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Wind and Earthquake Stresses in Tall Buildings

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WIND AND EARTHQUAKE STRESSES IN TALL BUILDINGS

By

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INTRODUCTION

With the increasing cost of land in the modern city it has been necessary to expand upward rather than on the surface. The result has been to concentrate commercial and industrial enterprises in small areas whose influence is felt over the entire world. Notable examples are New York and Chicago. In these developments the engineer has played no small part.

It is claimed by some that the maximum economic height has been reached at a thousand feet. However, both the Chrysler and Empire State Buildings have slightly passed this mark. It is not improbable that even taller buildings than these will be built in the near future despite the prophesies to the contrary.

One of the major factors in the design of tall buildings is the force of the wind against the walls. It is the purpose of this paper to present a few of the most used methods for determining wind stresses in the steel frames of buildings.

Fortunately the localities in which most of the extremely tall buildings are constructed are not subject to earthquakes as the layman thinks of them. That is, judging from the past and from present observations it is not probable that there will be any tremors of sufficient intensity to to warrant special design. Comparatively little is known concerning the distribution of stresses in a structure subject to earthquake shock. At present the best safeguards seem to be to limit the height and to base calculations upon the assumption that the structure is a rigid body. Chapter IV deals with this subject.

CHAPTER I

STATIC STRESSES AND WIND FORCES

In the design of tall buildings it is important that the bents be as simple as possible with the neutral axis in the center if at all feasible. No hard and fast rules can be laid down due to the fact that the first considerations are economic. Each building must be planned in advance to suit the type of tenants expected to occupy it. In most cases a building under construction is rented or leased in part or as a whole long before it is completed. However, close cooperation between the architect and engineer will result in many savings to the buyer as well as simplifying technical matters.

The calculation of direct stresses consists only of the addition of the dead and live loads for each girder and column. With these stresses known the size of member may be selected from a handbook in accordance with the flexure and column formulas. A recommended handbook is "Steel Construction" published by the American Institute of Steel Construction Inc. All sizes and shapes considered standard for American practice are listed. In selecting members, moments and loads on columns and girders should be increased to allow for wind stresses as will be explained in later chapters. The amount of this increase depends upon the experience of the engineer. The allowance

