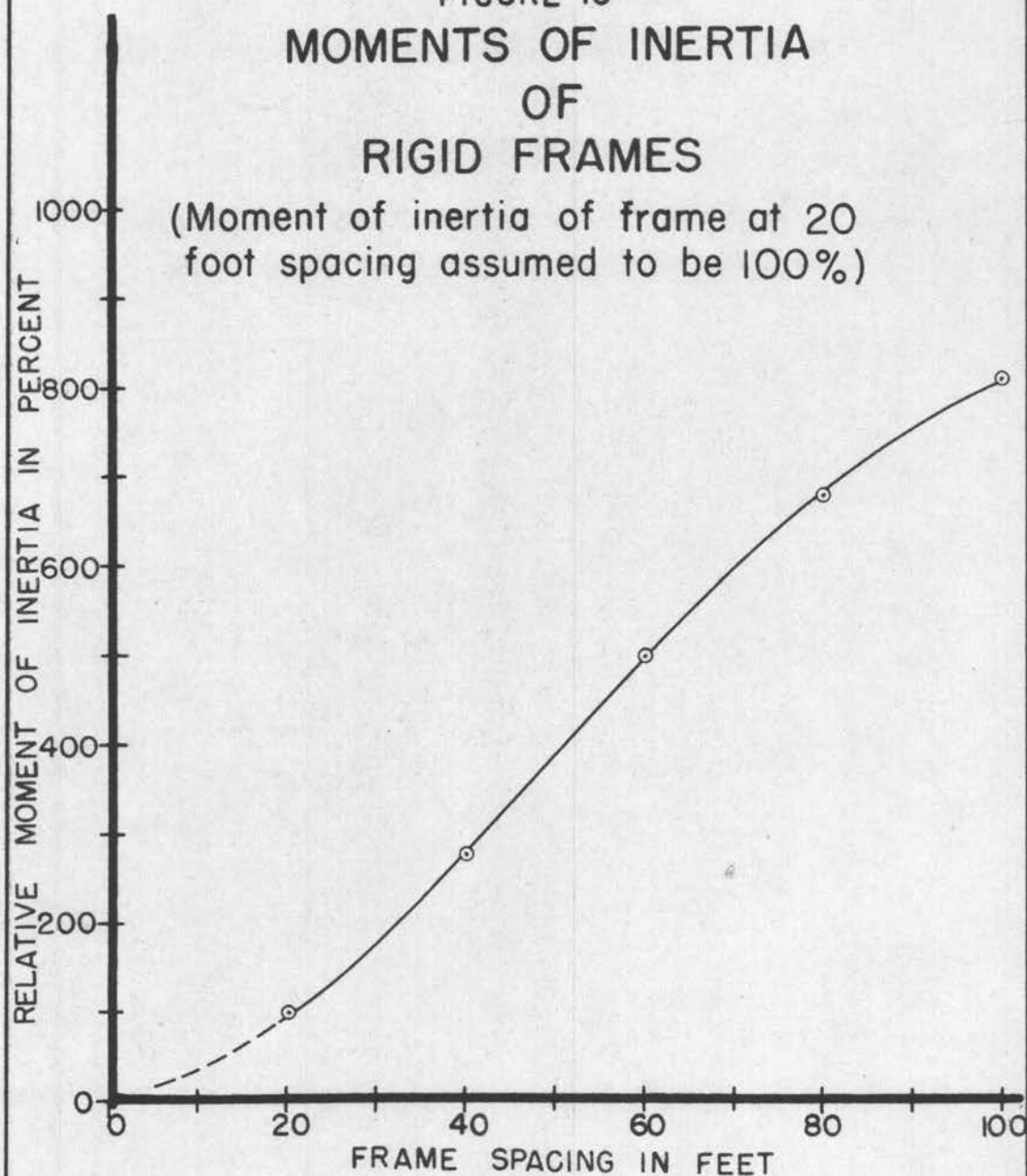


FIGURE 10  
MOMENTS OF INERTIA  
OF  
RIGID FRAMES

(Moment of inertia of frame at 20  
foot spacing assumed to be 100%)





A comparable increase in moment of inertia and weight up to a frame spacing of 80 feet is shown in Figures 8 and 10. Beyond 80 feet the weight increases more rapidly than the moment of inertia, thereby destroying the desired combination.

