

Guide for the Analysis of Guy and Stiffleg Derricks

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Data included in this publication was developed to suggest design criteria applicable to guy and stiffleg derricks. The data must be supplemented by the professional judgment of qualified engineering personnel. The reader is cautioned that independent professional judgment must be exercised at all times when the data set forth in this publication is considered and/or applied.

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FOREWORD

Standards of design applicable to cranes and derricks have never been established for the United States. Historically, the design and development of steel erection derricks evolved over the years from erection experience. Tests and design analyses were made principally by manufacturers, each of whom developed his own criteria. In addition, manufacturers took the position that much of their information was proprietary in nature and therefore were reluctant to release such material.

As a result of recent requirements by governmental bodies that derrick designs be certified, the AISC appointed a Task Force to study this problem and to prepare a Guide which could be used in the design and evaluation of guy and stiffleg derricks for steel erection. Derricks for performing duty cycle work were not included in the scope of the Committee assignment.

Several questions to which answers were not readily available were immediately identified by the Committee. Typical questions were:

- (a) Can an up-to-date guide be used to rate a derrick which was built many years ago?
- (b) Are any existing specifications, in total, suitable for the design of derricks, without modification?
- (c) What is the proper value of K (in the compression allowable stress formula) for derrick booms and masts?
- (d) What allowance should be made for impact and how should that impact allowance be applied?
- (e) Does wire rope, which is used for so many primary elements of a guy derrick, tend to damp the effect of impact?

Because some of these questions could only be answered by a test program on a full size derrick, a series of dynamic tests were made, under the auspices of the AISC and with the support of the National Erectors Association, on a guy derrick furnished by American Bridge Division of U. S. Steel Corp. at the U.S.S. Applied Research Laboratories in Monroeville, Pa. The program provided the needed answers and valuable full scale dynamic test information which is reflected in the recommendations of this Guide. A brief description of the test program and assembly is included as Appendix B.

The information developed from these guy derrick tests, design analyses and test data available from derrick manufacturers, a study of field experience with guy and stiffleg derricks, and an investigation of existing specifications for the design of steel structures form the basis for the recommendations and procedures presented in this Guide.

An AISC "Specification for the Design of Guy and Stiffleg Derricks" is presented as Appendix A of this Guide. This derrick specification includes applicable provisions of the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings", modified where required by the special considerations of derrick design. Fabrication is presumed to be in accordance with the AISC Specification for buildings. Approval of a grade or type material is not made with a view to excluding any other. It is obviously impractical to list all the structural materials (angles, pipes, plates, rivets, bolts, etc.) used for derricks which have given satisfactory service in the past and up to the present.

The derrick specification covers the design of structural components of the various types of guy and stiffleg derricks and Chicago Booms. It is limited to the design of derricks that are intended for use as heavy lift erection derricks.

The discussions which follow form a workable Guide for the evaluation of steel erection derricks of the guy and stiffleg type. Simple, straightforward solutions to the investigation of the structural components of the derricks are suggested. Crawler and truck cranes are excluded.

NOMENCLATURE

A	Total cross-sectional area, in. ²
A _c	Area of one chord, in. ²
A _e	Effective area of one chord, in. ²
A _w	Effective area exposed to wind, ft ² ; equal to C _d × (chord area + lacing area)
A ₁ , A ₂ , A ₃ , A ₄	Area of noted guys, in. ²
B	Reaction of boom at the boom pin, kips
C	Correction factor, equal to 1.05, to allow for transverse movement of mast top; applicable in multiple live guy cases
C _d	Shape factor for determination of effective area subject to wind force
C _m	Coefficient applied to bending terms in the Interaction Formulas; dependent upon curvature caused by applied moments
D	Dynamic factor applied to stresses computed on basis of live plus dead load; also termed "Impact Factor"
DL _{BT}	Total boom tip dead load, kips
DL _c	Concentrated dead load at boom tip (weight of topping falls block, upper main falls block, pin plates, one half the topping falls wire rope, etc.), kips
DL _{MF}	Main falls dead load (weight of main falls wire rope and lower main falls block), kips
E	Modulus of elasticity of steel (E = 29,000 ksi)
F _a	Allowable axial compressive stress in the absence of bending, ksi
F _b	Allowable bending stress in the absence of axial load, ksi
F' _e	Euler stress divided by factor of safety; equal to $\frac{12 \pi^2 E}{23 (K l/r)^2}$, ksi
F _y	Specified minimum yield point for the steel being used, ksi
G _E	Tensile force in the "effective guy", kips
G ₁ , G ₂ , G ₃ , G ₄	Total live guy force along the noted guy, kips
H	Total resultant horizontal force acting at the top of the mast, equal to (H _G + TF _H), kips
H _G	Horizontal force at the top of the mast in the plane of the boom resulting from the effect of the dead guys, kips
H _m	Horizontal reaction at the bottom of the mast, kips
H _s	Effective horizontal slewing (swinging) force applied at the boom tip normal to the axis of the boom, kips
H _w	Effective horizontal wind force applied at the boom tip normal to the axis of the boom, kips
I _{av}	Average moment of inertia of the tapered portion of a member, in. ⁴

I_1	Moment of inertia of a section at the small end of the tapered portion of a member (considering only chord angles), in. ⁴
I_0	Moment of inertia of a typical section in the prismatic portion of the member (considering only chord angles), in. ⁴
K	Effective length factor; equal to 1
LL	Live load (lifted load), kips
L	Length of member or length of panel, ft
L_{DG}	Average horizontal distance of all dead guys, ft
L_{GE}	Effective guy distance (horizontal projection of the slope length of the "effective guy", ft
L_m	Length (height) of mast, ft
L_1, L_2, L_3, L_4	Slope length of the noted live guy, ft
M	Moment, kip-in.
M_{DL}	Moment in boom due to uniformly distributed dead load, kip-in.
M_e	Moment in member due to concentrated moments applied at the end or ends of the member, kip-in.
MF	Main fall load (including weight of wire rope and lower main fall block); equal to $(LL + DL_{MF})$, kips
MFL	Lead line pull for the main falls, kips
M_H	Moment in boom due to effective horizontal slewing or wind force applied at the boom tip normal to the axis of the boom, kip-in.
N	Number of dead guys
P	Axial load at a particular section of the mast, kips
R	Reach or radius to the live load
T	Axial load or thrust at a particular section of the boom, kips
TF	Topping force, kips
TF_H	Horizontal force at the top of the mast in the plane of the boom resulting from the effect of the dead guys, kips
TFL	Force of topping lead line, kips
V	Shear, kips
V_m	Vertical reaction at the bottom of the mast, kips
b	Width of plate, in.
d	Distance between centroids of opposing chords, in.
f_a	Computed axial stress, neglecting impact, ksi
f_b	Computed bending stress, ksi
	f_{bx1} = Computed bending stress due to M_{DL}
	f_{bx2} = Computed bending stress due to M_e
	f_{by} = Computed bending stress due to M_H
h	Length of prismatic portion of the member, ft

l	Actual unbraced length in plane of bending, in.
p_w	Design wind pressure, lbs/ft ²
r	Radius of gyration, in.
t	Thickness of a plate, in.
w_{DL}	Average uniform dead load of boom excluding pin plates, kips/ft
w_m	Average uniform dead load of the mast, kips/ft
x	Distance from boom heel pin to section of boom under construction, ft; also, subscript indicating x-axis
x_1, x_2, x_3, x_4	Length of the x-ordinate distances of noted live guys, ft
β	Angle between topping falls and the centerline of the boom
γ	Coefficient equal to I_{av}/I_0
δ_{DL}	Deflection due to average dead load of boom w_{DL}
δ_{Me}	Deflection due to end moments M_e
θ	Angle between centerline of boom and a horizontal plane

GUIDE FOR THE ANALYSIS OF GUY AND STIFFLEG DERRICKS

I. BASIS FOR STRESS ANALYSIS

The design and development of steel erection derricks evolved from a combination of sound engineering principles, judgment, and erection experience, but without the benefit of uniform design standards. Available design specifications, developed primarily for buildings and bridges, did not address themselves to the unique problems of derrick design.

Over the years, derrick manufacturers developed parameters for good design practice and the structural analysis of booms and masts. These parameters, along with applicable provisions of the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings,* form the basis for the AISC Specification for the Design of Guy and Stiffleg Derricks** (see Appendix A). All analytical procedures suggested in this Guide meet the requirements of the Derrick Specification.

II. BOOMS AND MASTS

Two analytical approaches are available for the design or analysis of a boom or mast: (1) use of interaction formulas for combined axial and bending stresses and (2) deflection analysis. The interaction formulas of the Derrick Specification are slightly more conservative than the deflection analysis, but are easier to use. In this Guide, the analysis of booms and masts is made using interaction formulas; however, deflection analysis is equally satisfactory, and an example of deflection analysis of a guy derrick boom is presented in Section VIII, Design Examples.

The interaction formulas of the Derrick Specification are taken from the AISC Specification, February 1969 edition, but are modified in three ways. First, the term representing bending stress is separated into two parts; this permits the use of more precise values of the C_m factor for different conditions causing bending stress. Second, a new factor is introduced in the interaction equations. This is a "dynamic factor", D , which will be discussed in detail in Section II (F). Third, the effective length factor K of the AISC Specification does not appear in these interaction formulas, because the effective length is the actual length

($K = 1$) for guy and stiffleg derrick booms and masts.

The interaction formulas that must be satisfied for guy and stiffleg derrick booms and masts are:

$$\frac{f_a D}{F_a} + \frac{C_{mx1} f_{bx1} D}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{mx2} f_{bx2} D}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}} \leq 1.0 \quad (D1.5-1a)$$

and

$$\frac{f_a D}{0.60 F_y} + \frac{f_{bx} D}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (D1.5-1b)$$

where:

- D = Dynamic factor
- C_{mx1} = Coefficient taken as 1.0
- C_{mx2} = Coefficient equal to $0.6 - 0.4 (M_1/M_2) \leq 0.4$
where (M_1/M_2) is the ratio of smaller to larger moments at ends of portion of member unbraced in plane of bending
- C_{my} = Coefficient equal to 0.85
- F_a = Allowable axial compressive stress for boom as a whole in the absence of bending, ksi. (When computing F_a , the average moment of inertia, I_{av} , must be used.)
- F'_e = Euler stress divided by Factor of Safety, ksi; equal to $\frac{12\pi^2 E}{23 (l/r)^2}$
(when computing F'_e , the average moment of inertia, I_{av} , must be used.)
- F_{bx}, F_{by} = Allowable bending stress about x- or y-axis, respectively, in the absence of axial load, ksi; computed on the basis of column action of a chord between lacing points. (At plated end sections of booms and masts, F_b is the usual allowable bending stress equal to $0.6F_y$.)

*Hereafter referred to as the "AISC Specification."

**Hereafter referred to as the "Derrick Specification."

- f_a = Computed axial stress, based on effective area, neglecting impact, ksi
- f_{bx} = Computed bending stress about the x-axis due to dead load moment and end moment, ksi; equal to the sum of f_{bx1} and f_{bx2}
- f_{bx1} = Computed compressive bending stress about the x-axis due to transverse (dead) load moment neglecting impact, ksi; in latticed members, equal to half the axial compressive force of the couple resisting the moment divided by the effective area of one chord
- f_{bx2}, f_{by} = Computed compressive bending stress about the x- or y-axis, respectively, due to end load moments neglecting impact, ksi; in latticed members, equal to half the axial compressive force of the couple resisting the moment divided by the effective area of one chord
- l = Actual unbraced length in plane of bending, in.
- r = Radius of gyration, in.
- x, y = Subscripts indicating horizontal or vertical axis, respectively, of the boom or mast

Both interaction formulas must be satisfied and should be checked, although Formula (1.5-1b) is likely to govern only at the ends of the member. The individual terms of the formulas and the recommended constants C_{mx1} , C_{mx2} , C_{my} , and D to be used will be discussed in sections of the Guide dealing specifically with guy derrick booms, stiffleg derrick booms, and masts.

Note that the fourth term of Formula (D1.5-1a) and the third term of Formula (D1.5-1b), i.e., the y-axis bending terms, are applicable only to stiffleg derrick booms subject to wind or slewing forces. See discussions in Sects. II (D), Slewing, and II (E), Wind.

A. Moments of Inertia of Tapered Members

Derrick masts, booms and stifflegs very often are built with varying moments of inertia. The main portion of the assembly may be prismatic, while the ends may be a truncated pyramid. On the other hand, the assembly may merely be two truncated pyramids with the bases meeting at mid-length of the assembly.

The critical buckling load of a tapered column made up of four angles of constant cross-sectional area connected by lacing over the entire length of the column can be calculated using the concept of an average moment of inertia, I_{av} .^{*} This average moment of inertia can be found from the equation $I_{av} = \gamma I_0$, where I_0 is the moment of inertia of a section in the prismatic portion of the member, and γ can be found from Fig. 1a or 1b. However, actual booms or masts generally have plates on part of the length of one or more faces. If such plates are found only near the top and/or base, it is conservative to ignore them when determining the average moment of inertia.

When one face of the assembly has a continuous plate over the entire length, this plate should be used in calculating I_0 and I_1 to determine the average moment of inertia from Fig. 1a or 1b.

Note that if the moment of inertia I_1 at the top differs from that at the bottom, the smaller of the two values should be used in Figs. 1a and 1b to find the ratio I_1/I_0 .

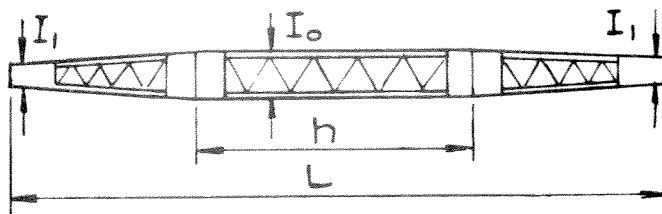
The radius of gyration, r , of the boom or mast acting as a column is found by using the average moment of inertia, I_{av} , in $r = \sqrt{I_{av}/A}$. For a latticed member made up only of angle chords and lacing, the total cross-sectional area, A , is constant (equal to $4A_C$, where A_C is the area of one chord). Where one face of the member has a continuous plate over its length, the sum of the area of the angles plus a weighted average area of the plate should be used.

B. Effective Areas of Compression Elements

Appendix C of the 1969 AISC Specification deals with slender compression elements having width-to-thickness ratios in excess of the applicable limits given in Section 1.9 of the Specification. Before the introduction of Appendix C into the 1969 AISC Specification, engineers used the concept of "effective area" for compression elements with excessive width-to-thickness ratios. Under this concept, the moment of inertia and radius of gyration are those of the full section. The allowable compression stresses are found by going directly to the tables of allowable column stresses using these cross section properties.

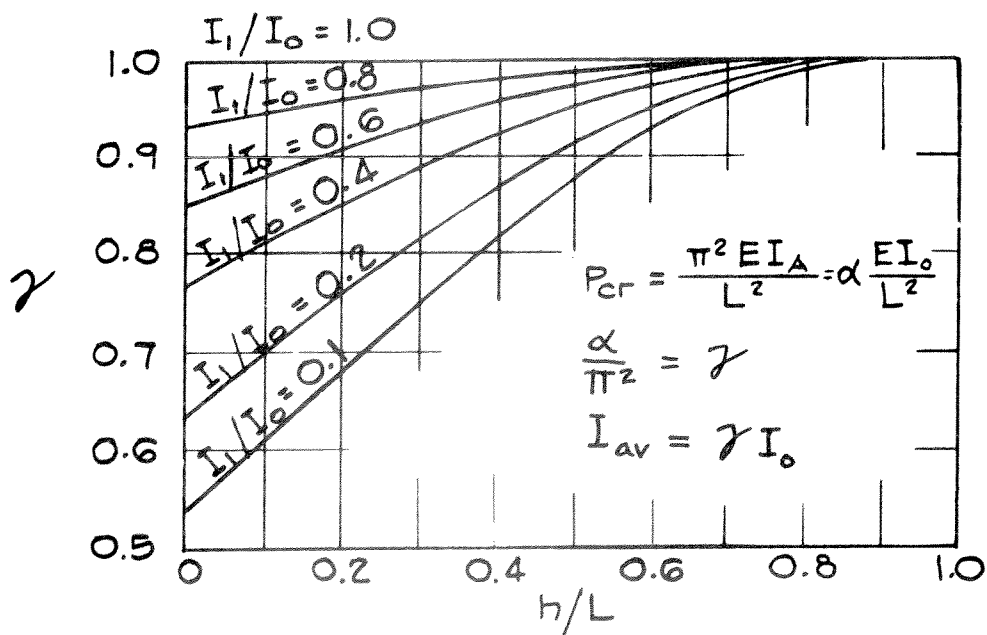
Effective area is that part of a cross section within which the width-to-thickness ratios do not exceed the applicable limits of AISC Specification Sect. 1.9. The effective area concept is conservative and less involved. It is recommended in derrick design in lieu of Appendix C of the AISC Specification, although use of Appendix C is also valid.

^{*}Timenshoko & Gere, "Theory of Elastic Stability," Second Edition, McGraw-Hill Book Co., New York, 1961.



$\frac{1}{I}$	h/L					
	0	0.2	0.4	0.6	0.8	1.0
0.1	0.548	0.675	0.818	0.937	0.993	1.000
0.2	0.645	0.760	0.873	0.957	0.995	1.000
0.4	0.772	0.853	0.929	0.976	0.998	1.000
0.6	0.863	0.916	0.962	0.987	0.999	1.000
0.8	0.937	0.963	0.982	0.995	1.000	1.000
1.0	1.000	1.000	1.000	1.000	1.000	1.000

(a) Table for determining values of γ



(b) Curves for determining values of γ

Figure 1

C. Allowable and Calculated Stresses

The value for the allowable axial compressive stress in the absence of bending, F_a , is found from Table 1, Appendix A of the AISC Specification, for the given yield stress steel, with $K = 1$, $l = 12L = \text{full length of the member in inches}$, and $r = \sqrt{I_{av}/A}$.

The calculated axial stress, f_a , is determined by dividing the axial force in the member by the effective area of the four chord angles.

F_b is defined as the allowable bending stress in the absence of axial load. For portions of booms and masts made up entirely of angles and lacing, F_b should not be taken as the usual allowable bending stress from the AISC Specification. For the purpose of determining F_b , laced booms should be considered as trusses; thus, F_b then becomes a chord segment between lacing points acting as a compression member and is equal to the value of F_a determined using l/r for that chord segment.

When determining the l/r of a chord segment for use in calculating the value of F_b , two types of lacing may be encountered. For staggered lacing (Fig. 2a), where one leg of an angle chord segment is braced at mid-length by lacing in the plane of the other leg, the lesser radius of gyration r_x or r_y should be used in determining the applicable slenderness ratio l/r . Where the lacing is not staggered, as in Fig. 2b, the radius of gyration about the z-axis must be used. For design of lacing, see Sect. D1.10.2.5 of the Derrick Specification in Appendix A.

The calculated bending stress, f_b , in any one chord is given by:

$$f_b = \frac{M}{2dA_e}$$

where d = distance between centroids of chords
 A_e = effective area of one chord
 M = moment at the cross section

At the ends of booms and masts, where plates on all faces form a box beam, the actual section modulus of the total plated cross section should be used to determine f_b in the equation $f_b = Mc/I$ and the allowable bending stress F_b is taken as $0.6F_y$.

D. Slewing (Swinging) of the Lifted Load

Guy derrick mast and boom assemblies are rotated (or swung) about the vertical axis of the mast by using manpower to operate a bull wheel or crank, since the hoist is not on the derrick floor. Swinging is necessarily very slow. Therefore, no lateral loads at the boom tip due to slewing (swinging) need to be considered for a guy derrick.

Stiffleg derricks are swung by using a bull wheel which is pulled round by wire rope pennants powered from auxiliary drums on the main hoist. Since acceleration and swing are much more rapid than in the case of a guy derrick, the load will lag behind the boom tip, causing the load falls to be out of plumb, thus creating a horizontal component to the falls load. The magnitude of this horizontal force at the boom tip has been the subject of a great deal of investigation among derrick designers. Essentially the problem is a function of the line speed and acceleration available at the auxiliary swinger drums and the diameter of the bull wheel. In construction derricks the line speed at the bull wheel is reduced by the introduction of slewing falls between the hoist drum and the pennant leading to the bull wheel. These slewing falls have a minimum of 3 parts, although 4 or even 5 are not unknown.

While maximum acceleration of the swinger lines is of theoretical interest, it is not a practical consideration when the load is handled without tag lines. There is no conceivable reason why a licensed operating engineer would accelerate a swinger hoist to full speed when capacity load is on the falls at a long reach.