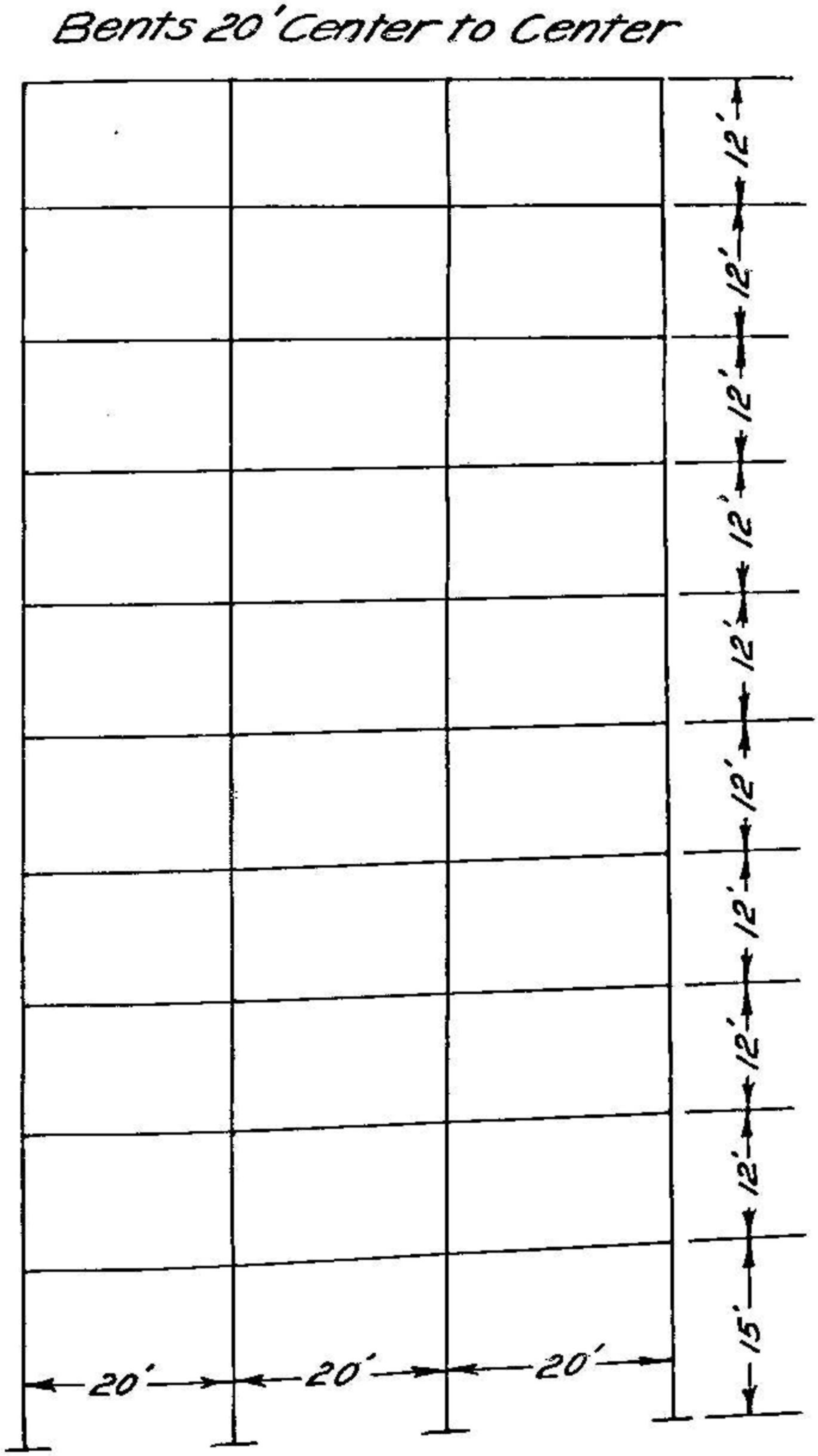


Figure 1

to be made for girders depends upon the span, story and method used to calculate the wind stresses. A typical bent is shown in figure 1.

Weight due to wall = 12(20)(100) = 24000

Live load of floor = 20(20)(100)/2 = 20000

Dead " " " = 20(25)(20)/2 = 5000

Total weight = 49000#

Since the spans are equal the moments in the girders are equal.

M = WL/8 = 49000(20)(12)/8 = 1470000#"

The section modulus is m/S = 1470000/18000 = 81.7

A 15", 65.0# "I" beam of section modulus 84.28 is chosen. If an increase is to be made to allow for wind stress it should be used with the weight or moment and not with the section modulus.

The same calculations are repeated for the transverse bent except that there are no loads on the girders from the floor.

Weight due to wall = 12(100)(20) = 24000#

M = WL/8 = 24000(20)(12)/8 = 720000#"

The section modulus is M/S = 720000/18000 = 40.0

A 12", 40.8# "I" beam of section modulus 44.8 is chosen.

There is now sufficient data for the selection of the corner columns. It is usually considered good practice to allow columns of the same section to continue for two or three stories.

Weight due to floor etc. = 49000/2	=	24500#
Assumed weight of roof = 49000/2	=	24500
Weight of transverse girder = 40.8(20)/2	=	480
Weight of first girder = 65.0(20)/2	=	650
Weight of transverse wall = 100(12)(20)/2	=	12000

Total weight 62000#

For the prelimenary design no account is taken of the effect of moments on the columns due to wind. The A.I.S.C. specifications for columns allows 15000 pounds per square inch up to 60 L/r where L is the unsupported length and r is the least radius of gyration, both in inches. Over 60 L/r the recommended formula is $S = \frac{18000}{1-\left(\frac{L^2}{18000r^2}\right)}$ Tables listing the various column sections in terms of the allowable total stress are given in handbooks. For the above column a 10", 21 "H" Bethlehem section capable of resisting 64000# may be used.