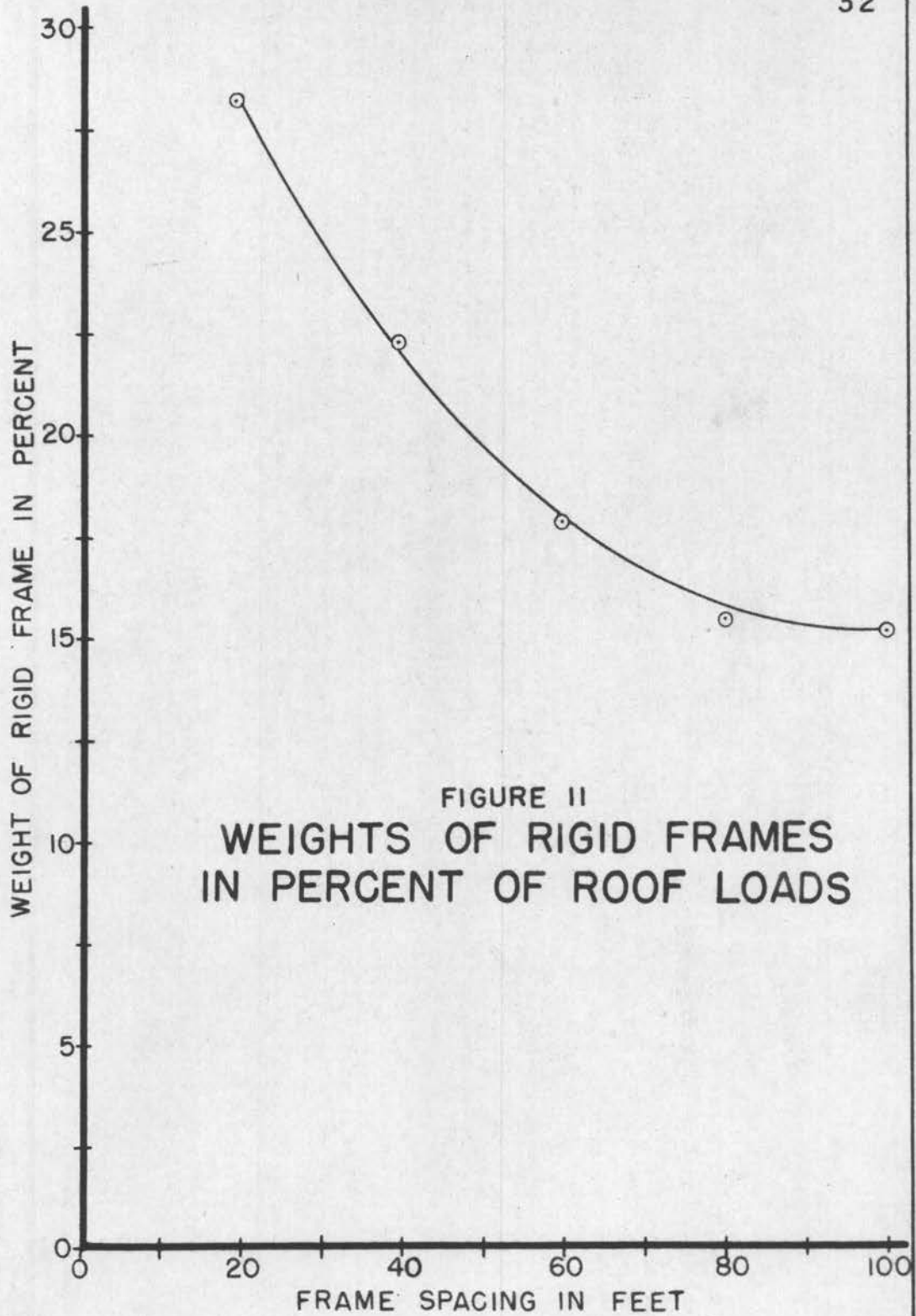
The loads on a structure are categorized as live load and dead load. The live load includes the loads to be supported by a structure and dead load includes the weight of the supporting structure. An ideal structure is one which would be weightless and accomplish its purpose. The structure more nearly approaching the ideal is one in which the dead load is a minimum percent of the live load. In the structure under discussion, the roof load is live load and the weight of rigid frame is dead load. The curve of Figure 11 shows the percentage of dead load to live load decreasing rapidly and considerably with an increase in spacing from 20 to 60 feet. Beyond this point the decrease is much more gradual and an apparent reversal is indicated at the 100 foot spacing.