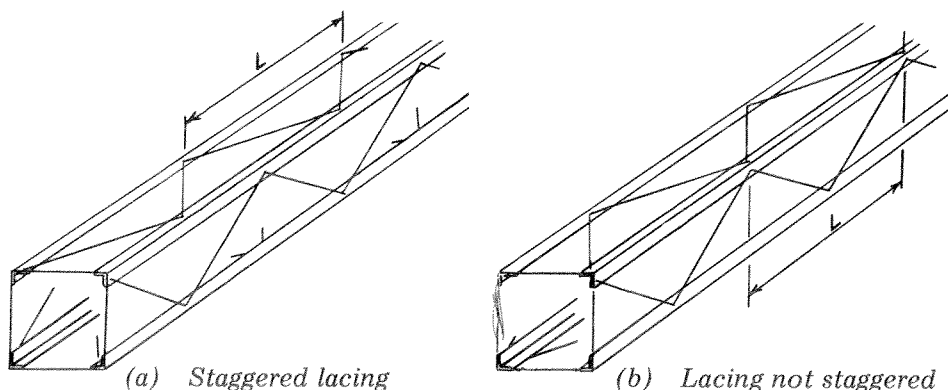


Figure 2

Therefore, for stiffleg derricks as used in steel erection, it is recommended that the following horizontal forces due to slewing, H_s , be applied at the boom tip:

- (1) For booms making an angle of 60° , or more, with the horizontal, $H_s = 2\%$ of the combined live load plus dead load at the boom tip = $0.02 (LL + DL_{BT})$.
- (2) For booms which make an angle of less than 60° with the horizontal, H_s varies linearly from 2% of $(LL + DL_{BT})$ when the boom is 60° with the horizontal, to 3% of the combined loads when the boom is flat.

These values are based on stop watch readings of swinger hoist accelerations on actual stiffleg derricks on construction projects. Since minimum diameter bull wheels were on these derricks, the most critical cases were observed.

E. Wind

1. Stiffleg Derrick Booms

(a) Wind on the Lifted Load

In building erection, wind on the lifted load has so small an effect that it can safely be omitted from stress calculations. Steel is hoisted parallel to the building face and wind exposure on the lifted load is restricted to a small area.

In bridge erection, wind on the lifted load of a stiffleg derrick must be considered. Here the load may present a large sail area and proper consideration of wind force on the derrick boom should be included in the analysis. For such applications, the wind force on the lifted load should be taken as 2% of the live load (lifted load), applied as a lateral load at the boom tip. (Dead loads on the boom, including tackle weight, are not considered part of the lifted load.) Note that the effect of wind on the lifted load must be added to the effect of wind against the boom (see (b) below) in determining the total wind stresses on the boom.

(b) Wind on the Boom

The stresses in a boom due to wind force against the side face of the boom are only significant when there are also stresses due to wind on the lifted load. Therefore, in building erection, wind on the stiffleg derrick boom can be safely omitted from stress calculations.

In bridge erection, the effect of wind against the boom of a stiffleg derrick must be added to the effect of wind against the lifted load to determine the total wind stresses in the boom. The effective force acting at the boom

tip due to wind pressure against the boom face, H_W (boom), can be considered to be one-half the total wind force against the boom face:

$$H_W (\text{boom}) = C_d p_W A_W / 2$$

where

$$p_W = \text{Design wind pressure, lbs/ft}^2$$

$$A_W = \text{Total area exposed to wind} \\ = (\text{Chord Area} + \text{Lacing Area}), \text{ft}^2$$

$$C_d = \text{shape factor} \\ = 2.00 \text{ for angles and tees} \\ = 2.03 \text{ for square or rectangular tubes} \\ = 1.21 \text{ for round tubes}$$

Note: Use the chord area for one face and the lacing area as seen in elevation (may include both vertical faces) in determining area A_W .

Design wind pressures,* regardless of the height of the derrick above datum, for the design of booms, are as follows:

Normal Conditions

$p_W = 1.0 \text{ lb/ft}^2$ (20 mph) with rated load on the main falls.

Abnormal Conditions:

$p_W = 6.4 \text{ lbs/ft}^2$ (50 mph) with no load on the main falls.

(Note that under abnormal conditions, no lift would be made. In more severe high wind velocity emergencies, the boom should be pulled up tight against the mast or boomed out flat and secured; therefore, design for extreme wind conditions is not required.)

(c) Combined Wind and Slewing Forces

In bridge erection, booms of stiffleg derricks may be subject to slewing forces acting in the same direction as wind forces. In practice, it is an unusual circumstance for the boom to be subjected to the combination of wind and slewing forces simultaneously. When handling large capacity loads, the load is generally swung with the boom raised (the load, therefore, being normal to the boom and presenting a small area). When the load is approximately in line with its final position, the derrick is boomed out. Small adjustments by swinging are made very slowly. It is recommended that only the greater of either the slewing force (horizontal force H_s acting at the boom tip (see Section II (D)) or a wind force equal to 2% of the live load plus $\frac{1}{2}$ of the total wind load against the boom (horizontal force H_W acting at the boom tip) be used in stress calculations.

*Design wind pressures are based on $P_W = 0.00256V^2$ (dynamic pressure). Refer to Paper No. 3269, "Wind Forces on Structures", ASCE Transactions, Vol. 126, Part II, 1961.

2. Guy Derrick Booms

Wind forces on guy derrick booms can safely be omitted from stress calculations, whether the derrick is used for building or bridge erection. Guy derrick booms are essentially free to rotate under wind load, so that stresses induced by wind are of negligible magnitude. (In the case of guy derricks used in bridge erection as travelers or material transfer derricks, wind on the lifted load may create a handling problem, but is not a design consideration.)

3. Masts and Stifflegs

Wind loads on masts of guy derricks and masts and stifflegs of stiffleg derricks are not significant and are not generally considered in the design of such members. The maximum stresses due to design live and dead loads always occur near the ends of the mast, whereas the maximum stresses due to wind on the mast will be small and will occur at mid-height.

F. Dynamic Allowances

Dynamic stresses are induced during normal operations of a derrick, such as picking a load from a truck and hoisting to the working floor. This dynamic (or impact) effect has been included in the interaction formulas of the Derrick Specification by the addition of the dynamic factor D (see Eqs. (1.5-1a) and (1.5-1b), Appendix A. This modification resulted from studies of the tests conducted by the AISC Task Force. It was found that by increasing the numerator in all terms of the formulas (except the "y-axis" terms) by some value of D, stresses measured in a boom under dynamic conditions were closely approached. On the other hand, when simply increasing the hook load by a percentage, correlation with test results was not good. This is because the energy applied by motion at the hook is absorbed immediately and simultaneously by all elements of the derrick setup and cannot be separately identified in the ensuing elastic vibrations of the masses of the individual elements; hence, the so-called "impact factor" applied to live load only is misleading in the analysis of derricks.

With dynamic loads (as differentiated from static or steady state loads), the time taken to apply the load is crucial. This time is measured in fractions of a second.

In selecting a hoist for use with a derrick, it is customary to choose one which can produce sufficient lead line pull at the drum with the maximum number of anticipated layers of wire rope on the drum. Transportation considerations as well as initial cost of the hoist will generally

work against the selection of an excessively powerful hoist.

The brakes furnished with a hoist are intimately related to lead line pull; therefore, the ability of a given brake to stop a load is related to the hoisting capacity. Brakes cannot stop a drum suddenly when that drum is loaded by a rated lead line pull and moving at rated speed. It takes time to decelerate the load. When the actual lead line pull is a small percentage of the rated lead line pull, the time required to decelerate is shorter than at full lead line pull. For this case, the percent of increase above static stress is high, but the static stress itself is necessarily small. For example, power brakes on a car moving at one mile per hour may stop a car in a few feet. But at 60 mph, those same brakes may require hundreds of feet to stop the same car, and there will be a smaller percentage of increase in dynamic stress over steady state stresses. The significance of this principle is clearly brought out in the data presented below.

In tests by a hoist manufacturer, the following data was observed for a hoist with spring-powered air release brakes:

Static Line Pull		Dynamic Line Pull to Stop		% Increase	Ratio Dynamic to Static
Kips	% of Rated	Kips	% of Rated		
25.0	100	33.0	132	32	1.32
17.5	70	28.8	115	64	1.64
10.0	40	21.5	86	115	2.15

The next-to-last column shows the percent of increase of the braking load over the steady state load. Note that for stopping a moving 10 kip load a braking load of 21.5 kips (an increase of 115%) was measured. Yet, 21.5 kips is only 86% of the 25 kip line pull at rated load.

The AISC Task Force tests on the guy derrick were made with 5 parts in both main falls and topping falls. The hoist had mechanical brakes and was rated at 10,000 lbs lead line pull. The actual lead line pulls at braking were observed to be:

First Test Series:

Boom Position	Lifted Load	Lead Line
Flat Boom	6 kips	1.6 kips
45° to Horiz.	10 kips	2.5 kips
Steep	14 kips	3.3 kips

Second Test Series:

Boom Position	Lifted Load	Lead Line
Flat Boom	12 kips	2.7 kips
60' Reach	18 kips	4.2 kips
20' Reach	26 kips	5.8 kips

For the first series, maximum lead line pull was 33% of rated hoist capacity, while for the second series it was 58%. If a smaller hoist had been used, the ratios of dynamic to static loads obtained in the tests (except for brake after acceleration) would have been smaller.

1. Dynamic Factors for Normal Conditions

Tests conducted by the Task Force were made in an attempt to obtain data for a variety of conditions — those pertaining to normal working conditions and those thought to come close to structural failure.

One condition, which occurs many times a day on any job, is the unloading of steel from a truck and raising it to the derrick working floor. The Task Force measured two phases of this operation. In the first phase, designated as "acceleration", the load was lifted and the hoist accelerated to normal speed as quickly as possible. For this phase, keeping in mind the previous discussion, ratios of total dynamic to total static stresses of approximately 1.22 were recorded with the boom in the flat position (90 ft radius) handling a rated capacity load of 6 tons. With the same 6 ton load, and the boom at the 40 ft and 20 ft radii, the stresses recorded were too small to give reliable dynamic-to-static ratios, since the strain gauge readings can be in error by 20 microstrains (± 600 psi).

After attaining normal speed, power was cut off and the brake applied. This was phase two, designated as "brake after acceleration". Here again, with the 6 ton load, the ratios of dynamic to static stresses, with the boom out flat, were about 1.22. Again, at radii of 40 ft and 20 ft, stresses were too small for reliable use.

However, with a lifted load of 13 tons, at both the 20 and 40 ft radii, the stress ratios for "brake after acceleration" were 1.48 and 1.35, respectively.

Two points must be considered. First, the tests which were made on the full size guy derrick were conducted under conditions which were excessively severe in that the hoist operator treated capacity loads in an extremely rough fashion. In fact he commented that he would "be run off the job if he treated the derrick this way". The ratios

1.48 and 1.35 are excessively high for a rational basis of design; under normal working conditions with a qualified operator, they would be considerably lower.

The second point concerns the dynamic yield point of steel. The rate of strain prescribed in ASTM A370, Section 10, is "not to exceed 100,000 psi per minute". For the old A7 steel, 33,000 psi yield, the test would take 20 seconds with a strain rate of 5.75×10^{-5} in./in./sec. For A36 steel, the yield point would be reached in 21.5 seconds. As the rate of strain increases, among other things the following takes place:

- (a) The yield stress increases to some dynamic value.
- (b) Strain hardening begins at a larger strain.*

Data obtained by the Task Force show that, for the boom as the 20 ft radius, dynamic fluctuations took place at a natural frequency of 3 cycles per second, where each cycle consists of a loading and unloading. If the steel were A7, the derrick test load would have been applied 240 times as rapidly as the maximum speed allowed by ASTM, or a rate of 1.38×10^{-2} in./in./sec., since each loading took place in 1/12 of a second.

The average percentage increase from normal yield at the ASTM test strain rate to dynamic yield at the strain rate of loading of the test derrick was 10%. This same trend is typical of all structural steel up to 50 ksi yield stress material.

The matching of hoist to load is not of any significance for the case of brake after acceleration since, when the brake is applied, only the inertia force of the rotating hoist drum is being overcome.

In summary, since normal operating conditions of a derrick will not be as severe as the test conditions were, and since the dynamic yield stress is about 10% higher than normal yield, it is recommended that when using Interaction Formulas (D1.5-1a) and (D1.5-1b) for normal operating conditions, the value of the dynamic factor D in the numerator of each term should be taken as 1.2.

2. Dynamic Factors During Slewing (Stiffleg Derricks)

In the case of stiffleg derricks, it is highly unlikely that there will be vertical acceleration or braking of the main falls lead line simultaneously with acceleration due to slewing while full capacity load is being handled. Therefore, it is recommended that, for the slewing condition, the dynamic factor D in the interaction formulas be taken as 1.03, to allow for minor vibrations due to swinging.

*See Norris et al, "Structural Design for Dynamic Loads", McGraw-Hill Book Co., New York, 1959.

3. Dynamic Factors for Abnormal Conditions

Every structure may be subjected on occasion to loads in excess of those considered in normal design. A condition of positive overload on a derrick is brought about by certain specifications which require that a derrick be tested above its rated capacity. In investigating a derrick for abnormal conditions, it is reasonable to permit stresses in excess of the normal allowable stresses. Use of the above recommendations for a dynamic factor of 1.2 for normal conditions at usual allowable stresses actually provides for dynamic amplification of more than 2.0 times the design live load on the hook without exceeding critical stress.

Nothing in this Guide is to be construed as condoning use of a derrick to pick loads during actual construction greater than those allowed by the manufacturer's Capacity Chart. Tests using actual overloads, made under carefully supervised conditions, when required by certification test specification, are the only exception. It is recommended that test loads on the hook, if required, be not more than 1.5 times the rated capacity.

G. Fatigue

The range in stress, from no load to rated capacity, can safely be repeated far in excess of the number of times that will actually occur in the life of a heavy lift steel erection derrick without producing fatigue. Hence, no provision is made in this Guide for the reduction of allowable stresses due to fatigue. Only derricks used in steel erection are covered. Duty cycle equipment is not included or implied in the consideration of fatigue effects on booms and masts within the scope of this Guide.

H. Camber

Occasionally booms are designed with camber built in. The dead load moment at any section being analyzed is modified by the product of the thrust and camber. Camber can be helpful in increasing capacity at maximum radii; however, it should be noted that boom camber can have a negative effect on capacity at short radii.

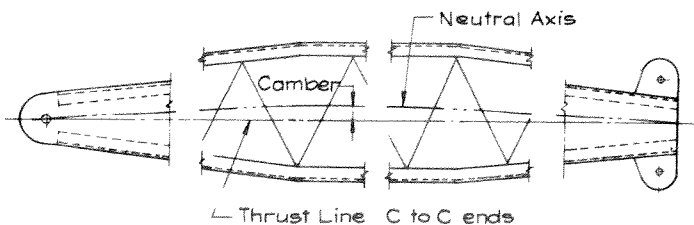


Fig. 3. Boom camber

In Fig. 3, the camber at any section is the distance from the neutral axis of the cross section to a straight line from the boom tip to the boom foot pin. The boom foot pin may or may not be on the neutral axis. For convenience in computation, the thrust is always assumed to be acting normal to the section being analyzed.

I. Guy Derrick Boom Stresses

Following is a general procedure for investigating the structural capacity of a guy derrick boom. (In designing a new boom, a trial section is selected and then investigated in a similar manner.)

1. Determine the Dead Loads on the Boom

- (a) Dead load of the main falls tackle, DL_{MF} , consisting of lower MF block and wire rope. (Note the lower MF block is included. This is different from crawler and truck crane applications where the lower MF block is considered part of the lifted load.)
- (b) Concentrated dead load at the boom tip, DL_C , consisting of the topping falls block, upper MF block, pin plates, one half of the topping falls wire rope, etc.)
- (c) Uniform dead load of the boom, w_{DLL} .

2. Determine The Boom Properties

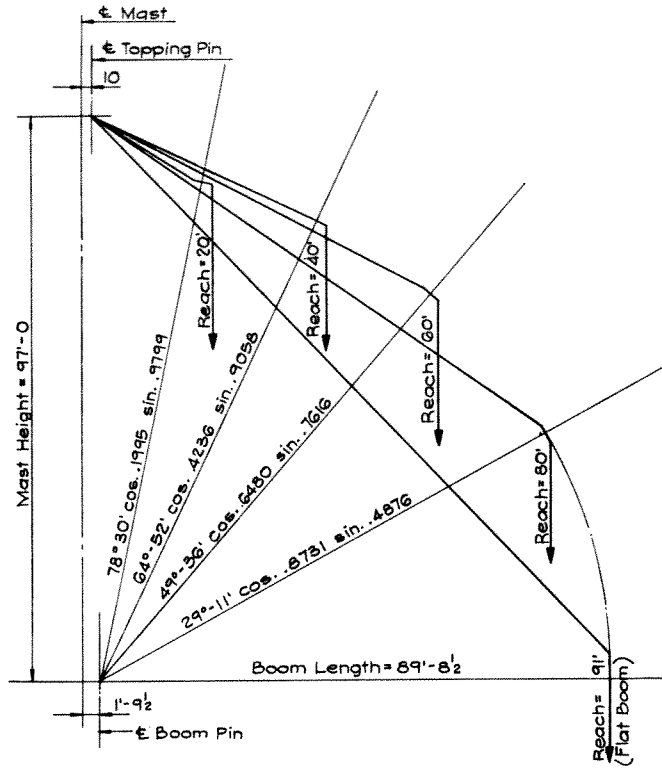
- (a) The total cross-sectional area, A , of the four chord angles or, if the width-thickness ratios exceed the limits of Sect. D1.6, the total effective area, $4A_e$.
- (b) The moments of inertia of the boom at the prismatic section, I_0 , and at the minimum section at the end of the boom (considering only chord angles), I_1 .
- (c) The ratio h/L , where h is the length of the prismatic portion of the boom and L is the boom length.
- (d) The average moment of inertia $I_{av} = \gamma I_0$.
- (e) The l/r for the boom as a whole based on I_{av} .

3. Boom Tip and Heel Geometry

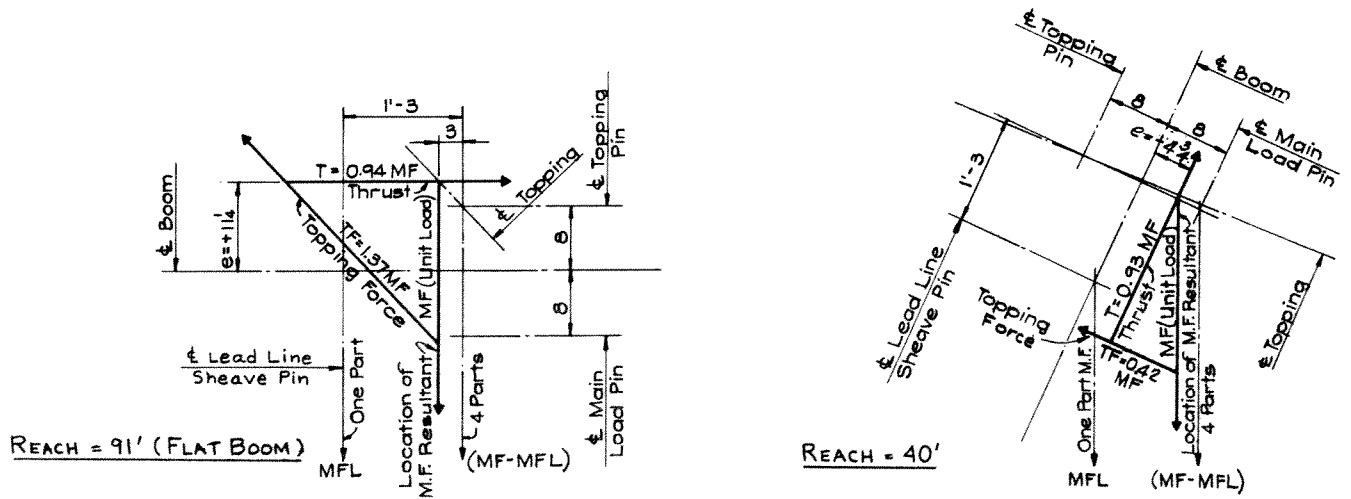
Lay out the boom tip and the boom heel to scale. On these details locate the topping falls pin, the main load falls pin, the main falls lead sheave pin and the boom heel pin.

4. Determine The External Loads On The Boom

- (a) Make a Boom Position Diagram (see Fig. 4a)
Determine the inclinations, relative to the centerline of the boom, of the topping and main falls for various increments of reach.