If we allow the columns to continue for two stories, the weight on the column for the fourth story down from the top is three times the above total weight plus the weight of additional members acting on the column. As the base of the building is approached the weight of the members becomes more and more a factor of design.

1 A.I.S.C. Standard Specification, American Institute of Steel Construction Inc. Steel Construction 225.

If static stresses were the only consideration in the design of a tall building, it is obvious from the previous calculations that little if any engineering skill would be required for the selection of members. The wind acting against the side of a building exerts a force which must be taken up by the steel frame. Moments are introduced into the structure that greatly complicate maters. Before the methods of calculating these moments are taken up it is well to review the concepts that are used as to how the wind acts against the sides of a building.

Man has long been familiar with the relation between force and velocity of the wind. The "harder" the wind blows the faster a windmill will revolve, consequently performing more work in a given time. Newton was the first to formulate this knowledge into a mathematical expression. He observed the resistance of plane surfaces to a fluid in motion and theoretically deduced the following originally written in Latin and given as Prop. XLVIII:[1]

> The velocities of pulses propagated in an elastic fluid are in a ratio compounded of the subduplicate ratio of the elastic force directly, and the subduplicate

[1] Robins Fleming, Wind Stresses in Tall Buildings 44.

> ratio of the density inversely; supposing the elastic force of the fluid to be proportional to its condensation.

Expressed in an equation $V = K(p^{\frac{1}{2}}/D^{\frac{1}{2}})$ or $P = CV^2$. This is the form of the equation most used today with experimental values of C.

Professor Marvin, who is now chief of the weather bureau, as a result of many experiments on Mount Wilson gave C the value of .004 where V is given in miles per hour and P in pounds per square foot.[1]

The wind pressure on a building is not the same at the top as it is at the base. This is due to the difference in velocity. Experiments have shown that water flowing through a pipe has a greater velocity at the center than next to the walls. This difference is due to friction. It is doubtful whether the friction at the earth's surface exerts enough force to seriously impede the total or mean velocity of air in motion.[2] To investigate accurately the relationship existing between velocities as a function of the height above the ground would require elaborate preparation and care for good results. The greatest inconsistencies would occur within the first thousand feet. Buildings, trees, shrubs or any irregular terrain would

1 Robins Fleming, Wind Stresses in Tall Buildings, 48.

2 Ibid.

result in eddy currents which would seriously affect the data. The results of the first thousand feet is of the greatest importance to the structural engineer. Many formulas have been developed and made to agree with experimental data. Three that are frequently quoted are:[1]