Figure 12 is a presentation of the data of Figure 11 but is based on the floor area instead of referring to the roof. As anticipated a considerable savings in steel is accomplished by increasing the spacing.

The points in Figure 13 vary somewhat from a straight line, but for ease of comparison a straight line was drawn through them.

The rigid frames become a proportionately smaller part of the total load as the spacing increases. The linear extension curve indicates that a corresponding change in total load accompanies a





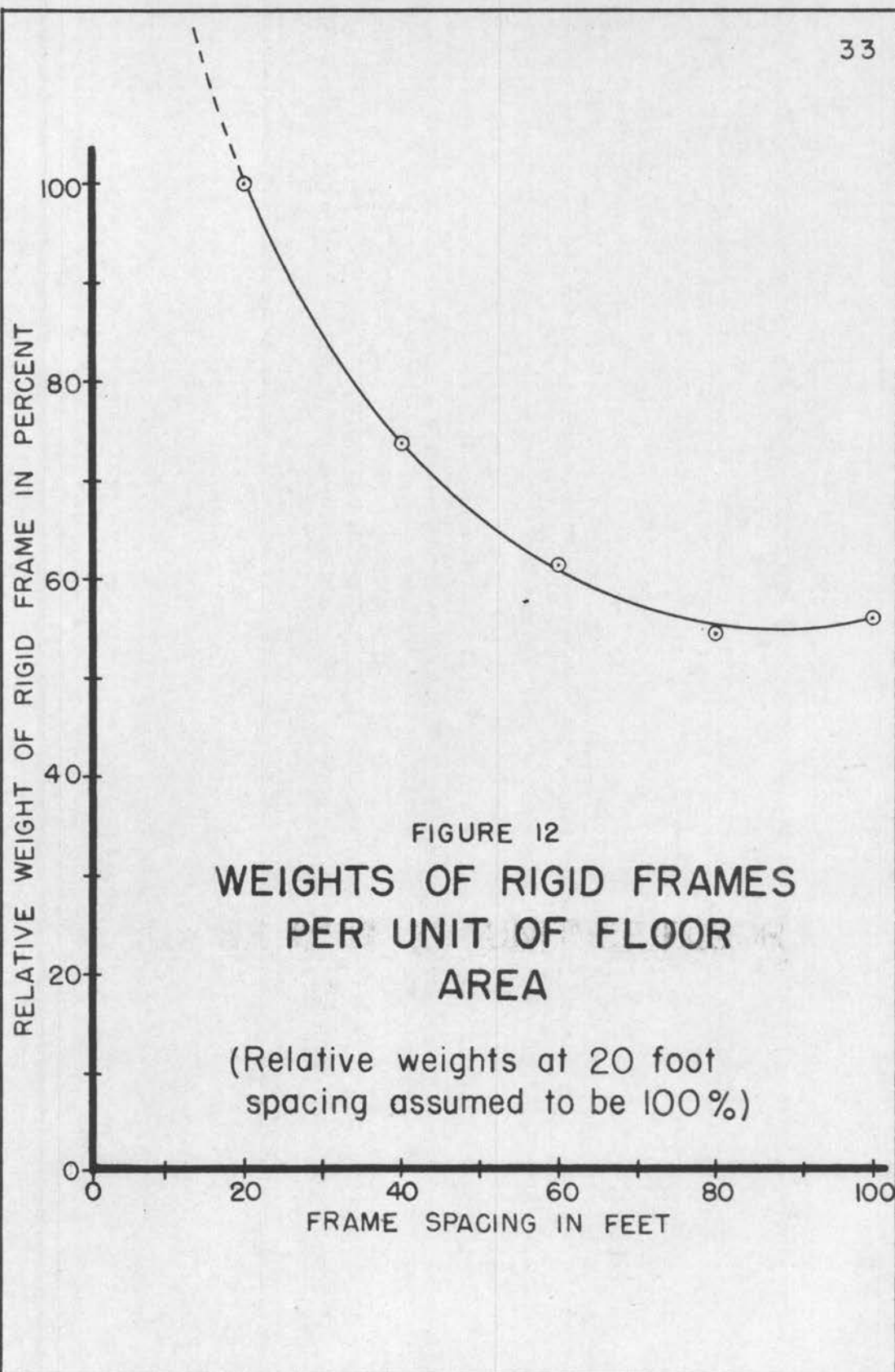
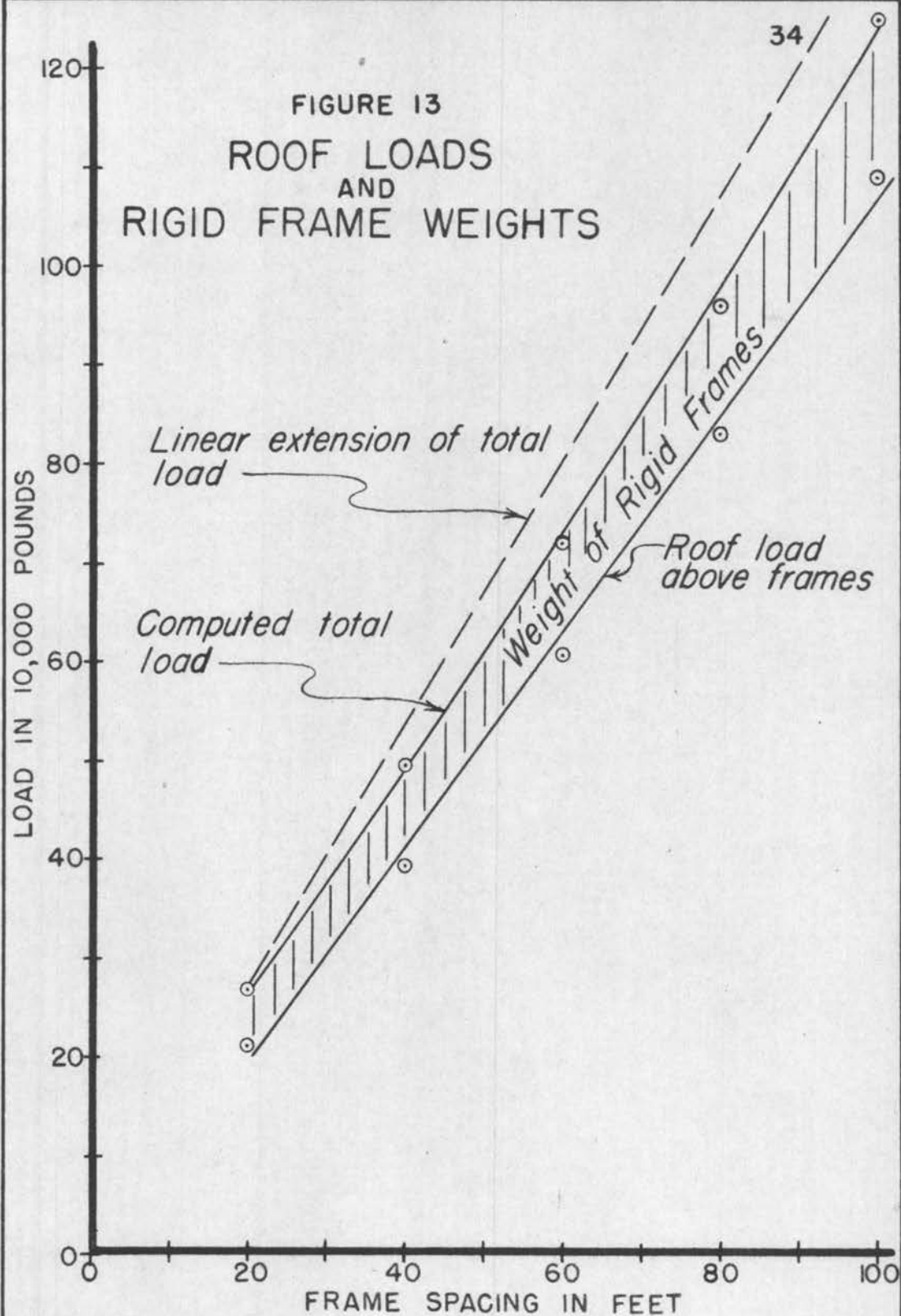