(a) Boom position diagram



(b) Unit load polygons

Figure 4

Locate the centerline of the boom for each position. Note that the resultant of the topping force (TF) and the main falls force (MF) does not act at the centerline of the boom.

(b) Make a Unit Load Polygon for Each Increment of Reach (see Fig. 4b).

Scale the unit load polygon (or calculate) to determine the coefficients which, when multiplied by the main falls force (MF) will give the value of the thrust (T) and topping force (TF).

The weight of main falls is always a vertical load. However, the line of action of the main falls does not act through the main falls pin, since the lead line generally passes over a sheave some distance down the boom from the tip. This means, then, that if there were 5 parts in the main falls, 4 parts act through the pin while one acts through the lead sheave pin. For a 5 part fall, the resultant acts at a point one-fifth of the horizontal distance between the main falls pin and the lead sheave pin.

(c) Make a Free Body Diagram of the Boom (see Fig. 5).

To determine the magnitude of the thrust, use a free body diagram such as Fig. 5. Since the thrust in the boom varies continuously due to the axial component of the dead load of the boom (w_{DL}), it is important to compute the thrust at each section being investigated. At a distance x feet from the boom heel pin, the thrust is

equal to the sum of the components parallel to the axis of the boom of the topping force TF, the main falls load MF, the concentrated boom tip dead load DL_c , and the uniform dead load above the section, $(L - x)w_{DL}$.

5. Determine the Moments on the Boom

(a) The moment at the boom tip is found by drawing a free body diagram of the boom tip (see Fig. 6) and summing the moments due to the eccentric topping load and the eccentric resultant of the main falls load about any convenient point. The dead load of the miscellaneous plates at the boom tip is assumed to have no effect and for the example illustrated by Fig. 6 is applied at point A, the selected convenient point about which moments were taken.

(b) The only force acting at the boom heel pin which may produce moment is the total thrust, if the pin is eccentric to the boom centerline.

(c) The moment due to uniform dead load is

$$M_{DL} = \frac{w_{DL}L^2}{8} \cos \theta.$$

(d) The moment at a section of the boom at distance x from the heel pin is $M_{DLx} = (x/L) M_{DL}$.

Note: Care with regard to use of a sign convention for moment and stress is important.

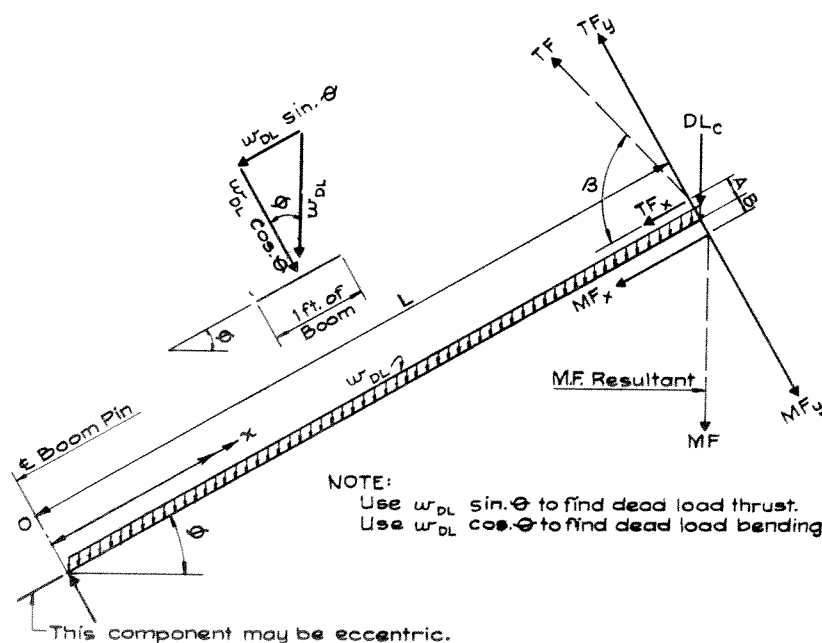


Fig. 5. Free body diagram of boom

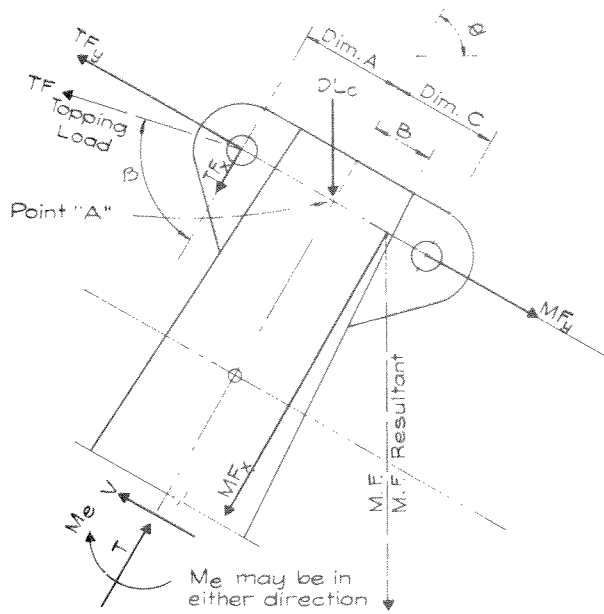


Fig. 6. Free body diagram of boom tip

6. Check the Interaction Equations

For guy derricks, the last (y-axis) terms of Interaction Formulas (D1.5-1a) and (D1.5-1b) are dropped, since neither slewing nor wind forces are considered for the reasons discussed previously. Formula (D1.5-1a) then takes the following form:

$$\frac{f_a D}{F_a} + \frac{C_{mx1} f_{bx1} D}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{mx2} f_{bx2} D}{(1 - f_a/F'_{ex}) F_{bx}} \leq 1.0$$

The first term in the interaction formula is the ratio of the computed axial stress under static loading times the dynamic factor divided by the allowable axial stress. The second term considers bending stress due to load normal to the x-axis of the boom with no end moments. Therefore $C_{mx1} = 1.0$. The third term considers bending stresses due to end moments only. For the usual condition where the boom heel pin is on the centerline of the boom, $C_{mx2} = 0.6$.

The question of dynamic allowance was discussed in Section II (F). For normal operating conditions, use $D = 1.2$ in all terms. It must be emphasized that when determining the actual stresses, f_a , f_{bx1} , and f_{bx2} for use in the interaction equations, static loading is assumed with no increase in the load applied at the hook.

Since the bending stresses will vary along the length of the boom, the critical section is found by using the interaction formulas at several sections along the length of the boom. In the tapered portions of the boom, it is possible that the distance between lacing points will be less than in the parallel portion. In such cases, a value of F_{bx}

based on this lesser distance between lacing points should be used in checking the stresses at the section.

For guy derricks, Formula (D1.5-1b) takes the following form:

$$\frac{f_a D}{0.6 F_y} + \frac{f_{bx} D}{F_{bx}} \leq 1.0$$

Usually this equation governs only near the ends of the member; however, both interaction equations should be checked at each section investigated.

J. Stiffleg Derrick Boom Stresses

An investigation of the adequacy of a stiffleg derrick boom is similar to, but somewhat more elaborate than, an investigation of a guy derrick boom.

The introduction of a horizontal slewing or wind force at the boom tip requires that the complete interaction formulas be used:

$$\frac{f_a D}{F_a} + \frac{C_{mx1} f_{bx1} D}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{mx2} f_{bx2} D}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{my} f_{by}}{(1 - f_a/F'_{ey}) F_{by}} \leq 1.0 \quad (D1.5-1a)$$

and

$$\frac{f_a D}{0.6 F_y} + \frac{f_{bx} D}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (D1.5-1b)$$

The last term in each interaction formula is due to the lateral (slewing or wind) force at the boom tip.

From Section II (D), the horizontal slewing force H_s , to be applied at the boom tip is:

- (1) For booms making an angle of 60 degrees, or more, with the horizontal:

$$H_s = 0.02 (LL + DL_{BT})$$

- (2) For booms which make an angle of less than 60 degrees with the horizontal, H_s is equal to a percentage of $(LL + DL_{BT})$ which varies linearly from 2% when the boom is 60 degrees with the horizontal to 3% when the boom is flat.

From Section II (E), the horizontal wind force, H_w , to be applied at the boom tip is:

$$H_w = 0.02 LL + \frac{C_d P_w A_w}{2}$$

When slewing into the wind, the greater of either H_s or H_w is considered to act horizontally at the boom tip.

From Section II (F), the dynamic factor D is:

In the absence of slewing: $D = 1.2$

In the presence of slewing: $D = 1.03$

As in the case of guy derricks, $C_{mx1} = 1.0$ and $C_{mx2} = 0.6$ (when the boom heel pin is on the centerline). The value of C_{my} is always 0.85.

The critical section for a stiffleg derrick boom is found in the same manner as for a guy derrick boom, by applying the interaction formulas at various sections along the boom length.

The question of what value of K should be used for Kl/r , in the case of derrick booms which are deflected laterally by the horizontal force developed by slewing or wind, has been examined by Timoshenko and Gere*. It is shown that $K = 1$, since the resultant force at the boom tip acts in the plane defined by the mast and the boom. In derricks equipped with boom stays, K is slightly less than 1.0. But the gain in capacity is certainly not worth the sophisticated calculations involved.

The boom heel pin for a stiffleg derrick deserves special attention. The horizontal force at the boom tip (caused by the slewing or wind action) is resisted by a couple at the boom heel pin. Bearing on the pin plates and bending in the pin should reflect the effects of this couple.

K. Mast Stresses

Much of the preceding discussion of booms is applicable to masts. Mast properties, geometry, external loads, moments, and thrust are all required for design investigation. However, since masts must be vertical, dead load bending is not considered. Dead load of the mast above any section under consideration should be added as a concentric load.

Horizontal forces on masts are generated by the stifflegs, guys, boom heel pins, and mast seats. These forces produce moments which are maximum near the ends; therefore, the value of C_m used in interaction formula (D1.5-1a) is worth investigating. Referring to Sect. D1.5.1 of the Derrick Specification, for members not subject to transverse loading between their supports in the plane of bending, $C_m = 0.6 - 0.4 (M_1/M_2) \leq 0.4$. It is also noted that the ratio M_1/M_2 is positive when the member is bent in reverse curvature and negative when it is bent in single curvature. In the sample problem for a guy derrick mast (see Section VIII, Example C) the mast is bent in reverse curvature and the value of C_m is less than 0.6. However, reverse curvature is not necessarily the case. The mast design must satisfy both interaction formulas (D1.5-1a) and (D1.5-1b).

Finally, since usually there are no loads applied between the ends normal to the centerline of the mast, interaction formula (D1.5-1a) for the mast usually consists of only two terms:

$$\frac{f_a D}{F_a} + \frac{C_{mx2} f_{bx2} D}{(1 - f_a/F'_{ex}) F_{bx}} \leq 1.0$$

Occasionally the main fall leads and/or the topping fall leads enter the mast over a sheave so located that the line to the foot of the mast is approximately on the centerline of the mast. With the sheave in this position, a moment is induced in the mast equal to the vertical components of the forces on the sheave multiplied by the eccentricity of the sheave pin with respect to the mast centerline. Additional moment is produced in the mast by the horizontal component of the lead line. Compared to the end moments on the mast, these moments are relatively small. They should, however, be included in the interaction formula as separate terms concerned with x-axis bending. A value of 0.8 for C_m would be conservative.

To simplify and facilitate calculations, a guy derrick is analyzed as if it were supported by a single guy, known as the "effective guy", located in the plane of the boom. This single, imaginary guy, which does not exist in actuality, may be thought of as a "resultant" guy representing one, two, or three back guys which are primarily stressed by the live load and boom tip dead load. The orientation of the boom relative to the system of guys varies with the rotation of the derrick. Thus, the back guys which are stressed by the live load and boom tip dead load are different for each boom orientation. Therefore, guy derrick mast analysis requires that a study be made for each stiffleg (or sill) length with the boom at a number of radii.

The "effective guy distance" (L_{GE}) is the distance measured in a horizontal plane through the boom heel pin, from the center of rotation of the mast to the intersection of the plane and the guy. The determination of the "effective guy distance" is illustrated in Section VI (A); see Fig. 13.

III. DERRICK JIBS

Jibs are extensions of the boom. They serve to make available auxiliary falls, with one or two parts, which can be used to raise light loads more quickly than with the numerous parts of the main load falls. They also provide for handling light loads at greater reaches. Jibs come in many forms and are connected to the boom tip in a variety of ways. Figure 7 illustrates some typical varieties.

*Timoshenko & Gere, "The Theory of Elastic Stability", 2nd Edition, McGraw-Hill Book Co., New York, N.Y., 1961

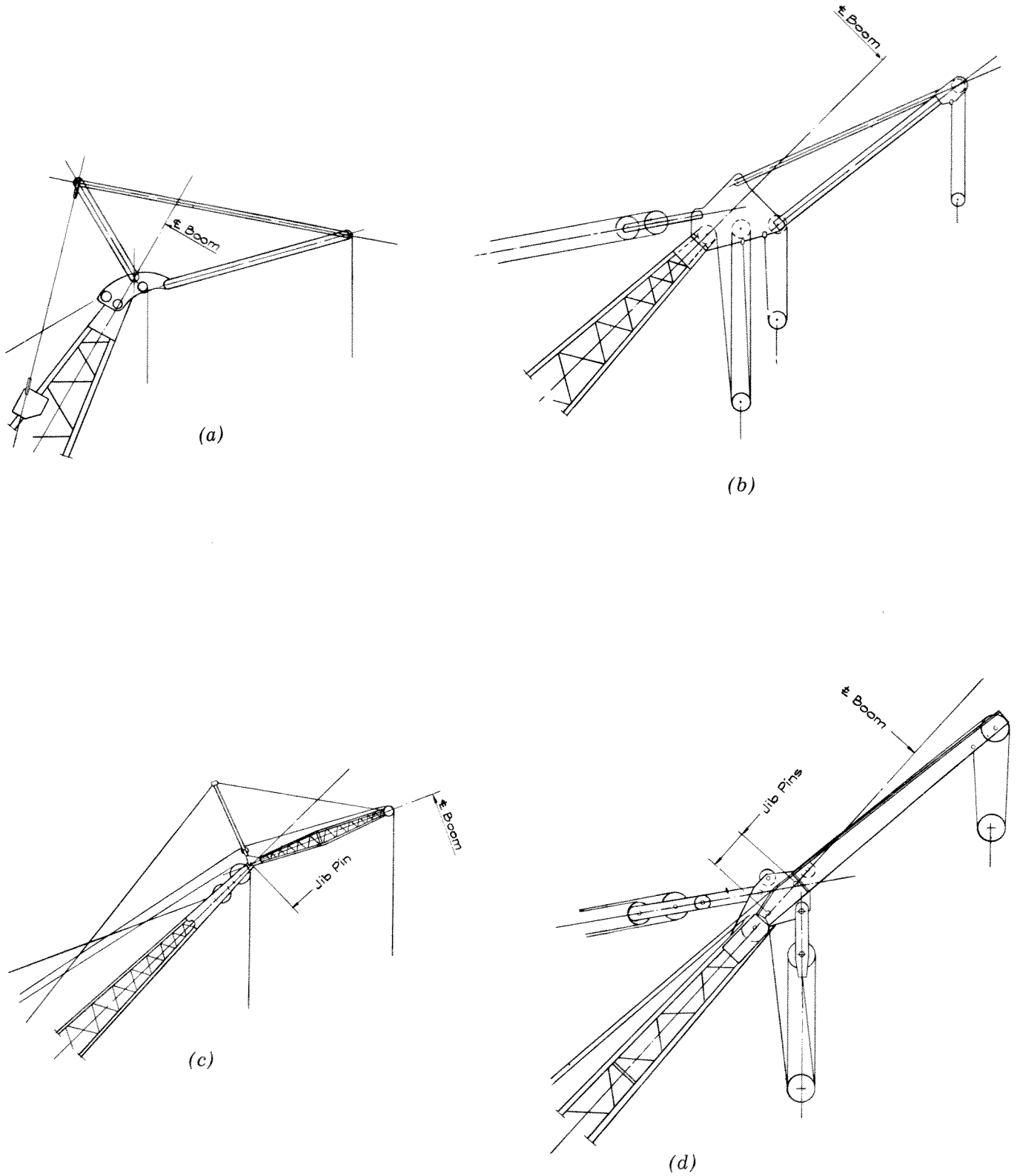


Fig. 7. Typical derrick jibs

Whenever the jib is on the boom, the capacity of the main falls is reduced by the dead weight of the jib and its tackle.

Capacities of jibs are controlled in one of two ways:

- (1) The capacity of the jib at any reach may not exceed the capacity of the main falls at that reach.
- (2) The jib may be considered an extension of the boom, but the analysis of its capacity must be based on the angle between the jib and the horizontal, rather than the angle between the main boom and the horizontal.

When the jib is connected to the boom tip by two pins, a moment is imposed on the boom tip.

When the jib is connected to the boom tip with a single pin and a gantry which is restrained by a backstay, there are vertical and horizontal loads at the boom tip. The connection of the backstay to the boom induces normal and axial loads on the boom.

It is difficult to make generalizations as to the use of jibs. But it is rare to use the jib and main falls simultaneously. For booms making an angle of more than 50 degrees to the horizontal, it would appear that the effects of the jib on the boom would be less severe than the effects of capacity loads on the main falls. Where the jib acts as an extension of the boom, a careful comparison of the effects on the boom of the loading due to capacity loads on the main falls acting alone and lighter loads on the jib acting alone should be made.

IV. STIFFLEGS

Stifflegs are usually pin connected structural members concentrically loaded either in tension due to tie-down or compression. Dead load bending may have some effect. If so, the interaction formulas with dynamic considerations must be used.

V. DERRICK SILLS

Derrick sills behave as beam-columns when the associated stiffleg is in tension. As will be noted in Figs. 8a and 8b, the path of the thrust (or tension) may have a large eccentricity in the vicinity of the mast. Where the mast casting is of the ball and socket variety, the point of application of the thrust is taken at the center of the ball. The connection between sill and footblock must be capable of transferring the shear due to dead load bending, as well as tension or compression.

It is interesting to note that there are footblock designs in which the footblock and sill are pin connected. In such a case, the sill is subjected to axial load only.

VI. WIRE ROPE

Wire rope should be in accordance with the American National Standards Institute ANSI B30.6 and/or SAE Standard J959. As noted in SAE J959, "The purpose of this SAE Standard is to set forth wire rope strength factors that have been proved by design and operating practice to be consistent with safety, economy, space, weight and other requirements . . ." SAE J959 Article 3.3 further states that "all forces shall be considered static as produced by the boom and suspended load without the effects of motion from lifting, lowering, swinging or traveling."

The wire rope strength factors in both ANSI B30.6-1.3 and SAE J959 are the same.

For designs in accordance with this Guide, the nominal breaking strength of each guy rope shall be no less than three (3) times the static load applied to the rope, considering the entire rope assembly as a unit. However, when considering the effect of the derrick on the supporting structure, the static load in any guy should be increased by a dynamic factor of 1.2. (See Sect. II (F), Dynamic Allowances.)

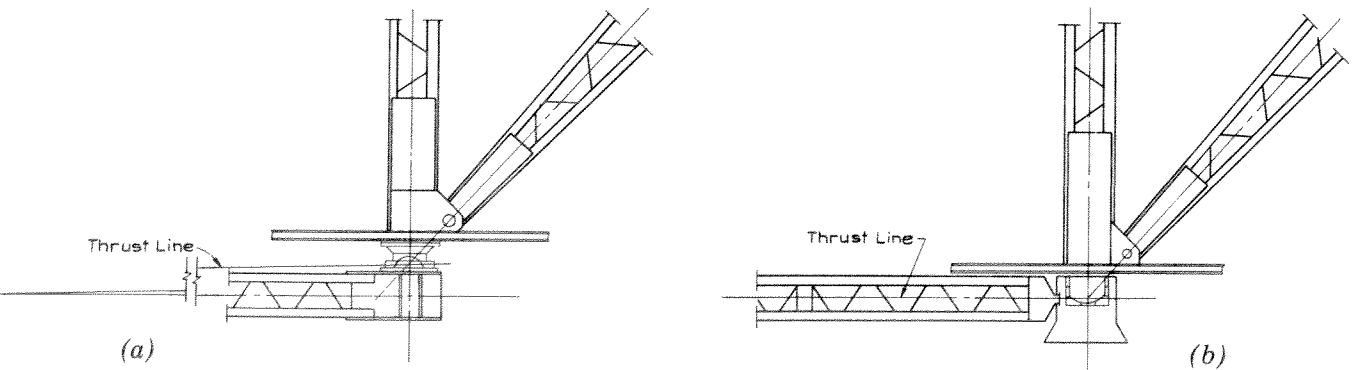


Fig. 8. Derrick sills

The nominal breaking strength requirements for boom hoist and main hoist ropes applied to the most heavily loaded rope in a system shall be no less than three and one half ($3\frac{1}{2}$) times the static load applied to the rope, with adequate consideration for frictional losses and approved reeving, considering the rope assembly as a unit.