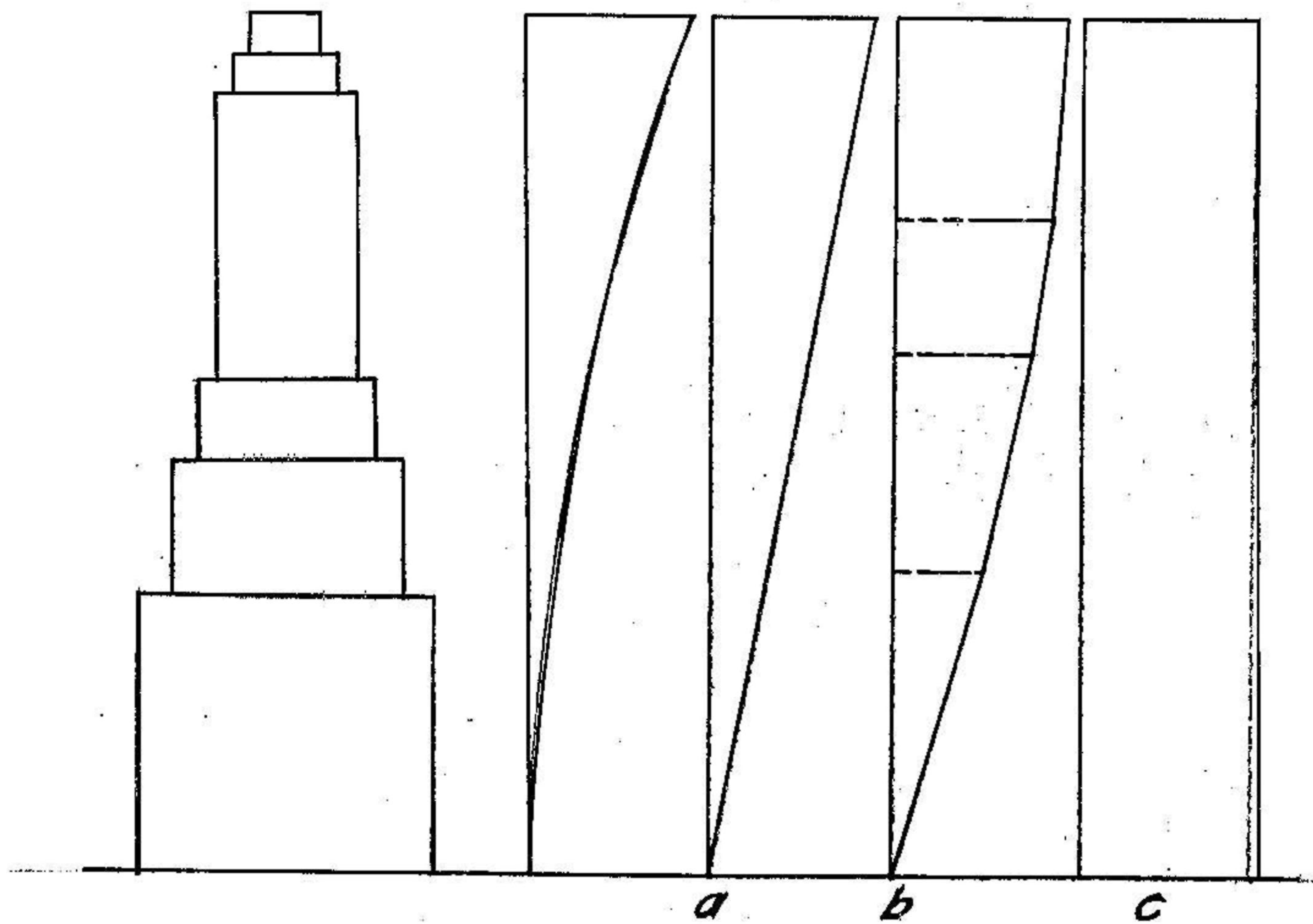
$$(1)\quad P = 0.00126\,h + 1.16\,P_g$$

$$(2)\quad V = v\left(\frac{H+72}{h+72}\right)^{1/2}$$

$$(3)\quad V = v\left(\frac{H}{h}\right)^{1/4}$$

(1) is the work of S. P. Wing and requires the use of a Dines anemometer or any anemometer which will register pressures. The data was collected over a range of 1000 feet. P is the pressure in pounds per square foot at any distance h above ground level and P_g is the pressure in pounds per square foot at the ground level. (2) and (3) are the respective works of Stevenson and Archibald. The data for (2) was collected over a range of 1095 feet and data for (3) was collected for a range not over 50 feet. In both cases V is the velocity at a height H feet above the ground. An anemometer reading velocities must be used. It is probable that the general exponential equation is a

1 Robins Fleming, Wind Stresses in Tall Buildings, 41-43.



Wind Loading Diagrams
Figure 2

better expression for actual conditions than the straight line.