FIGURE 13  
ROOF LOADS  
AND  
RIGID FRAME WEIGHTS



change in spacing, therefore no advantage would be gained. A curve falling above or below the linear curve would indicate an increase or decrease respectively in total load. The position of the computed total load curve falls below the linear curve, thereby indicating a savings in weight of steel with increased spacing. This savings in weight of steel occurs both in the purlins and the rigid frame. The reduction in purlin weight by the use of open web joist reduces the roof load above the frames which in turn reduces the weight of the rigid frames themselves.



## CONCLUSIONS

The substitution of the open web joists for wide flange beams as purlins permits much longer purlin spans without excessive weights.

The use of welding in the fabrication of the rigid frame instead of riveted construction increases the load that can be carried by rigid frames of the same weight.

The combination of lighter purlins and welded rigid frames allows an increase in rigid frame spacing.

The roof load including purlins, carried by the rigid frames varies on practically a linear basis with rigid frame spacing.

The weight of the rigid frames varies on a less than linear basis with rigid frame spacings up to 80 feet.

The rate of decrease in dead load of the rigid frames varies from being very rapid for small spacing to one of insignificance at 60 to 80 foot spacings, beyond which the dead load increases.

One of the basic principles when considering a rigid frame structure in the preliminary stages is to determine the spacing at which minimum dead weight occurs. This spacing may then be modified as necessary to satisfy other conditions on a compromise basis.



## BIBLIOGRAPHY

1. Bleich, Dr. Friedrich. Design of rigid frame knees. New York, American institute of steel construction, July 1943. 30p.
2. Griffiths, John D. Single span frames in steel. New York, American institute of steel construction, October 1948. 29p.
3. Grinter, Linton E. Design of modern steel structures. New York, Macmillan, November 1949. 452p.
4. Grinter, Linton E. Theory of modern steel structures. New York, Macmillan, December 1936. 2 vols.
5. Lyse, Inge and W. E. Black. An investigation of steel rigid frames. In American society of civil engineers' transactions. New York, 1942. pp.127-183. (Paper no. 2130)
6. Stang, Ambrose H., Martin Greenspan and William Osgood. Strength of a riveted steel rigid frame having a curved inner flange. Vol. 21. Washington, National bureau of standards, 1938. 2420p. (Research paper No. 1161)
7. Stang, Ambrose H. and Martin Greenspan. Strength of a welded steel rigid frame. Vol. 23. Washington, National bureau of standards, 1939. 2319p. (Research paper no. 1224)

## APPENDIX

ADVANCE FUND  
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