A. Guys

Guys are wire ropes used to hold the mast against the overturning force generated by the action of the boom and its suspended load. Although it is ideal to have six or more guys equally spaced, every particular installation is contingent on several factors. These factors are the type of structure, such as bridge, tower, building, etc., the geometric arrangement and size of members in the structure, and the sequence of erection of the structure. These factors will dictate the number, size, and location of guys for a particular installation.

Because a guy derrick installation is a deceptively complicated structure, the method outlined for a guy analysis will be approximate, but on the conservative side. The method suggested has been analytically derived with the assistance of computer-aided studies and observations of actual derrick installations and tests.

Before outlining the method of guy analysis, some guidelines for the analysis must be established. When a guy derrick is erected, the derrick is "tuned" by having the guys pretensioned in order to stabilize the mast tip from excessive movements transverse to the boom line as a load is handled. The erector will attempt to equalize the pretension in the guys, but exact pretension of the guys is not practical nor necessary. When a derrick picks a load, some of the guys, which will be called "dead guys", will tend to slacken, while the others will participate in the support of the load and will be the working guys, which will be called "live guys". Reference will be made to dead guys and live guys in the analysis, so a complete definition is as follows:

Live Guys -- are those guys that participate in carrying the load from the boom. Considering the boom as front, live guys are all guys on or behind a line through the mast tip normal to the vertical plane of the boom (Fig. 9.)

Dead Guys -- are those guys that are not participating in carrying the load from the boom. Considering the boom as front, dead guys are all guys in front of a line through the mast tip normal to the vertical plane of the boom (Fig. 9.)

As has been noted, the mast tip is prevented from excessive transverse movement during the handling of the load. Therefore, the analytical model for stresses in guys may be based on the criterion that the mast tip is guided in the direction of loading. The small inaccuracy from this assumption will be compensated for by use of a very small correction factor. This approach is more realistic than considering the mast tip to be elastically restrained laterally.

Dead guys will contribute a dead load component to the total force induced in the live guys; however, dead load of side guys have a diminishing effect on live guy forces as they approach 90° to the boom line. Side guys contribute a dead load component to the vertical load applied to the top of the mast, while providing stabilization for the mast tip. Likewise, the live guys which contribute to support of the load will be less effective as they approach 90° to the boom line. Effective sectors for the location of dead and live guys are illustrated in Fig. 10; the use of criteria illustrated in Fig. 10 in the analysis of guy stresses will be conservative.

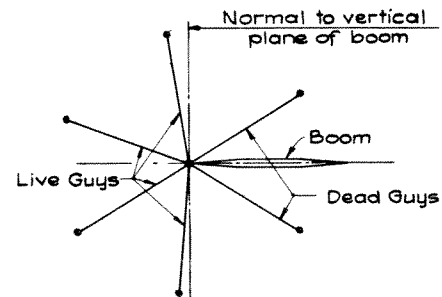


Fig. 9. Positions of live and dead guys

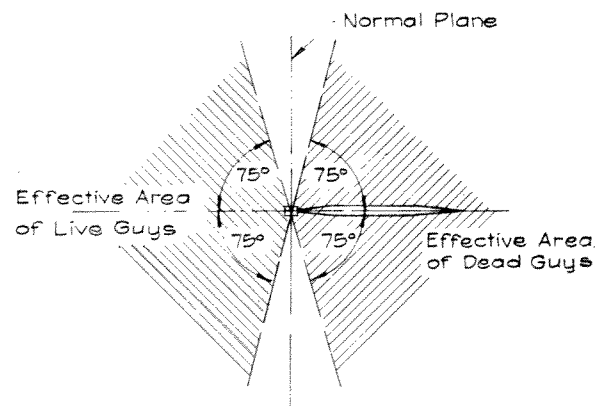


Fig. 10. Effective areas (sectors) of live and dead guys

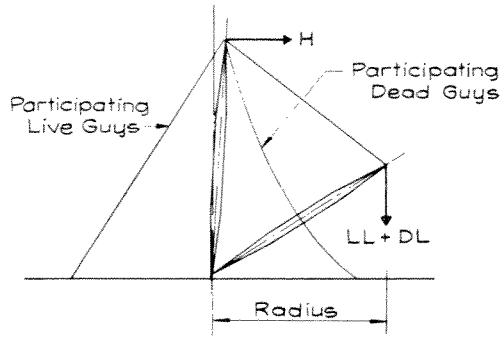


Figure 11

All guys in the effective area for dead guys should be considered as contributing to the total horizontal force at the top of the mast (H) for live guy design. All guys (up to 4 in number) in the effective area for live guys should be considered as acting to support the total force H, i.e., up to four guys may be in the effective area for live guys and accordingly considered in the design.

Referring to Fig. 11, the participating live guys will be designed for force $H = H_G + TF_H$, where:

H = The total resultant horizontal force acting at the top of the mast which will be considered in the analysis of the live guys and the mast, kips.

H_G = The horizontal force from the dead guys in the dead guy effective area, kips.
Where dead guys are wire rope:

1" ϕ and under: $H_G = 0.006N (L_{DG} - 12)$

Over 1" ϕ : $H_G = 0.012N (L_{DG} - 12)$

N = Number of effective dead guys

L_{DG} = Average horizontal distance of all the dead guys, based on the horizontal distances from centerline of the mast to the intersection of a horizontal plane through the boom foot pin, projected onto a vertical plane through the boom, ft

TF_H = The horizontal force at the top of the mast due to the total vertical load at the boom tip, kips

$$= \frac{(LL + DL_{BT}) R}{L_m}$$

DL_{BT} = Total dead load at the boom tip, kips

$$= DL_c + DL_{MF} + w_{DL} (L/2)$$

LL = Lifted load or live load, kips

L_m = Height of mast, ft

R = Reach, ft

Guy Stress Formulas

Where only one guy is in the Effective Live Guy Area:

$$G_1 = HL_1 / x_1$$

Where two guys are in the Effective Live Guy Area:

$$G_1 = \frac{C(H)}{\frac{x_1}{L_1} + \frac{L_1^2}{A_1 x_1} \left[\frac{A_2 x_2^2}{L_2^3} \right]}$$

$$G_2 = \frac{C(H)}{\frac{x_2}{L_2} + \frac{L_2^2}{A_2 x_2} \left[\frac{A_1 x_1^2}{L_1^3} \right]}$$

Where three guys are in the Effective Live Guy Area:

$$G_1 = \frac{C(H)}{\frac{x_1}{L_1} + \frac{L_1^2}{A_1 x_1} \left[\frac{A_2 x_2^2}{L_2^3} + \frac{A_3 x_3^2}{L_3^3} \right]}$$

$$G_2 = \frac{C(H)}{\frac{x_2}{L_2} + \frac{L_2^2}{A_2 x_2} \left[\frac{A_1 x_1^2}{L_1^3} + \frac{A_3 x_3^2}{L_3^3} \right]}$$

$$G_3 = \frac{C(H)}{\frac{x_3}{L_3} + \frac{L_3^2}{A_3 x_3} \left[\frac{A_1 x_1^2}{L_1^3} + \frac{A_2 x_2^2}{L_2^3} \right]}$$

Where four guys are in the Effective Live Guy Area:

$$G_1 = \frac{C(H)}{\frac{x_1}{L_1} + \frac{L_1^2}{A_1 x_1} \left[\frac{A_2 x_2^2}{L_2^3} + \frac{A_3 x_3^2}{L_3^3} + \frac{A_4 x_4^2}{L_4^3} \right]}$$

$$G_2 = \frac{C(H)}{\frac{x_2}{L_2} + \frac{L_2^2}{A_2 x_2} \left[\frac{A_1 x_1^2}{L_1^3} + \frac{A_3 x_3^2}{L_3^3} + \frac{A_4 x_4^2}{L_4^3} \right]}$$

$$G_3 = \frac{C(H)}{\frac{x_3}{L_3} + \frac{L_3^2}{A_3 x_3} \left[\frac{A_1 x_1^2}{L_1^3} + \frac{A_2 x_2^2}{L_2^3} + \frac{A_4 x_4^2}{L_4^3} \right]}$$

$$G_4 = \frac{C(H)}{\frac{x_4}{L_4} + \frac{L_4^2}{A_4 x_4} \left[\frac{A_1 x_1^2}{L_1^3} + \frac{A_2 x_2^2}{L_2^3} + \frac{A_3 x_3^2}{L_3^3} \right]}$$

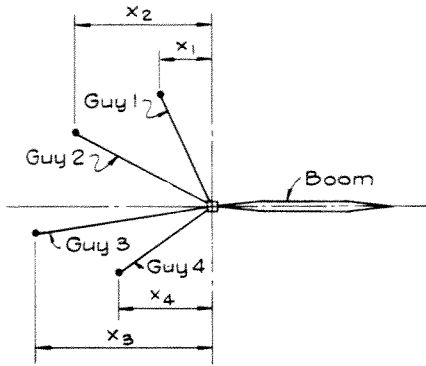


Figure 12

In the above formulas:

G_1, G_2, G_3, G_4 = Total live guy force along the noted guy (kips). See Fig. 12.

L_1, L_2, L_3, L_4 = Slope lengths of the noted live guys (ft)

x_1, x_2, x_3, x_4 = Length of the x ordinates of the noted live guys as seen in plan (ft). See Fig. 12.

A_1, A_2, A_3, A_4 = Cross-sectional area of guys 1, 2, 3 and 4 respectively (in.^2)

C = A correction factor, applicable in multiple live guy cases, to provide for the fact that the mast tip is not perfectly guided in the vertical plane of the boom as the load is lifted. C may be taken as 1.05 in all cases.

Note that the allowable stress in the guy must not be greater than one-third (1/3) the nominal breaking strength of the wire rope.

When the guy orientation is such that two live guys will clearly take a major portion of the reaction, the following simple and quick, as well as conservative, method is recommended for approximate determination of guy stress. If the guy stresses obtained through this computation exceed the allowable, then the more precise analysis should be followed.

Figure 13 shows a plan and elevation of a guy derrick. Note that the guy shown in elevation is the "effective guy", the single imaginary guy which is in the plane of the boom. The vertical component of the tensile force in the effective guy (G_{EV}) can

be found quickly by taking the summation of moments around the intersection of the centerlines of the boom and the mast. The vertical components of the two actual "live guys" are based on the ratio a/b . Having determined the value of G_{EV} , the force in the effective guy, the stress in each live guy is easily found graphically.

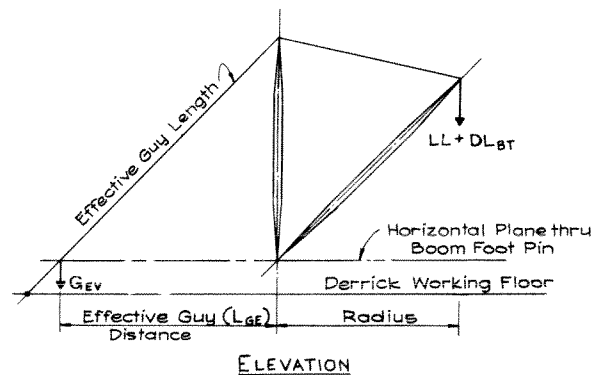
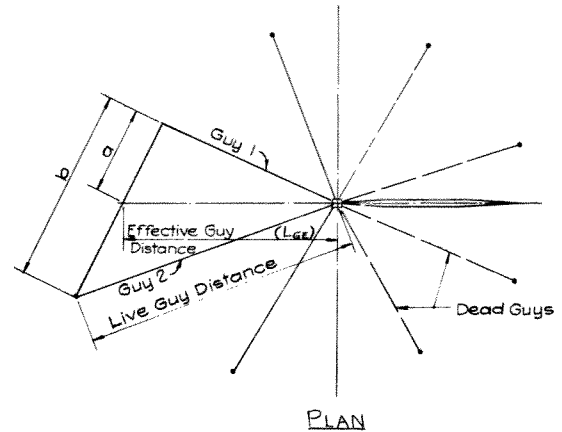


Fig. 13. Graphical determination of live guy stresses

VII. SHEAVES

ANSI B30.6-1.3.5(f) requires that boom and hoisting sheaves shall have pitch diameters not less than eighteen (18) times the nominal diameter of the rope used. The following ratios of sheave-to-rope diameter are recommended:

Load hoisting sheaves, on boom:	18.0 to 1
Load hoisting sheaves, on traveling blocks:	16.0 to 1
Boom hoisting sheaves:	15.0 to 1

VIII. DESIGN EXAMPLES

A. Guy Derrick Boom (Fig. 14)

Given:

Capacity load in flat position: 12 kips

Chords: 4 L $3\frac{1}{2} \times 3 \times 5/16$; $F_y = 45$ ksi

$DL_{MF} = 600$ lbs = 0.6 kips

$DL_C = 400$ lbs = 0.4 kips

$w_{DL} = 60.5$ lbs/ft = 0.0605 kips/ft

$L = 89' - 8\frac{1}{2}''$

c. to c. staggered lacing points = 20''

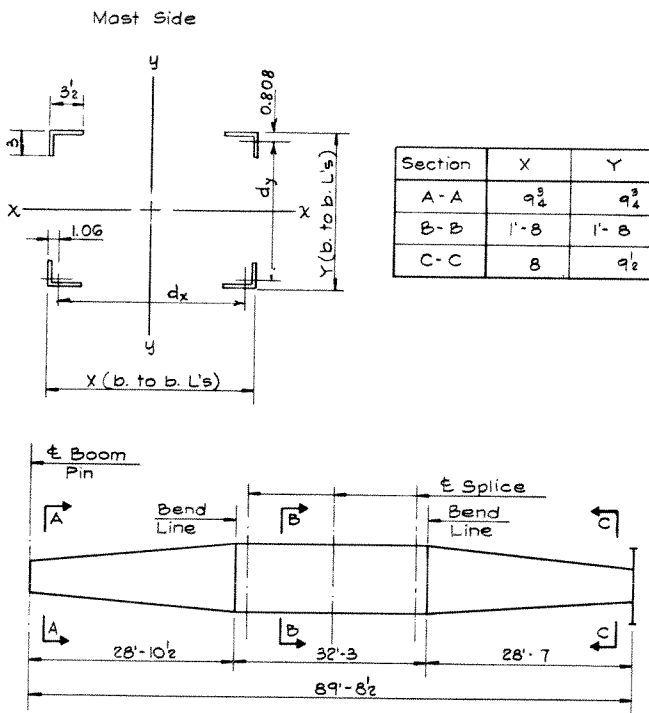


Fig. 14. Guy derrick boom dimensions

Problem:

Investigate the boom stresses at a distance 58.5 ft from the boom pin (the point of maximum moment) under capacity load in the flat position.

Solution:

1. Check width-thickness ratio (Sect. D1.6.1.2):

$$\frac{b}{t} = \frac{3.5}{5/16} = 11.2 < \frac{76.0}{\sqrt{45}} = 11.3 \quad \text{o.k.}$$

Area of one $3\frac{1}{2} \times 3 \times 5/16$ chord: $A_c = 1.93$ in.²

Total cross-sectional area: $A = 4(1.93) = 7.72$ in.²

2. Find actual moments of inertia:

$$I = A (d/2)^2$$

At Sect. A-A:

$$I_{Ax} = 7.72 (9.75/2 - 0.808)^2 = 128 \text{ in.}^4$$

$$I_{Ay} = 7.72 (9.75/2 - 1.06)^2 = 112 \text{ in.}^4$$

$$d_y = 8.13 \text{ in.}, \quad d_x = 7.63 \text{ in.}$$

At Sect. B-B:

$$I_{Bx} = 7.72 (20/2 - 0.808)^2 = 652 \text{ in.}^4$$

$$I_{By} = 7.72 (20/2 - 1.06)^2 = 617 \text{ in.}^4$$

$$d_y = 18.4 \text{ in.}, \quad d_x = 17.9 \text{ in.}$$

At Sect. C-C:

$$I_{Cx} = 7.72 (9.5/2 - 0.808)^2 = 120 \text{ in.}^4$$

$$I_{Cy} = 7.72 (8/2 - 1.06)^2 = 66.7 \text{ in.}^4$$

$$d_y = 7.88 \text{ in.}, \quad d_x = 5.88 \text{ in.}$$

3. Find average moments of inertia and radius of gyration:

Buckling out of vertical (y-y) plane:

$$I_0 = I_{Bx} = 652 \text{ in.}^4$$

$$I_1 = I_{Cx} = 120 \text{ in.}^4 < I_{Ax} = 128 \text{ in.}^4$$

$$\frac{I_1}{I_0} = \frac{120}{652} = 0.184; \quad \frac{h}{L} = \frac{32.25}{89.7} = 0.360$$

$$\gamma \text{ (from Fig. 1 or Table D1.4.4)} = 0.840$$

$$I_{av(x)} = \gamma I_0 = 0.840 (652) = 548 \text{ in.}^4$$

$$r_x = \sqrt{I_{av(x)}/A} = \sqrt{548/7.72} = 8.43 \text{ in.}$$

Buckling out of transverse (x-x) plane:

$$I_0 = I_{By} = 617 \text{ in.}^4$$

$$I_1 = I_{Cy} = 66.7 \text{ in.}^4 < I_{Ay} = 113 \text{ in.}^4$$

$$\frac{I_1}{I_0} = \frac{66.7}{617} = 0.108; \quad \frac{h}{L} = 0.360$$

$$\gamma \text{ (from Fig. 1 or Table D1.4.4)} = 0.795$$

$$I_{av(y)} = \gamma I_0 = 0.795 (617) = 491 \text{ in.}^4$$

$$r_y = \sqrt{491/7.72} = 7.98 \text{ in.}$$

4. Determine allowable stresses:

Axial stresses:

$$\frac{l}{r_x} = \frac{89.7 (12)}{8.43} = 128$$

$$\frac{l}{r_y} = \frac{89.7 (12)}{7.98} = 135$$

From Table 2, AISC Spec. Appendix A:

$$F'_{ex} = 9.11 \text{ ksi}; \quad F'_{ey} = 8.19 \text{ ksi}$$

From Table 1-45, AISC Spec. Appendix A:

$$F_a = 8.19 \text{ ksi}$$

Bending stresses:

Allowable bending stress in the boom, F_b , is equal to the allowable compressive stress in one chord between lacing points, F_a (chord).

l (c. to c. staggered lacing points) = 20 in.

r_y (for L $3\frac{1}{2} \times 3 \times 5/16$) = 0.905 in.*

$$\frac{l}{r_y} = \frac{20}{0.905} = 22.1$$

From Table 1-45, AISC Spec. Appendix A:

$$F_a \text{ (chord)} = 25.38 \text{ ksi} \therefore F_b = 25.38 \text{ ksi}$$

5. Boom analysis:

Main falls load:

$$MF = LL + DL_{MF} = 12 + 0.6 = 12.6 \text{ kips}$$

$$MF_x = 0$$

$$MF_y = MF = 12.6 \text{ kips}$$

Topping force (see Unit Load Polygon, Fig. 4b (flat boom) and Fig. 15):

$$\begin{aligned} TF_y &= LL + DL_{MF} + DL_c + w_{DL} (L/2) \\ &= 12 + 0.6 + 0.4 + (0.0605) (89.7/2) \\ &= 15.7 \text{ kips} \end{aligned}$$

$$TF = 1.37 TF_y = 1.37 (15.7) = 21.5 \text{ kips}$$

$$TF_x = 0.94 TF_y = 0.94 (15.7) = 14.8 \text{ kips}$$

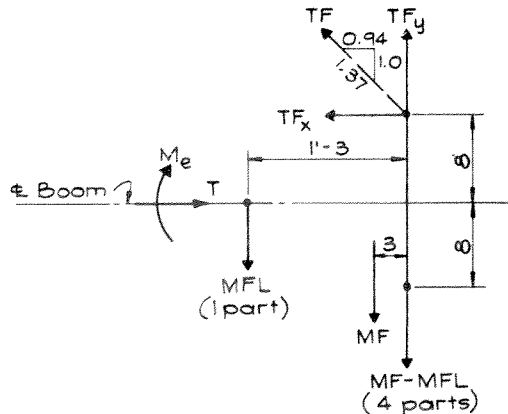


Figure 15

*If lacing is not staggered, r_z of the chord angle should be used in the computation of l/r .

Axial stress:

$$\begin{aligned} T &= MF_x + TF_x + MFL \text{ (1 of 5 parts)} \\ &= 0 + 14.8 + (12.6/5) = 17.3 \text{ kips} \end{aligned}$$

$$f_a = T/A = 17.3/7.72 = 2.24 \text{ ksi}$$

End moment at boom tip (due to eccentricity):

$$\begin{aligned} M_c &= 8TF_x + 3MF = 8 (14.8) + 3 (12.6) \\ &= 156 \text{ kip-in. (tension in bottom chords)} \end{aligned}$$

Moment at 58.5' from boom pin due to M_e :

$$M_{ex} = M_e (x/L) = 156 (58.5/89.7)$$

$$= 102 \text{ kip-in. (tension in bottom chords)}$$

Moment at 58.5' from boom pin due to w_{DL} :

$$\begin{aligned} M_{DLx} &= (w_{DL}x/2) (L - x) \\ &= \frac{0.0605 (58.5) (89.7 - 58.5)}{2} \times 12 \\ &= 663 \text{ kip-in. (tension in bottom chords)} \end{aligned}$$

Bending stress:

Area of one chord (L $3\frac{1}{2} \times 3 \times 5/16$)

$$= A_c = 1.93 \text{ in.}^2$$

f_{bx1} = bending at x due to M_{DLx}

$$= \frac{M_{DLx}}{2d A_c} = \frac{663}{2 (18.4) (1.93)} = 9.33 \text{ ksi}$$

f_{bx2} = bending at x due to M_e

$$= \frac{M_{ex}}{2d A_c} = \frac{102}{2 (18.4) (1.93)} = 1.44 \text{ ksi}$$

Interaction Formulas (y-axis terms omitted):

$$C_{mx1} = 1.0; \quad C_{mx2} = 0.6; \quad D = 1.2$$

Formula (D1.5-1a):

$$\frac{f_a D}{F_a} + \frac{C_{mx1} f_{bx1} D}{(1 - f_a/F'_{ex}) F_{bx}} + \frac{C_{mx2} f_{bx2} D}{(1 - f_a/F'_{ex}) F_{bx}} \leq 1.0$$

$$\frac{(2.24) (1.2)}{8.19} + \frac{(1.0) (9.33) (1.2)}{\left(1 - \frac{2.24}{9.11}\right) (25.38)} +$$

$$\frac{(0.6) (1.44) (1.2)}{\left(1 - \frac{2.24}{9.11}\right) (25.38)}$$

$$= 0.328 + 0.585 + 0.054 = 0.967 < 1.0 \text{ o.k.}$$

Formula (D1.5-1b):

$$\frac{f_a D}{0.60 F_y} + \frac{f_{bx} D}{F_{bx}} \leq 1.0$$

$$\frac{(2.24)(1.2)}{0.60(45)} + \frac{(9.33 + 1.44)(1.2)}{25.38} = 0.100 + 0.509 = 0.609 < 1.0 \text{ o.k.}$$

B. Guy Derrick Boom (Deflection Analysis)

Given:

Same data as Example A. See Figs. 14 and 16.

Problem:

Same as Example A.

Solution:

In lieu of amplifying the calculated bending stress when using the interaction formula, the maximum deflection caused by the applied moments will be multiplied by the thrust to obtain the actual addition in bending stress produced by the eccentricity created by the applied moments.

1. Compute the following properties as in

Example A:

Boom area $A = 7.72$ in.

$I_{Bx} = 652$ in.⁴ (= I_x at 58.5 ft)

$I_{Ax} = 548$ in.⁴

$l/r_x = 128$; $F_a = F_{ex} = 9.11$ ksi

l/r_y (chord) = 22.1; $F_b = 25.38$ ksi

2. Compute TF_y , TF_x , T , and M_e as in Example A:

$TF_y = 15.7$ kips

$TF_x = 14.8$ kips

$T = 17.3$ kips

$M_e = 156$ kip-in.

3. Deflection where $x = 58.5$ ft:

$$\delta_{DL} = \frac{wDLx}{24EI} (L^3 - 2Lx^2 + x^3)$$

$$= \frac{(0.0605)(58.5)}{24(29,000)(548)} \times$$

$$[(89.7)^3 - 2(89.7)(58.5)^2 + (58.5)^3] (12)^3 = 4.94 \text{ in. (down)}$$

$$\delta_{Me} = \frac{M_e x}{6EIL} (L^2 - x^2)$$

$$= \frac{156(58.5)}{6(29000)(548)(89.7)} \times$$

$$[(89.7)^2 - (58.5)^2] (12)^2 = 0.710 \text{ in. (down)}$$

The maximum deflection produced by the applied moments must be amplified by the ratio of applied thrust to the Euler load P_e (not P'_e):

$$\delta_x = \frac{(\delta_{DL} + \delta_{Me})}{1 - \frac{T}{P_e}}$$

where

$$P_e = F'_{ex} A \left(\frac{23}{12} \right) = \frac{(9.11)(7.72)(23)}{12} = 135$$

Then,

$$\delta_x = \frac{(4.94 + 0.710)}{1 - \frac{17.3}{135}} = 6.48 \text{ in.}$$

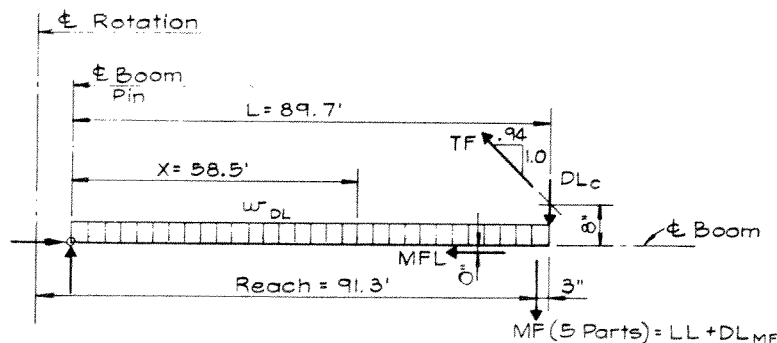


Figure 16

4. Moments where $x = 58.5$ ft:

$$\delta_x T = (6.48) (17.3) = 113 \text{ kip-in.}$$

$$M_{DLx} = \frac{WDLx}{2} (L - x)$$

$$= \frac{(0.0605) (58.5) (89.7 - 58.5) (12)}{2}$$

$$= 663 \text{ kip-in.}$$

$$M_{ex} = M_e (x/L) = 157 (58.5/89.7) = 102 \text{ kip-in.}$$

Total moment:

$$M_x = \delta_x T + M_{DLx} + M_{ex}$$

$$= 113 + 663 + 102 = 878 \text{ kip-in.}$$

5. Stresses where $x = 58.5$ ft:

Actual stress:

$$f_x = f_a + f_{bx} = \frac{T}{A} + \frac{M_x}{2d A_c}$$

$$= \frac{17.3}{7.72} + \frac{878}{2 (18.4) (1.93)}$$

$$= 2.24 + 12.4 = 14.8 \text{ ksi}$$

Using D (dynamic factor) = 1.2:

$$f_x = (14.8) (1.2)$$

$$= 17.8 \text{ ksi} < F_b = 25.38 \text{ ksi o.k.}$$

6. Check Interaction Equation:

Use Interaction Formula (D1.5-1a) without amplification factors, since an amplification factor was used in the computation of δ_x and is reflected in the computed value of f_{bx} .

$$\frac{f_a D}{F_a} + \frac{f_{bx} D}{F_b} \leq 1.0$$

$$\frac{2.24 (1.2)}{9.11} + \frac{12.36 (1.2)}{25.38} = 0.879 < 1.0 \text{ o.k.}$$

Note: The value 0.879 indicates a load capacity approximately 10% greater than indicated by the Interaction Formula method in Example A.

C. Guy Derrick Mast (Fig. 17)

Given:

$$L = 99.0 \text{ ft}$$

$$\text{Chords: } 4L \ 3\frac{1}{2} \times 3 \times 3/8; F_y = 45 \text{ ksi}$$

c. to c. staggered lacing points:

$$2' - 1\frac{3}{4}'' \text{ max. at end section}$$

$$1' - 11\frac{1}{2}'' \text{ max. at center section}$$

$$\text{Effective guy length: } L_{GE} = 40' - 0''$$

$$\text{Dead guys: Two, } 1'' \text{ } \phi \text{ wire rope, with average horizontal distance } L_{DG} = 46' - 0''$$

$$w_m = 84 \text{ lbs/ft} = 0.084 \text{ kips/ft}$$

$$\text{Boom: Same as Example A; } L = 89' - 8\frac{3}{4}''$$

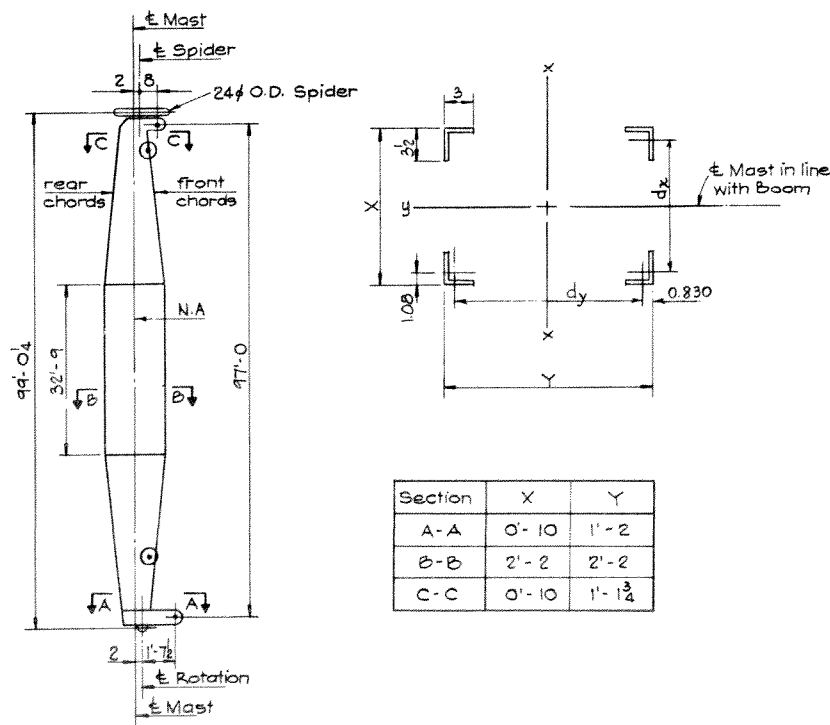


Fig. 17. Guy derrick mast dimensions

Problem:

Investigate mast for 6 ton load capacity at 91' - 4" reach (flat boom).

Solution:

1. Check width-thickness ratios (Sect. D1.6.1.2):

$$\frac{b}{t} = \frac{3.5}{3/8} = 9.33 < \frac{76.0}{45} = 11.3 \text{ o.k.}$$

$$\text{Area of one chord (L } 3\frac{1}{2} \times 3 \times 3/8) = A_c = 2.30 \text{ in.}^2$$

$$\text{Total cross-sectional area} = A = 4(2.30) = 9.20 \text{ in.}^2$$

2. Find actual moments of inertia:

$$I = A (d/2)^2$$

At Sect. A-A:

$$I_{Ax} = 9.20 (14/2 - 0.83)^2 = 350 \text{ in.}^4$$

$$I_{Ay} = 9.20 (10/2 - 1.08)^2 = 141 \text{ in.}^4$$

$$d_y = 12.3 \text{ in.}; d_x = 7.84 \text{ in.}$$

At Sect. B-B:

$$I_{Bx} = 9.20 (26/2 - 0.83)^2 = 1,363 \text{ in.}^4$$

$$I_{By} = 9.20 (26/2 - 1.08)^2 = 1,307 \text{ in.}^4$$

$$d_y = 24.3 \text{ in.}; d_x = 23.8 \text{ in.}$$

At Sect. C-C:

$$I_{Cx} = 9.20 (13.75/2 - 0.83)^2 = 336 \text{ in.}^4$$

$$I_{Cy} = 9.20 (10/2 - 1.08)^2 = 141 \text{ in.}^4$$

$$d_y = 12.1 \text{ in.}; d_x = 7.84 \text{ in.}$$

3. Find average moments of inertia and radius of gyration:

Buckling out of vertical (y-y) plane:

$$I_0 = I_{Bx} = 1,363 \text{ in.}^4$$

$$I_1 = I_{Cx} = 336 \text{ in.}^4 < I_{Ax} = 350 \text{ in.}^4$$

$$\frac{I_1}{I_0} = \frac{336}{1,363} = 0.247; \frac{h}{L} = \frac{32.8}{99.0} = 0.331$$

$$\gamma \text{ (interpolated from Fig. 1 or Table D1.4.4)} = 0.849$$

$$I_{av(x)} = \gamma I_0 = 0.849 (1,360) = 1,150 \text{ in.}^4$$

$$r_x = \sqrt{I_{av(x)} / A} = \sqrt{1,150 / 9.20} = 11.2 \text{ in.}$$

Buckling out of transverse (x-x) plane:

$$I_0 = I_{By} = 1,310 \text{ in.}^4$$

$$I_1 = I_{Ay} = I_{Cy} = 141 \text{ in.}^4$$

$$\frac{I_1}{I_0} = \frac{141}{1,310} = 0.108; \frac{h}{L} = 0.331$$

$$\gamma \text{ (interpolated from Fig. 1 or Table D1.4.4)} = 0.774$$

$$I_{av(y)} = \gamma I_0 = 0.774 (1,310) = 1,010 \text{ in.}^4$$

$$r_y = \sqrt{1,010 / 9.20} = 10.5 \text{ in.}$$

Determine allowable stresses:

Axial stress:

$$\frac{l}{r_x} = \frac{99.0 (12)}{11.2} = 106; \frac{l}{r_y} = \frac{99.0 (12)}{10.5} = 113$$

From Table 2, AISC Spec. Appendix A:

$$F'_{ex} = 13.29 \text{ ksi}; F'_{ey} = 11.69 \text{ ksi}$$

From Table 1-45, AISC Spec. Appendix A:

$$F_a = 11.69 \text{ ksi}$$

Bending stress (max. allowable compressive stress in one chord):

$$l = \text{max. c. to c. lacing points} = 25\frac{3}{4} \text{ in. (at end section)}$$

$$r_y \text{ (for L } 3\frac{1}{2} \times 3 \times 3/8) = 0.897 \text{ in.}$$

$$\frac{l}{r_y} = \frac{25.8}{0.897} = 28.8$$

From Table 1-45, AISC Spec. Appendix A:

$$F_a \text{ (chord)} = 24.73 \text{ ksi}$$

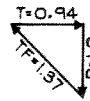
$$\therefore F_b = 24.73 \text{ ksi}$$

4. Mast analysis:

Make a free body diagram of the mast (boom flat, $\theta = 0$)* (see Fig. 18).

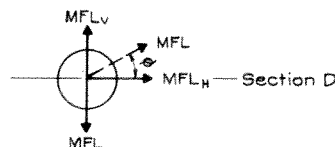
Forces on mast:

Topping force (TF):



$$\text{TF (mast)} = \text{TF (boom)} = 21.6 \text{ kips (see Example A for computation.)}$$

*When $\theta > 0$, the forces on the main falls (at Sect. D) are:



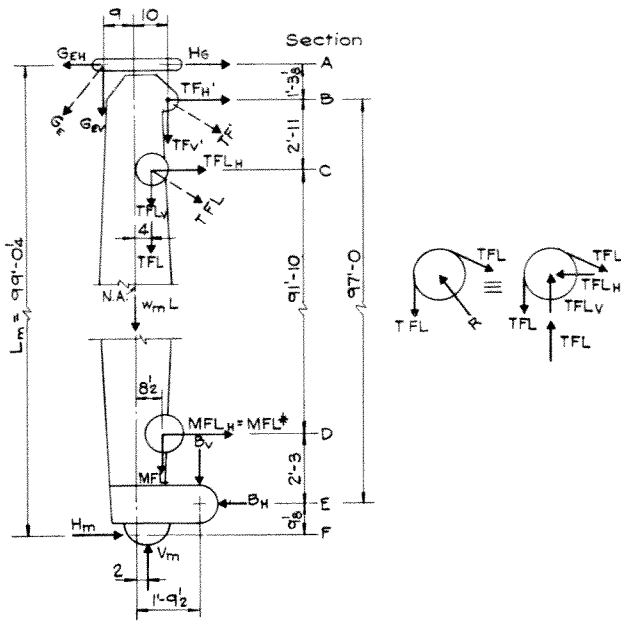


Fig. 18. Free body diagram of mast

Topping has 5 parts, including lead line.

$$TF' = (4/5) (21.6) = 17.3 \text{ kips}$$

$$TF'_V = 17.3 (1/1.37) = 12.6 \text{ kips}$$

$$TF'_H = 17.3 (0.94/1.37) = 11.9 \text{ kips}$$

$$TFL = (1/5) (21.6) = 4.32 \text{ kips}$$

$$TFL_V = 4.32 (1/1.37) = 3.15 \text{ kips}$$

$$TFL_H = 4.32 (0.94/1.37) = 2.96 \text{ kips}$$

Effect of dead guys:

Dead guys: 2 guys, 1" ϕ , with average horizontal distance $L_{DG} = 46 \text{ ft}$

$$H_G = 0.006N (L_{DG} - 12)$$

$$= 0.006 (2) (46 - 12) = 0.408 \text{ kips}$$

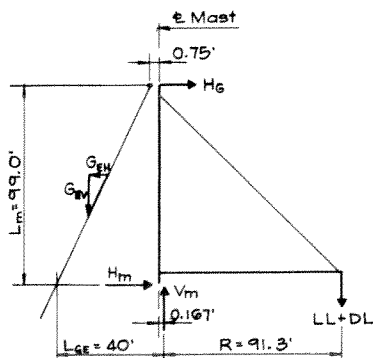


Figure 19

Guy tension (see Fig. 19):

$$\text{Effective guy distance} = L_{GE} = 40 \text{ ft}$$

$$DL_{BT} = DL_{MF} + DL_C + w_{DL} (L/2)$$

$$= 0.6 + 0.4 + (0.0605 \times 89.7/2)$$

$$= 3.71 \text{ kips}$$

$$G_{EV} = \frac{(LL + DL_{BT}) (R) + H_g (L_m)}{L_{GE}}$$

$$= \frac{(12 + 3.71) (91.3) + 0.408 (99.0)}{40}$$

$$= 36.9 \text{ kips}$$

$$G_{EH} = \frac{(LL + DL_{BT}) (R) - (\text{Eccentricity}) (G_{EV})}{L_m} + H_G$$

$$= \frac{(12 + 3.73) (91.3) - (0.75 + 0.167) (36.9)}{99.0} + 0.408$$

$$= 14.6 \text{ kips}$$

Main falls lead line pull:

Main falls has 5 parts.

$$MF = \text{Main Falls Load} = LL + DL_{MF}$$

$$= 12 + 0.6 = 12.6 \text{ kips}$$

$$MFL = \frac{MF}{5} = \frac{12.6}{5} = 2.52 \text{ kips}$$

$$MFL_H = 2.52 \text{ kips (boom flat)}$$

$$MFL_V = 0$$

Boom pin reaction:

$$B_H = TF'_H + TFL_H + MFL_H$$

$$= 11.9 + 2.96 + 2.52 = 17.4 \text{ kips}$$

$$B_V = \frac{1}{2} w_{DL} L_{\text{Boom}} = \frac{1}{2} (0.0605) (89.7)$$

$$= 2.71 \text{ kips}$$

Mast reaction:

$$V_m = G_{EV} + TF'_V + TFL_V + TFL + MFL + B_V + w_m L_m$$

$$= 36.9 + 12.6 + 3.15 + 4.32 +$$

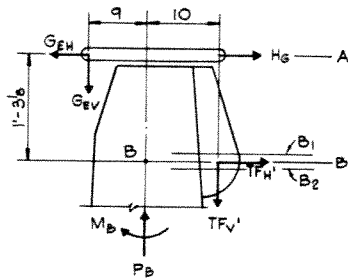
$$2.52 + 2.71 + (0.084)(99.0)$$

$$= 70.5 \text{ kips}$$

$$H_m = G_{EH} - H_G = 14.6 - 0.408 = 14.2 \text{ kips}$$

Moments and axial loads at Sections A through F:

Sections A and B:



$$M_A = 9G_{EV} = 9(36.9) = 332 \text{ kip-in.}$$

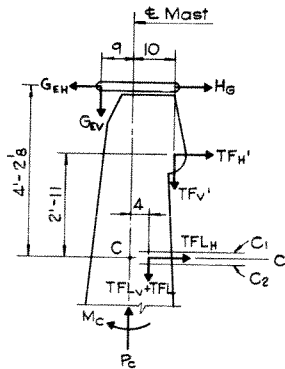
$$\begin{aligned} M_{B1} &= 9G_{EV} + 15.1(G_{EH} - H_G) \\ &= 9(36.9) + 15.1(14.6 - 0.408) \\ &= 546 \text{ kip-in. (tension in front chords)} \end{aligned}$$

$$P_{B1} = G_{EV} = 36.9 \text{ kips}$$

$$\begin{aligned} M_{B2} &= M_{B1} - 10(TF'_V) = 546 - 10(12.6) \\ &= 420 \text{ kip-in. (tension in front chords)} \end{aligned}$$

$$P_{B2} = G_{EV} + TF'_V = 36.9 + 12.6 = 49.5 \text{ kips}$$

Section C:



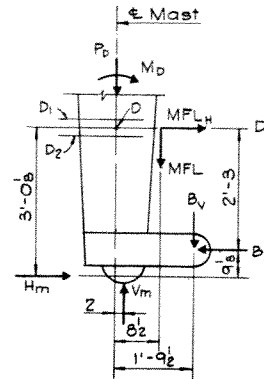
$$\begin{aligned} M_{C1} &= 50.1(G_{EH} - H_G) - 35TF'_H + \\ &\quad 9G_{EV} - 10TF'_V \\ &= 50.1(14.6 - 0.408) - 35(11.9) + \\ &\quad 9(36.9) - 10(12.6) \\ &= 501 \text{ kip-in. (tension in front chords)} \end{aligned}$$

$$P_{C1} = G_{EV} + TF'_V = 36.9 + 12.6 = 49.5 \text{ kips}$$

$$\begin{aligned} M_{C2} &= M_{C1} - 4(TFL_V + TFL) \\ &= 501 - 4(3.15 + 4.32) \\ &= 471 \text{ kip-in. (tension in front chords)} \end{aligned}$$

$$\begin{aligned} P_{C2} &= P_{C1} + TFL_V + TFL \\ &= 49.5 + 3.15 + 4.32 = 57.0 \text{ kips} \end{aligned}$$

Section D:



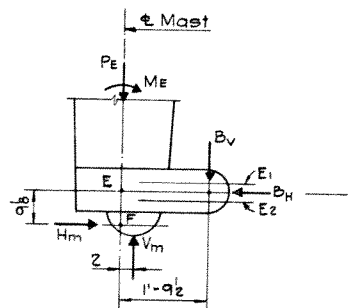
$$\begin{aligned} M_{D1} &= 2V_m + 36.1H_m - 21.5B_V - \\ &\quad 27B_H - 8.5(MFL) \\ &= 2(70.5) + 36.1(14.2) - 21.5(2.73) - \\ &\quad 27(17.4) - 8.5(2.52) \\ &= 104 \text{ kip-in. (compr. in front chords)} \end{aligned}$$

$$\begin{aligned} P_{D1} &= V_m - B_V - MFL = 70.5 - 2.73 - 2.52 \\ &= 65.3 \text{ kips} \end{aligned}$$

$$\begin{aligned} M_{D2} &= 2V_m + 36.1H_m - 21.5B_V - 27B_H \\ &= 2(70.5) + 36.1(14.2) - 21.5(2.73) - \\ &\quad 27(17.4) \\ &= 125 \text{ kip-in. (compr. in front chords)} \end{aligned}$$

$$P_{D2} = V_m - B_V = 70.5 - 2.73 = 67.8 \text{ kips}$$

Section E:



$$\begin{aligned} M_{E1} &= 2V_m + 9.13H_m - 21.5B_V \\ &= 2(70.5) + 9.13(14.2) - 21.5(2.73) \\ &= 212 \text{ kip-in. (compr. in front chords)} \end{aligned}$$

$$\begin{aligned} P_{E1} &= V_m - B_V = 70.5 - 2.73 \\ &= 67.8 \text{ kips} \end{aligned}$$

$$\begin{aligned}
 M_{E2} &= 2V_m + 9.13H_m \\
 &= 2(70.5) + 9.13(14.2) \\
 &= 271 \text{ kip-in. (compr. in front chords)}
 \end{aligned}$$

$$P_{E2} = V_m = 70.5 \text{ kips}$$

Section F: (Moment due to eccentric reaction)

$$M_F = 2V_m = 2(70.5) = 141 \text{ kip-in.}$$

$$P_F = V_m = 70.5 \text{ kips}$$

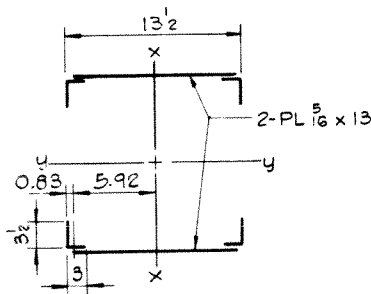
Draw moment diagram (see Fig. 20).

Determine value of C_m :

Assume moments above Sect. C and below Sect. D have little effect on C_m and can be ignored.

$$\begin{aligned}
 C_m &= 0.6 - 0.4 \frac{M_1}{M_2} = 0.6 - 0.4 \frac{104}{471} \\
 &= 0.512 > 0.4
 \end{aligned}$$

Check stress at Section B1:



$$\begin{aligned}
 I_{B1x} &= 4 [2.30 (5.92)^2] + 2 (5/16) (13/12)^3 \\
 &= 322 + 114 = 450 \text{ in.}^4
 \end{aligned}$$

$$A = 4(2.3) + 2(5/16)(13) = 17.3 \text{ in.}^2$$

$$f_{bx} = \frac{M_{B1c}}{I_{B1x}} = \frac{(546)(6.75)}{436} = 8.45 \text{ ksi}$$

$$f_a = \frac{P_{B1}}{A} = \frac{36.9}{17.3} = 2.13 \text{ ksi}$$

$$F_{bx} = 0.6 F_y = 0.6(45) = 27.0 \text{ ksi}$$

$$F_a = 11.69 \text{ ksi (calculated previously)}$$

$$F'_{ex} = 13.29 \text{ ksi (calculated previously)}$$

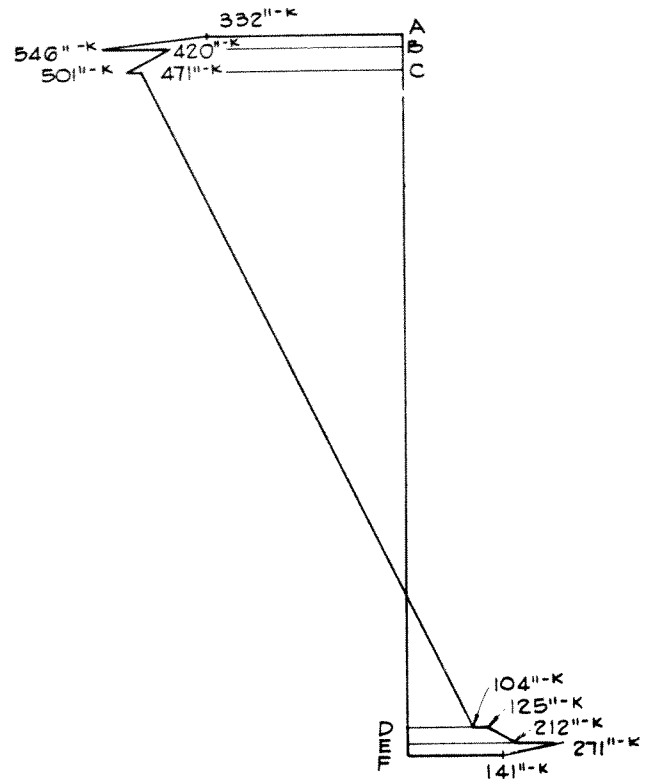


Fig. 20. Mast moment diagram

Interaction Equations at B_1 :

Use $D = 1.2$

$$\begin{aligned}
 \frac{f_a D}{F_a} + \frac{C_m f_{bx} D}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} &< 1.0 \quad (D1.5-1a) \\
 \frac{(2.13)(1.2)}{11.69} + \frac{(0.512)(8.45)(1.2)}{\left(1 - \frac{2.13}{13.29}\right)(27.0)} \\
 &= 0.219 + 0.229 = 0.448 < 1.0 \text{ o.k.}
 \end{aligned}$$

$$\begin{aligned}
 \frac{f_a D}{0.6 F_y} + \frac{f_{bx} D}{F_{bx}} &< 1.0 \quad (D1.5-1b) \\
 \frac{(2.13)(1.2)}{27.0} + \frac{(8.45)(1.2)}{27.0} &= 0.095 + 0.376 \\
 &= 0.471 < 1.0 \text{ o.k.}
 \end{aligned}$$

Check Stress at Section C:

Assume side plates extend to a point below Section C so as to be effective at both C_1 and C_2 . Truss action must be checked where plates are no longer effective. For simplicity, check for truss action at C_2 as if plates were omitted (this is conservative).

Section C_2 :

Back-to-back of chords = 14-5/8 in.

$d = 13.0$ in.

$$f_{bx} = \frac{M_{C2}}{2dA_c} = \frac{471}{2(13.0)(2.30)} = 7.88 \text{ ksi}$$

$$f_a = \frac{P_{C2}}{A} = \frac{57.0}{9.20} = 6.20 \text{ ksi}$$

L (chord length between lacing points at Section C) = 1' - 2 1/4"

$$\frac{l}{r} = \frac{14.25}{0.897} = 15.9$$

F_a (chord) = 25.92 ksi = F_{bx} (mast)

F_a (mast) = 11.69 ksi (calculated previously)

$F'_{ex} = 13.29$ ksi (calculated previously)

Interaction Equations at C_2 :

Use $D = 1.2$

$$\frac{F_a D}{F_a} + \frac{C_m f_{bx} D}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} < 1.0 \quad (D1.5-1a)$$

$$\frac{(6.20)(1.2)}{11.69} + \frac{(0.512)(7.88)(1.2)}{\left(1 - \frac{6.20}{13.29}\right)(25.92)} = 0.636 + 0.350 = 0.986 < 1.0 \text{ o.k.}$$

$$\frac{f_a D}{0.6F_y} + \frac{f_{bx} D}{F_{bx}} < 1.0 \quad (D1.5-1b)$$

$$\frac{(6.20)(1.2)}{27.0} + \frac{(7.88)(1.2)}{25.92} = 0.276 + 0.365 = 0.641 < 1.0 \text{ o.k.}$$

\therefore mast is adequate for 6 tons with boom flat.

D. Stiffleg Derrick Boom (Fig. 21)

Given:

$$L = 115' - 0''$$

$$w_{DL} = 188 \text{ lbs/ft} \\ = 0.188 \text{ kips/ft (excluding pin plates)}$$

Chords: 4 L 8 × 6 × 5/8; $F_y = 50$ ksi

c. to c. staggered lacing points: 5' - 6"

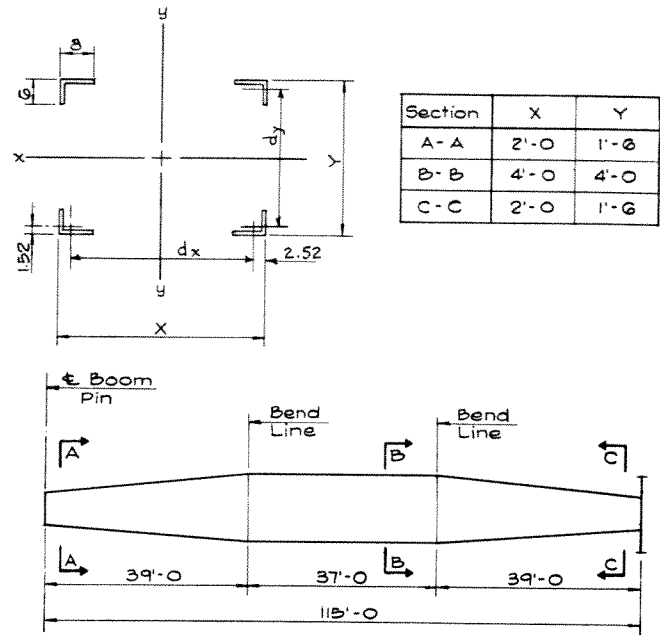


Fig. 21. Stiffleg derrick boom dimensions

Dead loads on boom:

Lower main falls block: 1,900 lbs

200' wire rope (main falls tackle, 7 parts):
1,700 lbs

Topping falls: 4,000 lbs

Topping block and hanger: 4,000 lbs

Upper main falls block and hanger: 2,900 lbs

Runner block: 700 lbs

Jib (20'): 3,400 lbs

Jib and boom sheaves: 600 lbs

Pin plates on upper boom: 2,000 lbs

Problem:

Investigate the boom stresses for a lifted load of 37.5 tons at 110' reach, for bridge erection. Assume a critical section at 46' from boom pin.

Solution:

1. Check width-thickness ratios (Sect. D1.6.1.2):

$$\text{Check } 8'' \text{ leg: } \frac{b}{t} = \frac{8}{5/8} = 12.8 > \frac{76.0}{\sqrt{50}} = 10.7 \text{ n.g.}$$

Max. effective $b = 10.7 (5/8) = 6.69$ in.

Req'd area reduction for 8'' leg = $(8 - 6.69) (5/8) = 0.82 \text{ in.}^2$

Actual area of L8 × 6 × 5/8 = 8.36 in.²

Actual area of total section = 33.4 in.²

A_c = Effective area one chord = 8.36 - 0.82 = 7.54 in.²

A = Effective area total section = 4 (7.54) = 30.2 in.²

2. Find actual moments of inertia:

$$I = A (d/2)^2$$

At Sects. A-A and C-C:

$$I_{Ax} = I_{Cx} = 33.4 (18/2 - 1.52)^2 = 1,870 \text{ in.}^4$$

$$I_{Ay} = I_{Cy} = 33.4 (24/2 - 2.52)^2 = 3,002 \text{ in.}^4$$

$$d_y = 15.0 \text{ in.}, d_x = 19.0 \text{ in.}$$

At Sect. B-B:

$$I_{Bx} = 33.4 (48/2 - 1.52)^2 = 16,900 \text{ in.}^4$$

$$I_{By} = 33.4 (48/2 - 2.52)^2 = 15,400 \text{ in.}^4$$

$$d_y = 45.0 \text{ in.}, d_x = 43.0 \text{ in.}$$

3. Find average moments of inertia and radius of gyration:

Buckling out of vertical (y-y) plane:

$$I_0 = I_{Bx} = 16,900 \text{ in.}^4$$

$$I_1 = I_{Ax} = I_{Cx} = 1,870 \text{ in.}^4$$

$$\frac{I_1}{I_0} = \frac{1,870}{16,900} = 0.111; \quad \frac{h}{L} = \frac{37}{115} = 0.322$$

$$\gamma \text{ (interpolated from Fig. 1 or Table D1.4.4)} \\ = 0.770$$

$$I_{av(x)} = \gamma I_0 = 0.770 (16,900) = 13,000 \text{ in.}^4$$

$$r_x = \sqrt{I_{av(x)}/A} = \sqrt{13,000/33.4} = 19.7 \text{ in.}$$

Buckling out of transverse (x-x) plane:

$$I_0 = I_{By} = 15,400 \text{ in.}^4$$

$$I_1 = I_{Ay} = I_{Cy} = 3,002$$

$$\frac{I_1}{I_0} = \frac{3,002}{15,400} = 0.195; \quad \frac{h}{L} = 0.322$$

$$\gamma \text{ (interpolated from Fig. 1 or Table D1.4.4)} \\ = 0.825$$

$$I_{av(y)} = 0.825 (15,400) = 12,700 \text{ in.}^4$$

$$r_y = \sqrt{12,700/33.4} = 19.6 \text{ in.}$$

4. Determine allowable stresses:

Axial stress:

$$\frac{l}{r_x} = \frac{115 (12)}{19.7} = 70.1; \quad \frac{l}{r_y} = \frac{115 (12)}{19.6} = 70.4$$

From Table 2, AISC Spec. Appendix A:

$$F'_{ex} = 30.39 \text{ ksi}; \quad F'_{ey} = 30.14 \text{ ksi}$$

From Table 1-50, AISC Spec. Appendix A:

$$F_a = 20.86 \text{ ksi}$$

Bending:

Allowable bending stress in the boom, F_b , is equal to the allowable compressive stress in one chord between lacing points, F_a (chord).

$$l \text{ (c. to c. staggered lacing points)} = 66 \text{ in.}$$

$$r_y \text{ (for } L 8 \times 6 \times 5/8) = 1.77 \text{ in.}$$

$$l/r_y = 66/1.77 = 37.3$$

From Table 1-50:

$$F_a \text{ (chord)} = 26.21 \text{ ksi} >$$

$$F_a \text{ (axial)} = 20.86 \text{ ksi } \underline{\text{o.k.}}$$

$$\therefore F_{bx} = F_{by} = 26.21 \text{ ksi}$$

5. Boom tip dead loads

$$DL_{MF} = \text{Lower main falls block} + 200' \text{ wire rope} \\ = 1.9 + 1.7 = 3.6 \text{ kips}$$

DL_c (dead load concentrated at boom tip):

$$\frac{1}{2} \text{ topping falls} = 2.0 \text{ kips}$$

$$20' \text{ jib} = 3.4 \text{ kips}$$

$$\text{Jib and boom sheaves} = 0.6 \text{ kips}$$

$$\text{Topping block and hanger} = 4.0 \text{ kips}$$

$$\text{Upper MF block and hanger} = 2.9 \text{ kips}$$

$$\text{Runner block} = 0.7 \text{ kips}$$

$$\text{Pin plates (upper boom)} = 2.0 \text{ kips}$$

$$\Sigma = DL_c = 15.6 \text{ kips}$$

DL_{BT} (total DL at boom tip):

$$DL_{BT} = DL_{MF} + DL_c + w_{DL}L/2$$

$$= 3.6 + 15.6 + (0.188) (115)/2$$

$$= 30.0 \text{ kips}$$

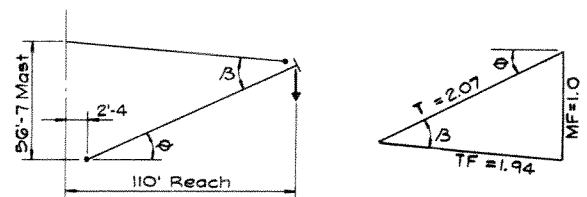
6. Boom analysis

$$\text{Live loads: } LL = 37.5 \text{ tons} = 75.0 \text{ kips}$$

$$\text{Main falls loads: } MF = LL + DL_{MF}$$

$$= 75.0 + 3.6 = 78.6 \text{ kips}$$

Topping force: Draw unit load polygon.



$$\theta = 21^\circ 20'; \quad \sin \theta = 0.364; \quad \cos \theta = 0.931$$

$$\beta = 28^\circ 40'; \quad \sin \beta = 0.480; \quad \cos \beta = 0.877$$

$$TF = 1.94 (LL + DL_{BT})$$

$$= 1.94 (75.0 + 30.0) = 204 \text{ kips}$$

Components of MF and TF:

Draw a free body diagram of the boom tip, with loads in vertical plane only (Fig. 22).

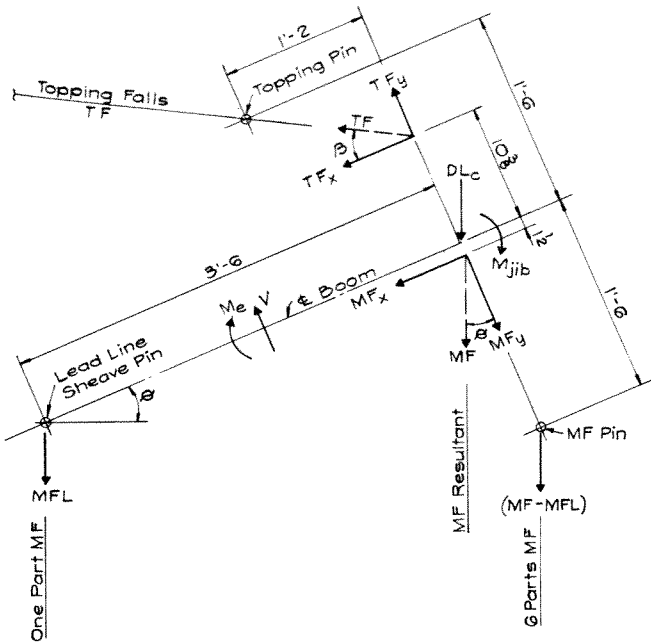


Fig. 22. Free body diagram of boom tip

$$\begin{aligned}
 TF_x &= TF \cos \beta \\
 &= (204) (0.877) = 179 \text{ kips} \\
 TF_y &= TF \sin \beta \\
 &= (204) (0.480) = 97.9 \text{ kips} \\
 MF_x &= MF \sin \theta \\
 &= (78.6) (0.364) = 28.6 \text{ kips} \\
 MF_y &= MF \cos \theta \\
 &= (78.6) (0.931) = 73.2 \text{ kips}
 \end{aligned}$$

Axial stress:

$$\begin{aligned}
 T &= MF_x + TF_x + \frac{MF}{7} + DL_c \sin \theta + \\
 &\quad w_{DL} (L - x) \sin \theta \\
 &= 28.6 + 179 + \frac{78.6}{7} + (15.6) (0.364) + \\
 &\quad (0.188) (115 - 46) (0.364) \\
 &= 230 \text{ kips}
 \end{aligned}$$

$$F_a = T/A = 230/30.2 = 7.62 \text{ ksi}$$

End moment at boom tip:

$$\begin{aligned}
 M_e &= 10.38 TF_x - 1.5 MF_x - \\
 &\quad M_{runner \text{ block}} - M_{jib} \\
 &= 10.38 (179) - 1.5 (28.6) - \\
 &\quad 20 (0.7) (12) - (20/2) (3.4) (12) \\
 &= 1,240 \text{ kip-in. (tension in bottom chords)}
 \end{aligned}$$

Moments at assumed critical section ($x = 46$ ft):

Moment due to M_e :

$$\begin{aligned}
 M_{ex} &= M_e \left(\frac{x}{L} \right) = 1240 \left(\frac{46}{115} \right) \\
 &= 496 \text{ kip-in. (tension in bottom chords)}
 \end{aligned}$$

Moment due to uniform dead load, w_{DL} :

$$\begin{aligned}
 M_{DLx} &= \frac{w_{DL}}{2} (L - x) \\
 &= \frac{(0.188) (46) (115 - 46)}{2} \quad (12) \\
 &= 3,580 \text{ kip-in.}
 \end{aligned}$$

Moment due to horizontal slewing load:

For 110 reach:

$$\begin{aligned}
 H_s &= 0.03 (LL + DL_{BT}) \\
 &= 0.03 (75 + 30) = 3.15 \text{ kips} \\
 M_{Hsx} &= H_s (L - x) \\
 &= 3.15 (115 - 46) (12) \\
 &= 2,610 \text{ kip-in. (transverse bending)}
 \end{aligned}$$

Moment due to horizontal wind load:

Wind on lifted load:

$$H_w (\text{load}) = 0.02 (75) = 1.5 \text{ kips}$$

Wind on boom:

Surface area (windward face) $A_w = 173 \text{ ft}^2$
(computed from detail drawings not shown)

$$\begin{aligned}
 H_w (\text{boom}) &= \frac{C_d P_w A_w}{2} \\
 &= \frac{2.0 (1.0) (173)}{2} \\
 &= 173 \text{ lbs} = 0.17 \text{ kips}
 \end{aligned}$$

Wind on jib:

$$\text{Surface area (windward face) } A_W = 20 \text{ ft}^2$$

$$H_W (\text{jib}) = C_{dp} p_W A_W = 2.0 (1.0) (20) \\ = 40 \text{ lbs} = 0.04 \text{ kips}$$

Total wind load at boom tip (H_W):

$$H_W = H_W (\text{load}) + H_W (\text{boom}) + H_W (\text{jib}) \\ = 1.5 + 0.17 + 0.04 = 1.71 \text{ kips}$$

$$M_{H_W X} = H_W (L - x) = 1.71 (115 - 46) (12) \\ = 1,416 < M_{H_S X} = 2,610 \text{ kip-in.}$$

∴ Stress due to wind need not be considered when checking boom for swinging condition.

Bending stresses at $x = 46 \text{ ft}$:

$$f_b = \frac{M}{2dA_c}$$

$$\text{where } A_c = \text{effective area of one chord} \\ = 7.54 \text{ in.}^2$$

Due to M_{DLX} :

$$C_{mx1} = 1.0$$

$$f_{bx1} = \frac{3580}{2 (45.0) (7.54)} = 5.28 \text{ ksi}$$

Due to M_{EX} :

$$C_{mx2} = 0.6$$

$$f_{bx2} = \frac{496}{2 (45.0) (7.54)} = 0.731 \text{ ksi}$$

Due to slewing moment M_{HS} :

$$C_{my} = 0.85$$

$$f_{bys} = \frac{2610}{2 (43.0) (7.54)} = 4.03 \text{ ksi}$$

Due to wind moment M_{HW} :

$$C_{my} = 0.85$$

$$f_{byw} = \frac{1416}{2 (43.0) (7.54)} = 2.17 \text{ ksi}$$

Interaction Formula (D1.5-1a):

$$\frac{f_a D}{F_a} + \frac{f_{bx1} C_{mx1} D}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{F_{bx2} C_{mx2} D}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{f_{by} C_{my}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0$$

Swinging Condition ($D = 1.03$ and effect of wind not considered):

$$\frac{(7.62) (1.03)}{20.86} + \frac{(5.28) (1.0) (1.03)}{\left(1 - \frac{7.62}{30.39}\right) (26.21)} + \frac{(0.731) (0.6) (1.03)}{\left(1 - \frac{7.62}{30.39}\right) (26.21)} + \frac{(4.03) (0.85)}{\left(1 - \frac{7.62}{30.14}\right) (26.21)} \\ = 0.376 + 0.277 + 0.023 + 0.175 \\ = 0.851 < 1.0 \text{ o.k.}$$

Raising or lowering load ($D = 1.2$ and effect of wind considered):

$$\frac{(7.62) (1.2)}{20.86} + \frac{(5.28) (1.0) (1.2)}{\left(1 - \frac{7.62}{30.39}\right) (26.21)} + \frac{(0.731) (0.6) (1.2)}{\left(1 - \frac{7.62}{30.39}\right) (26.21)} + \frac{(2.17) (0.85)}{\left(1 - \frac{7.62}{30.14}\right) (26.21)} \\ = 0.438 + 0.323 + 0.027 + 0.094 \\ = 0.882 < 1.0 \text{ o.k.}$$

Note: Other sections must be checked to find condition of maximum combined stress.

Interaction Formula (D1.5-1b):

$$\frac{f_a D}{0.60 F_y} + \frac{f_{bx} D}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0$$

Swinging condition (D = 1.03 and wind not considered):

$$\frac{(7.62)(1.03)}{(0.6)(50)} + \frac{(5.28 + 0.731)(1.03)}{26.21} + \frac{4.03}{26.21}$$

$$= 0.261 + 0.236 + 0.154$$

$$= 0.651 < 1.0 \text{ o.k.}$$

Raising or lowering load (D = 1.2 and effect of wind considered):

$$\frac{(7.62)(1.2)}{(0.6)(50)} + \frac{(5.28 + 0.731)(1.2)}{26.21} + \frac{2.17}{26.21}$$

$$= 0.305 + 0.275 + 0.083$$

$$= 0.663 < 1.0 \text{ o.k.}$$

IX. REFERENCES

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7. "Steel Design Manual", Brockenbrough and Johnston, United States Steel Corporation, 1968.
8. "Lifting Crane, Wire Rope Strength Factors", SAE959 Report of the Construction and Industrial Machinery Technical Committee, Approved June 1966.
9. "The Safety Code for Cranes, Derricks, Hoists, Jacks and Slings", ANSI - B30.6-1969, American Society of Mechanical Engineers, Section 6-1.3, Ropes and Reeving Accessories.
10. "Numerical Descriptions of the Ambient Low Temperature and High Strain Rate Flow and Fracture Behavior of Plain Carbon Steel", A.R. Rosenfield and G.T. Hahn - Transactions of the ASM Volume 59 - 1966, pages 962-975.

APPENDIX A
SPECIFICATION FOR THE DESIGN OF GUY AND STIFFLEG DERRICKS*

SECT. D1.1 SCOPE

This specification is intended for the analysis of structural elements of guy and stiffleg derricks for use as heavy lift derricks in the erection of steel structures. Provisions for the analysis of derricks to be used in performing duty cycle work are not included. This specification is not applicable to structures other than guy and stiffleg derricks.

SECT. D1.2 LOADS AND FORCES

D1.2.1 Dead Load

The dead load to be assumed in design shall consist of the weight of the boom, mast, guys, hook, ball, sheaves and all rope the weight of which is supported on the foot block.

D1.2.2 Live Load

The live load, which will be referred to as the rated capacity of the derrick, shall consist of the lifted weight.

D1.2.3 Impact

D1.2.3.1 To provide for the dynamic effect of accelerating and decelerating live load motion, all stresses computed on the basis of live plus dead load, as defined in Sects. D1.2.1 and D1.2.2, shall be increased by a dynamic factor, D, equal to 1.2, except as otherwise provided in Sect. D1.2.3.2 and in interaction Formulas (D1.5-1a) and (D1.5-1b).

D1.2.3.2 For stiffleg derrick booms subjected to a slewing force as provided in Sect. D1.2.4, a dynamic factor, D, equal to 1.03 shall be used in the computation of interaction Formulas (D1.5-1a) and (D1.5-1b).

D1.2.4 Slewing Force

For stiffleg derrick booms operating at an angle greater than 60° with respect to a horizontal plane, a horizontal slewing force at the boom tip

equal to 2 percent of the combined live load and total dead load at the boom tip shall be applied normal to the boom centerline. With the boom flat, this force shall be taken as 3 percent of the combined live load and total dead load at the boom tip. For boom angles between 60° and 0° the percentage shall vary linearly between 2 and 3 percent.

D1.2.5 Wind Load

D1.2.5.1 For guy derricks, and for stiffleg derricks in building construction, wind load can be disregarded in stress calculations.

D1.2.5.2 For stiffleg derrick booms in bridge erection, a horizontal wind force shall be applied at the boom tip in a direction normal to the boom centerline. This wind force shall be equal to the sum of the wind force on the boom face, H_W (boom), plus the wind force on the lifted load, H_W (load), or $H_W = H_W$ (boom) + H_W (load), where

$$H_W \text{ (boom)} = \frac{C_d p_w A_w}{2} \text{ (kips)}$$

$$H_W \text{ (load)} = 0.2LL \text{ (kips)}$$

$$LL = \text{Lifted load (kips)}$$

$$p_w = \text{Design wind pressure (lbs/ft}^2\text{)}$$

$$A_w = \text{Surface area exposed to horizontal wind (ft}^2\text{)}$$

$$C_d = \text{Shape factor of chord and lacing members}$$

For the slewing condition, the greater of either the horizontal slewing force, as determined by Sect. D1.2.4, or the horizontal wind force acting at the boom tip shall be considered in the "y-axis bending" terms of interaction Formulas (D1.5-1a) and (D1.5-1b), and the dynamic factor D shall be as provided in Sect. D1.2.3.2.

For the condition of no slewing, the horizontal wind force shall be considered in the "y-axis bending" terms of the interaction formulas, and the dynamic factor D shall be as provided in Sect. D1.2.3.1.

*Some Sections of this Derrick Specification are identical with Sections of the AISC "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings." In such cases, the AISC Specification Section number is given in parentheses directly following the Section number of the Derrick Specification.

SECT. D1.3 (1.4) MATERIAL

D1.3.1 (1.4.1) Structural Steel

D1.3.1.1 (1.4.1.1) Material conforming to one of the following listed (latest date of issue) is approved for use under this Specification:

- Structural Steel, ASTM A7
- Structural Steel, ASTM A36
- Welded and Seamless Steel Pipe, ASTM A53, Grade B
- High-Strength Structural Steel, ASTM A440
- High-Strength Low Alloy Structural Manganese Vanadium Steel, ASTM A441
- High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality, ASTM A572
- High-Strength Low-Alloy Structural Steel with 50,000 psi Minimum Yield Point to 4 in. Thick, ASTM A588

Certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6 and the governing specification shall constitute sufficient evidence of conformity with one of the above ASTM specifications.

D1.3.1.2 Mention of the above grades of steel is not intended to exclude other grades which have given satisfactory service in the past and are currently in use in existing derricks.

D1.3.1.3 Unidentified steel, if free from surface imperfections, may be used for parts of minor importance, or for unimportant details, where the precise physical properties of the steel would not affect the strength of the structure.

D1.3.2 (1.4.2) Other Metals

Cast steel shall conform to one of the following specifications, latest edition:

- Mild-to-Medium-Strength Carbon-Steel Castings for General Application, ASTM A27, Grade 65-35
- High-Strength Steel Castings for Structural Purpose, ASTM A148, Grade 80-50

Steel forgings shall conform to one of the following specifications, latest edition:

- Carbon Steel Forgings for General Industrial Use, ASTM A235, Class C1, F and G. (Class C1 Forgings that are to be welded shall be ordered in accordance with Supplemental Requirements S5 of A235.)
- Alloy Steel Forgings for General Industrial Use, ASTM A237, Class A

Certified test reports shall constitute sufficient evidence of conformity with the specifications.

D1.3.3 (1.4.3) Rivets

Rivets shall conform to the provisions of the Specification for Structural Rivets, ASTM A502, Grade 1 or Grade 2, latest edition.

Manufacturer's certification shall constitute sufficient evidence of conformity with the specifications.

D1.3.4 (1.4.4) Bolts

High strength steel bolts shall conform to one of the following specifications, latest edition:

- High Strength Bolts for Structural Steel Joints, Including Suitable Nuts and Plain Hardened Washers, ASTM A325
- Quenched and Tempered Alloy Steel Bolts for Structural Steel Joints, ASTM A490

Other bolts shall conform to the Specification for Low-Carbon Steel Externally and Internally Threaded Standard Fasteners, ASTM A307, latest edition, hereinafter designated as A307 bolts.

Manufacturer's certification shall constitute sufficient evidence of conformity with the specifications.

D1.3.5 Wire Rope

Wire rope shall meet the requirements of the American National Standards Institute, ANSI B30.6, and/or SAE J959.

SECT. D1.4 ALLOWABLE STRESSES

Except as provided in Sect. D1.5, all components of the structure shall be so proportioned that the stress, in kips per square inch, shall not exceed the following values:

D1.4.1 (1.5.1) Structural Steel

D1.4.1.1 (1.5.1.1) Tension

On the net section, except at pin holes:

$$F_t = 0.06F_y$$

but not more than 0.5 times the minimum tensile strength of the steel.

On the net section at pin holes in eyebars, pin-connected plates or built-up members:

$$F_t = 0.45F_y$$

For tension on threaded parts see Table 1.4.2.1.

D1.4.1.2 (1.5.1.2) Shear

On the gross section:

$$F_v = 0.40F_y$$

(The gross section of rolled and fabricated shapes may be taken as the product of the overall depth and the thickness of the web.)

TABLE D1.4.2.1

Description of Fastener	Tension (F _t)	Shear (F _v)	
		Friction-Type Connections	Bearing-Type Connections
A502, Grade 1, hot-driven rivets	20.0		15.0
A502, Grade 2, hot-driven rivets	27.0		20.0
A307 bolts	20.0 ¹		10.0
Threaded parts ³ of steel meeting the requirements of Sect. 1.4.1	0.60F _y ¹		0.30F _y
A325 and A449 bolts, when threading is not excluded from shear planes	40.0 ²	15.0	15.0
A325 and A449 bolts, when threading is excluded from shear planes	40.0 ²	15.0	22.0
A490 bolts, when threading is not excluded from shear planes	54.0 ^{2, 4}	20.0	22.5
A490 bolts, when threading is excluded from shear planes	54.0 ^{2, 4}	20.0	32.0

¹ Applied to tensile stress area equal to $0.7854 [D - (0.9743/n)]^2$ where D is the major thread diameter and n is the number of threads per inch.

² Applied to the nominal bolt area.

³ Since the nominal area of an upset rod is less than the stress area, the former area will govern.

⁴ Static loading only.

D1.4.1.3 Compression

D1.4.1.3.1 On the web of rolled shapes at the top of the fillet (crippling):

$$F_a = 0.75F_y$$

D1.4.1.3.2 On the gross section of axially loaded compression members, when l/r , the largest effective slenderness ratio of any unbraced segment, is less than C_c :

$$F_a = \frac{\left[1 - \frac{(l/r)^2}{2C_c^2} \right] F_y}{\frac{5}{3} + \frac{3(l/r)}{8C_c} - \frac{(l/r)^3}{8C_c^3}} \quad (1.4-1)$$

where

$$C_c = \sqrt{2\pi^2 E/F_y}$$

l = actual unbraced length, in.

r = radius of gyration about x or y axis*, in.

E = modulus of elasticity, in.⁴

F_y = yield stress of material, ksi

D1.4.1.3.3 On the gross section of axially loaded compression members, when l/r exceeds C_c :

$$F_a = \frac{12\pi^2 E}{23 (l/r)^2} \quad (1.4-2)$$

D1.4.1.3.4 Maximum Ratios

The slenderness ratio l/r of compression members shall not exceed 200.

The slenderness ratio l/r of tension members, other than rods and wire rope, preferably should not exceed:

For main members 240

For bracing and other secondary members. 300

D1.4.1.4 Bending

Tension and compression due to bending at extreme fibres of hot-rolled or built-up members:

$$F_b = 0.60F_y$$

except as otherwise provided in Sect. D1.4.4.2.

*For the determination of equivalent radius of gyration of tapered members, see Sect. D1.4.7.1.

D1.4.1.5 (1.5.1.5) Bearing (on contact area)

D1.4.1.5.1 (1.5.1.5.1) Milled surfaces, including bearing stiffeners and pins in reamed, drilled, or bored holes:

$$F_p = 0.90F_y^*$$

D1.4.2 (1.5.2) Rivets, Bolts, and Threaded Parts

D1.4.2.1 (1.5.2.1) Allowable tension and shear stresses on rivets, bolts and threaded parts (kips per square inch of area of rivets before driving of unthreaded-body area of bolts and threaded parts except as noted) shall be as given in Table 1.4.2.1. High strength bolts required to support applied load by means of direct tension shall be so proportioned that their average tensile stress, computed on the basis of nominal bolt area and independent of any initial tightening force, will not exceed the appropriate stress given in Table D1.4.2.1. The applied load shall be the sum of the external load and any tension resulting from prying action produced by deformation of the connected parts.

D1.4.2.2 (1.5.2.2) Allowable bearing stress on projected area of bolts in bearing-type connections and on rivets:

$$F_p = 1.35F_y$$

where F_y is the yield stress of the connected part. (Bearing stress is not restricted in friction-type connections assembled with A325, A449 or A490 bolts.)

D1.4.3 (1.5.4) Cast Steel and Steel Forgings

Allowable stresses same as those provided in Section D1.4.1, where applicable.

D1.4.4 Design Properties for Tapered Members

D1.4.4.1 The moment of inertia, section modulus and radius of gyration to be used in the design of box-type booms and masts having tapered ends of similar proportions and constant-size corner angles shall be those of a prismatic member of equivalent stiffness.

The moment of inertia I at any cross-section within the tapered portion may be computed as

$$I_Z = I_1 (Z/a)^2$$

where I_1 is the moment of inertia at small end of tapered portion and distances Z and a are as shown on Fig. D1.4.4. At the large end of the taper,

$$I_0 = I_1 (d_0/d_1)^2$$

where d_0 and d_1 are, respectively, the out-to-out distance of chord angles at the large and small end of the tapered portion.

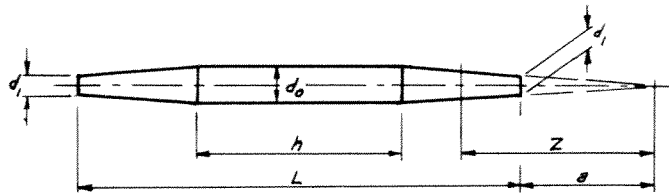


Figure D1.4.4

The equivalent radius of gyration, r , for use in determining the slenderness ratio of the member, is:

$$r = \sqrt{\gamma I_0 / A}$$

where

- I_0 = Moment of inertia of prismatic central portion of the member (in.⁴)
- γ = Applicable coefficient from Table D1.4.4
- A = Area of cross-section at prismatic central portion (in.²)

TABLE D1.4.4
Coefficients γ

I_1 I_0	h/l					
	0	0.2	0.4	0.6	0.8	1.0
0.1	0.548	0.675	0.818	0.937	0.993	1.000
0.2	0.645	0.760	0.873	0.957	0.995	1.000
0.4	0.772	0.853	0.929	0.976	0.998	1.000
0.6	0.863	0.916	0.962	0.987	0.999	1.000
0.8	0.937	0.963	0.982	0.995	1.000	1.000
1.0	1.000	1.000	1.000	1.000	1.000	1.000

D1.4.4.2 In computing the bending stress, F_b , produced by moments applied to a latticed member, each corner angle is treated as an axially loaded element coupled with a similar element on the opposite side of the axis of bending of the member. The axial force in each angle is equal to one-half of the moment at a given section divided by the distance between the centroid axes of the coupled elements. When acting in compression, the value of F_b is obtained using Formula (D1.4-1) or (D1.4-2) and a slenderness ratio equal to the distance between panel points (measured in one of the laced planes) divided by the appropriate radius of gyration of the chord angle. If the panel points in the two legs of an angle are staggered, the applicable slenderness ratio is the larger value l/r_x or l/r_y ; if the panel points are not staggered, the applicable slenderness ratio is l/r_z .

*When parts in contact have different yield stresses, F_y shall be the smaller value.

SECT. D1.5 COMBINED STRESSES

D1.5.1 Axial Compression and Bending

Latticed members subjected to both axial compression and bending stresses shall be proportioned to satisfy the following requirements:

$$\frac{f_a D}{F_a} + \frac{C_{mx1} f_{bx1} D}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{mx2} f_{bx2} D}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} + \frac{C_{my} f_{by}}{\left(1 - \frac{f_a}{F'_{ey}}\right) F_{by}} \leq 1.0 \quad (D1.5-1a)$$

and

$$\frac{f_a D}{0.60 F_y} + \frac{f_{bx} D}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (D1.5-1b)$$

The fourth term of Formula (D1.5-1a) and the third term of Formula (D1.5-1b), i.e., the y-axis bending terms, are applicable only to stiffleg derrick booms subjected to wind or slewing forces.

In Formulas (D1.5-1a) and (D1.5-1b), the subscripts x and y, combined with subscripts b, m and e, indicate the axis of bending about which a particular stress or design property applies; subscripts 1 and 2 indicate stress due to end moment and uniformly distributed moment, respectively.

D = a dynamic factor as defined in Sect. D1.2.3

F_a = axial stress that would be permitted if axial force alone existed in the member as a whole, ksi

F_b = compressive bending stress that would be permitted if bending moment alone existed in the member as a whole, ksi

F_e = $\frac{12\pi^2 E}{23 (l_b/r_b)^2}$ ksi, where l_b is the actual unbraced length in the plane of bending and r_b is the corresponding radius of gyration or equivalent r_b in the case of tapered members

f_a = computed axial stress, ksi

f_b = computed compressive bending stress at the point under consideration, ksi
= M/2dA_e in the case of latticed members

C_m = a coefficient whose value shall be taken as follows:

1. For compression stress due to gravity moment: C_m = 1.0

2. For compression stress due to end moment, including that due to slewing force or wind on stiffleg derrick booms: C_m = 0.6 - 0.4 (M₁/M₂), but not less than 0.4, where M₁/M₂ is the ratio of the smaller to larger moments at the ends of that portion of the member unbraced in the plane of bending under consideration. M₁/M₂ is positive when the member is bent in reverse curvature and negative when it is bent in single curvature.

D1.5.2 (1.6.3) Shear and Tension

Rivets and bolts subject to combined shear and tension shall be so proportioned that the tension stress, in kips per square inch, produced by forces applied to the connected parts, shall not exceed the following:

For A502 Grade 1 rivets

$$F_t = 28.0 - 1.6f_v \leq 20.0$$

For A502 Grade 2 rivets

$$F_t = 38.0 - 1.6f_v \leq 27.0$$

For A307 bolts (applied to stress area)

$$F_t = 28.0 - 1.6f_v \leq 20.0$$

For A325 and A449 bolts in bearing —

$$\text{type joints} \dots \dots F_t = 50.0 - 1.6f_v \leq 40.0$$

For A490 bolts in bearing-type joints

$$\dots \dots \dots F_t = 70.0 - 1.6f_v \leq 54.0$$

where f_v, the shear stress produced by the same forces, shall not exceed the value for shear given in Sect. D1.4.2.

For bolts used in friction-type joints, the shear stress allowed in Sect. D1.4.2 shall be reduced so that:

For A325 and A449 bolts

$$\dots \dots \dots F_v \leq 15.0(1 - f_t A_b / T_b)$$

For A490 bolts \dots \dots F_v ≤ 20.0(1 - f_tA_b/T_b)

where f_t is the average tensile stress due to a direct load applied to all of the bolts in a connection and T_b is the specified pretension load of the bolt.

SECT. D1.6 (1.9) WIDTH-THICKNESS RATIOS

D1.6.1 (1.9.1) Unstiffened Elements Under Compression

D1.6.1.1 (1.9.1.1) Unstiffened (projecting) compression elements are those having one free edge parallel to the direction of compression stress. The width of unstiffened plates shall be taken from the free edge to the first row of fasteners; the width of legs of angles, channel and zee flanges, and stems of tees shall be taken as the full nominal dimension; the width of flanges of I-shape members and tees shall be taken as one-half the full nominal width.

The thickness of a sloping flange shall be measured halfway between a free edge and the corresponding face of the web.

D1.6.1.2 (1.9.1.2) Unstiffened elements subject to axial compression or compression due to bending shall be considered as fully effective when the ratio of width to thickness is not greater than the following:

Single-angle struts; double-angle struts with separators	76.0/√F _y
Struts comprising double angles in contact; angles or plates projecting from girders, columns or other compression members; com- pression flanges of beams; stiffeners on plate girders	95.0/√F _y
Stems of tees	127/√F _y

D1.6.1.3 When the width-thickness ratio of an unstiffened element exceeds the foregoing applicable limit, the actual radius of gyration may be used to compute the slenderness ratio, but only that portion of the actual area lying within the prescribed width-thickness limits shall be considered as the effective area, A_e.

D1.6.2 (1.9.2) Stiffened Elements Under Compression

D1.6.2.1 (1.9.2.1) Stiffened compression elements are those having lateral support along both edges which are parallel to the direction of the compression stress. The width of such elements shall be taken as the distance between nearest lines of fasteners or welds, or between the roots of the flanges in the case of rolled sections.

D1.6.2.2 (1.9.2.2) Stiffened elements subject to axial compression, or to uniform compression due to bending as in the case of the flange of a flexural member, shall be considered as fully effective when the ratio of width to thickness is not greater than the following:

Flanges of square and rectangular box sections of uniform thickness	238/√F _y
Unsupported width of cover plates perforated with a succession of access holes* .	317/√F _y
All other uniformly compressed stiffened elements	253/√F _y

D1.6.2.3 When the width-thickness ratio of a stiffened element exceeds the foregoing applicable limit, the actual radius of gyration may be used to compute the slenderness ratio, but only that portion of the actual area lying within the prescribed width-thickness limits shall be considered as the effective area, A_e.

*Assumes net area of plate at widest hole as basis for computing compression.

SECT. D1.7 (1.14) GROSS AND NET SECTIONS

D1.7.1 (1.14.1) Definitions

The gross section of a member at any point shall be determined by summing the products of the thickness and the gross width of each element as measured normal to the axis of the member. The net section shall be determined by substituting for the gross width the net width computed in accordance with Sects. D1.7.3 to D1.7.6, inclusive.

D1.7.2 (1.14.2) Application

Unless otherwise specified, tension members shall be designed on the basis of net section. Compression members shall be designed on the basis of gross section.

D1.7.3 (1.14.3) Net Section

In the case of a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters of all the holes in the chain, and adding, for each gage space in the chain, the quantity

$$s^2 / 4g$$

where

- s = longitudinal spacing (pitch, in inches) of any two consecutive holes
- g = transverse spacing (gage, in inches) of the same two holes

The critical net section of the part is obtained from that chain which gives the least net width; however, the net section taken through a hole shall in no case be considered as more than 85 percent of the corresponding gross section.

D1.7.4 (1.14.4) Angles

For angles, the gross width shall be the sum of the widths of the legs less the thickness. The gage for holes in opposite legs shall be the sum of the gages from back of angles less the thickness.

D1.7.5 (1.14.5) Size of Holes

In computing net area, the diameter of a rivet or bolt shall be taken as 1/16-inch greater than the nominal dimension of the hole normal to the direction of applied stress.

D1.7.6 (1.14.6) Pin-Connected Members

Eyebars shall be of uniform thickness without reinforcement at the pin holes.** They shall have "circular" heads in which the periphery of the head beyond the pin hole is concentric with the pin hole. The radius of transition between the circular head and the body of the eyebar shall be equal to or greater than the diameter of the head.

**Members having a different thickness at the pin location are termed "built-up."

The width of the body of the eyebar shall not exceed 8 times its thickness, and the thickness shall not be less than 1/2-inch. The net section of the head through the pin hole, transverse to the axis of the eyebar, shall not be less than 1.33 nor more than 1.50 times the cross-sectional area of the body of the eyebar. The diameter of the pin shall not be less than 7/8 of the width of the body of the eyebar. The diameter of the pin hole shall not be more than 1/32-inch greater than the diameter of the pin. For steels having a yield stress greater than 70 ksi, the diameter of the pin hole shall not exceed 5 times the plate thickness.

The minimum net section across the pin hole, transverse to the axis of the member, in pin-connected plates and built-up members shall be determined at the stress allowed for such sections in Sect. D1.4.1.1. The net section beyond the pin hole, parallel to the axis of the member, shall not be less than 2/3 of the net section across the pin hole. The corners beyond the pin hole may be cut at 45° to the axis of the member, provided the net section beyond the pin hole on a plane perpendicular to the cut is not less than that required beyond the pin hole parallel to the axis of the member. The parts of the member built up at the pin hole shall be attached to each other by sufficient fasteners to support the stress delivered to them by the pin.

The distance, transverse to the axis of a pin-connected plate or any separated element of a built-up member, from the edge of the pin hole to the edge of the member or element, shall not exceed 4 times the thickness at the pin hole. The diameter of the pin hole shall not be less than 1.25 times the smaller of the distances from the edge of the pin hole to the edge of a pin-connected plate or separated element of a built-up member at the pin hole. The diameter of the pin hole shall not be more than 1/32-inch greater than the diameter of the pin. In the case of pin-connected plates of uniform thickness for steels having a yield stress greater than 70 ksi, the diameter of the pin hole shall not exceed 5 times the plate thickness.

Thickness limitations on both eyebars and pin-connected plates may be waived whenever external nuts are provided so as to tighten pin plates and filler plates into snug contact. When the plates are thus contained, the allowable stress in bearing shall be no greater than as specified in Sect. D1.4.1.5.1.

SECT. D1.8 (1.15) CONNECTIONS

D1.8.1 Minimum Connections

Connections carrying calculated stresses, except for lacing, shall be designed to support not less than 6 kips.

D1.8.2 (1.15.2) Eccentric Connections

Axially stressed members meeting at a point shall have their gravity axes intersect at a point if practicable; if not, provisions shall be made for bending stresses due to the eccentricity.

D1.8.3 Placement of Rivets, Bolts, and Welds

Except as hereinafter provided, groups of rivets or bolts at the ends of any member transmitting axial stress into that member shall have their centers of gravity on the gravity axis of the member unless provision is made for the effect of the resulting eccentricity. Eccentricity between the gravity axes of single angle, double angle, and similar type members and the gage lines for their riveted or bolted end connections may be neglected.

D1.8.4 (1.15.6) Fillers

When rivets or bolts carrying computed stress pass through fillers thicker than 1/4-inch, except in friction-type connections assembled with high strength bolts, the fillers shall be extended beyond the splice material and the filler extension shall be secured by enough rivets or bolts to distribute the total stress in the member uniformly over the combined section of the member and the filler, or an equivalent number of fasteners shall be included in the connection.

D1.8.5 (1.15.7) Connection of Tension and Compression Members in Trusses

The connections at ends of tension or compression members in trusses shall develop the force due to the design load, but not less than 50 percent of the effective strength of the member.

D1.8.6 Compression Members with Bearing Joints

Where members are finished to bear at splices, there shall be sufficient rivets or bolts to hold all parts securely in place.

All joints shall be proportioned to resist any tension that would be developed by the forces.

D1.8.7 (1.15.11) High Strength Bolts (in Friction-Type Joints) in Combination with Rivets

In new work and in making alterations, rivets and high strength bolts, installed in accordance with the provisions of Sect. D1.9.1 as friction-type connections, may be considered as sharing the stresses resulting from dead and live loads.

SECT. D1.9 (1.16) RIVETS AND BOLTS

D1.9.1 (1.16.1) High Strength Bolts

Use of high strength bolts shall conform to the provisions of the Specifications for Structural Joints Using ASTM A325 or A490 Bolts as approved by the Research Council on Riveted and Bolted Structural Joints.

D1.9.2 (1.16.2) Effective Bearing Area

The effective bearing area of rivets and bolts shall be the diameter multiplied by the length in bearing, except that for countersunk rivets and bolts half the depth of the countersink shall be deducted.

D1.9.3 (1.16.3) Long Grips

Rivets and A307 bolts which carry calculated stress, and the grip of which exceeds 5 diameters, shall have their number increased 1 percent for each additional 1/16-inch in the grip.

D1.9.4 (1.16.4) Minimum Pitch

The minimum distance between centers of rivet and bolt holes shall be not less than 2-2/3 times the nominal diameter of the rivet or bolt but preferably not less than 3 diameters.

D1.9.5 (1.16.5) Minimum Edge Distance

The minimum distance from the center of a rivet or bolt hole to any given edge, used in design or in preparation of shop drawings, shall be that given in Table D1.9.5.

TABLE D1.9.5

Rivet or Bolt Diameter (inches)	Minimum Edge Distance for Punched, Reamed or Drilled Holes (inches)	
	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars or Gas Cut Edges**
1/2	7/8	3/4
5/8	1-1/8	7/8
3/4	1 1/4	1
7/8	1 1/2*	1-1/8
1	1 3/4*	1 1/4
1-1/8	2	1 1/2
1 1/4	2 1/4	1-5/8
Over 1 1/4	1 3/4 × Diameter	1 1/4 × Diameter

*These may be 1/4-in. at the ends of beam connection angles.

**All edge distances in this column may be reduced 1/8-in. when the hole is at a point where stress does not exceed 25% of the maximum allowed stress in the element.

D1.9.6 Minimum Edge Distance in Line of Stress

In connections for tension members, the distance from the center of the end fastener to that end of the connected part towards which the stress is directed, measured parallel to the direction of the stress, shall not be less than

$$2d f_p / F_u$$

but not less than 1.5d nor the distances in Table D1.9.5, where

d = nominal fastener diameter

f_p = computed force bearing on the fastener

F_u = tensile strength of the part applying that force

SECT. D1.10 BUILT-UP MEMBERS

D1.10.1 Open Box-Type Beams and Grillages

Where two or more rolled beams or channels are used side-by-side to form a flexural member, they shall be connected together at intervals of not more than 5 feet. Through-bolts and separators may be used, provided that in beams having a depth of 12 inches or more, no fewer than 2 bolts shall be used at each separator location. When concentrated loads are carried from one beam to the other, or distributed between the beams, diaphragms having sufficient stiffness to distribute the load shall be riveted, bolted, or welded between the beams.

D1.10.2 (1.18.2) Compression Members

D1.10.2.1 (1.18.2.1) All parts of built-up compression members and the transverse spacing of their lines of fasteners shall meet the requirements of Sect. D1.9.

D1.10.2.2 The longitudinal spacing for intermediate rivets or bolts in built-up members shall be adequate to provide for the transfer of calculated stress. However, where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $190\sqrt{F_y}$ nor 18 inches. The maximum longitudinal spacing of rivets or bolts connecting two rolled shapes in contact with one another shall not exceed 24 inches.

D1.10.2.3 (1.18.2.4) Compression members composed of two or more rolled shapes separated from one another by intermittent fillers shall be connected to one another at these fillers at intervals such that the slenderness ratio l/r of either shape, between the fasteners, does not exceed the governing slenderness ratio of the built-up member. The least radius of gyration r shall be used in computing the slenderness ratio of each component part.

D1.10.2.4 Open sides of compression members built up from plates or shapes shall be provided with lacing having tie plates at each end, and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In main members carrying calculated stress, the end tie plates shall have a length of not less than the distance between the lines of rivets or bolts con-

necting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than $1/50$ of the distance between the lines of rivets or bolts connecting them to the segments of the members. The pitch in tie plates shall be not more than 6 diameters and the tie plates shall be connected to each segment by at least three fasteners.

D1.10.2.5 (1.18.2.6) Lacing, including flat bars, angles, channels or other shapes employed as lacing, shall be so spaced that the ratio l/r of the flange included between their connections shall not exceed the governing ratio for the member as a whole. Lacing shall be proportioned to resist a shearing stress normal to the axis of the member equal to 2 percent of the total compressive stress in the member. The ratio l/r for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at their intersections. Lacing bars in compression may be treated as secondary members, l being taken as the unsupported length

of the lacing bar between rivets connecting it to the components of the built-up member for single lacing and 70 percent of that distance for double lacing. The inclination of lacing bars to the axis of the member shall preferably be not less than 60 degrees for single lacing and 45 degrees for double lacing. When the distance between the lines of rivets in the flanges is more than 15 inches, the lacing shall preferably be double or be made of angles.

D1.10.2.6 (1.18.2.7) The function of tie plates and lacing may be performed by continuous cover plates perforated with a succession of access holes. The width of such plates at access holes, as defined in Sect. D1.6.2, is assumed available to resist axial stress, provided that: the width-to-thickness ratio conforms to the limitations of Sect. D1.6.2; the ratio of length (in direction of stress) to width of hole shall not exceed 2; the clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting rivets, bolts or welds; and the periphery of the holes at all points shall have a minimum radius of $1\frac{1}{2}$ inches.

APPENDIX B
GUY DERRICK TEST PROGRAM

Following is a description of the test set up and program for direct observation of the stress amplification due to dynamic loads applied to a steel erection guy derrick, conducted under the auspices of the AISC Task Force Committee and with the support of the National Erectors Association.

(a) The guy derrick, erected on an approximately 20 ft high tower, had a 97 ft mast and 90 ft boom with 4 guys to the back and 2 to the front. Both the main falls and the topping falls had 5 parts of 5/8" dia. wire rope. The hoist was a 100 horsepower diesel with torque converter, rated at 100,000 lbs. lead line pull.

(b) Strain gauges were located on all the chord angles at 5 cross sections of both boom and mast.

In addition, a total of 11 load cells were installed: at the hook, at the boom tip, in the load falls, in the topping falls, on the working guys, and under the mast.

Three 7-channel tape recorders were used to record information from 21 sources. Time mark pulses were superimposed on the signals from the load cell at the hook.

(c) When tests were planned, no information on observed stress amplifications in derricks was available. The test of necessity had to be of a non-destructive nature, since available funds did not permit replacement of damaged elements; therefore, two series of tests were required.

In the first series, static and dynamic tests were run with 3, 5 and 7 ton loads. These loads were not more than 50% of the rated capacity of the derrick and were chosen, as has been noted, because of the unknown effect of dynamic loading. The 3 ton loading was used to evaluate the derrick in 5 boom positions from flat to a 20 ft radius. The 5 ton load was used in 3 positions, from 60 to 20 ft radii, while the 7 ton load was used at a 20 ft radius.

A second series of tests was conducted in which the loads used were at, or close to, the rated capacity. For this second series of tests, a 6 ton load was used in 5 positions from flat to a 20 ft radius; 7 tons in 4 positions from 80 ft to 20 ft radii; 9 tons in 3 positions from 60 ft to 20 ft radii; and finally 13 tons at 2 positions, 40 ft and 20 ft radii.

In the first test series, abnormally high "panic type" starts and stops were used. In the second series, starts and stops which were more severe than would ordinarily be encountered with a qualified operator under emergency conditions were used.

(d) Dynamic tests consisted of:

1. Acceleration of load up.
2. Braking after acceleration up.
3. Release of load followed by braking.

(e) Approximately 6000 static and dynamic readings were taken during the first series of tests, while about 2700 readings were made during the second series.

(f) Zero readings were taken with the boom up tight against the mast. Calculated stresses for the derrick in this position were extremely small; it is assumed that use of this position for establishing strain gauge zero reference had no real significance when the ratios of dynamic to static stresses were computed.

(g) The strain gauges were accurate to within 20 microstrains (± 600 psi).

Therefore, when amplification factors (dynamic to static) involved low stress levels, or when the stresses were due to small loads on the hook, the amplification factors were not meaningful. For the test derrick, for example, the most critical stresses in the boom occurred when the boom was in the flat position. With a load of 6 tons on the hook, total stresses (including dynamic effects) of 25 ksi were recorded. (Obviously, the boom at 20 ft radius was not the critical member when derrick capacity was determined.)