As was already stated the general equation used to determine the relation between pressure and velocity is $P = CV^2$. Not all investigators agree with the value of C equal to .004 as found by Professor Marvin; the range of variation is from .005 to .0032.[1]

In practice it is assumed that the wind pressure varies with the height above ground as a straight line. Figure 2 shows three illustrations of the assumed variation compared with the probable under ideal conditions. (a) and (b) are used for extremely tall buildings and (c) is used for moderate heights. The assumptions for (a) and (b) are valid because adjacent buildings tend to break the wind while the assumption of (c) is a confession of lack of knowledge of what takes place near the ground in built up districts. Building codes state the pressures that shall be used under the three assumptions. The pressures as given by the code is the highest expected under normal conditions. No attempt is made to design for hurricanes or tornadoes. Designs for such abnormal conditions as these would result in such expensive structures that they would be impossible economically.

1 Charles M. Spofford, Theory of Structures, 20.

CHAPTER II

METHODS OF CALCULATING WIND STRESSES BASED UPON AN EXACT METHOD

The distribution of wind stresses in a steel building is complex and statically indeterminate. Several methods which are theoretically correct have been developed, but practically all of them with the exception of the slope deflection method are unworkable for tall buildings, because of the complexity and number of equations involved. All of the so-called exact and approximate methods have the common assumption that the joints of a bent are absolutely rigid which is not strictly true. It is not impertinent to say that the only difference between the approximate and exact methods is the number of assumptions that are made. The accuracy of an approximate method varies directly with the initial assumptions over and above those made for the exact method . Generally speaking, we might say that an exact method is a solution for a statically indeterminate structure and an approximate method adds enough assumptions to enable the same structure to be solved statically.

SLOPE DEFLECTION METHOD

The slope deflection method is an exact method and is used to compare the accuracy of approximate methods.

The fundamental assumptions upon which the analysis is based are:[1]

1. The connections between the columns and girders are perfectly rigid.
2. The change in length of a member due to the direct stress is equal to zero.
3. The length of a girder is the distance between the neutral axis of the columns which it connects and the length of a column is the distance between the neutral axis of the girders which it connects.
4. The deflection of a member due to internal shearing stresses is equal to zero.
5. The wind load is resisted entirely by the steel frame.

The propositions upon which the general slope deflection method for any statically indeterminate figure depends are:[2]

1. When a member is subjected to flexure, the difference in the slope of the elastic curve between any two points is equal in magnitude to the area of the M/EI diagram for the portion of the member between the two points.
2. When a member is subjected to flexure, the distance of any point Q on the elastic curve, measured normal to initial position of member, from a tangent drawn to the elastic curve at any other point P is equal in magnitude to the first or statical moment of the area of the M/EI diagram between the two points, about the point Q.

For the proof of proposition 1 in the above refer to

1 W.M. Wilson, and G.A. Maney, Wind Stresses in the Steel Frames of Office Buildings, Bull No. 80, Eng. Exp. Sta. Univ. of Ill., 9.

2 W.M. Wilson, F.E. Richart, Camillo Weiss; Analysis of Statically Indeterminate Structures by the Slope Deflection Method, Bull. No. 108, Eng. Exp. Sta. Univ. of Ill., 10.

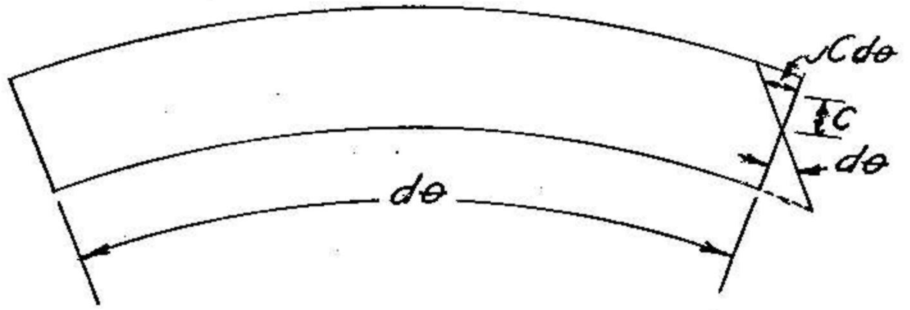


Figure 3

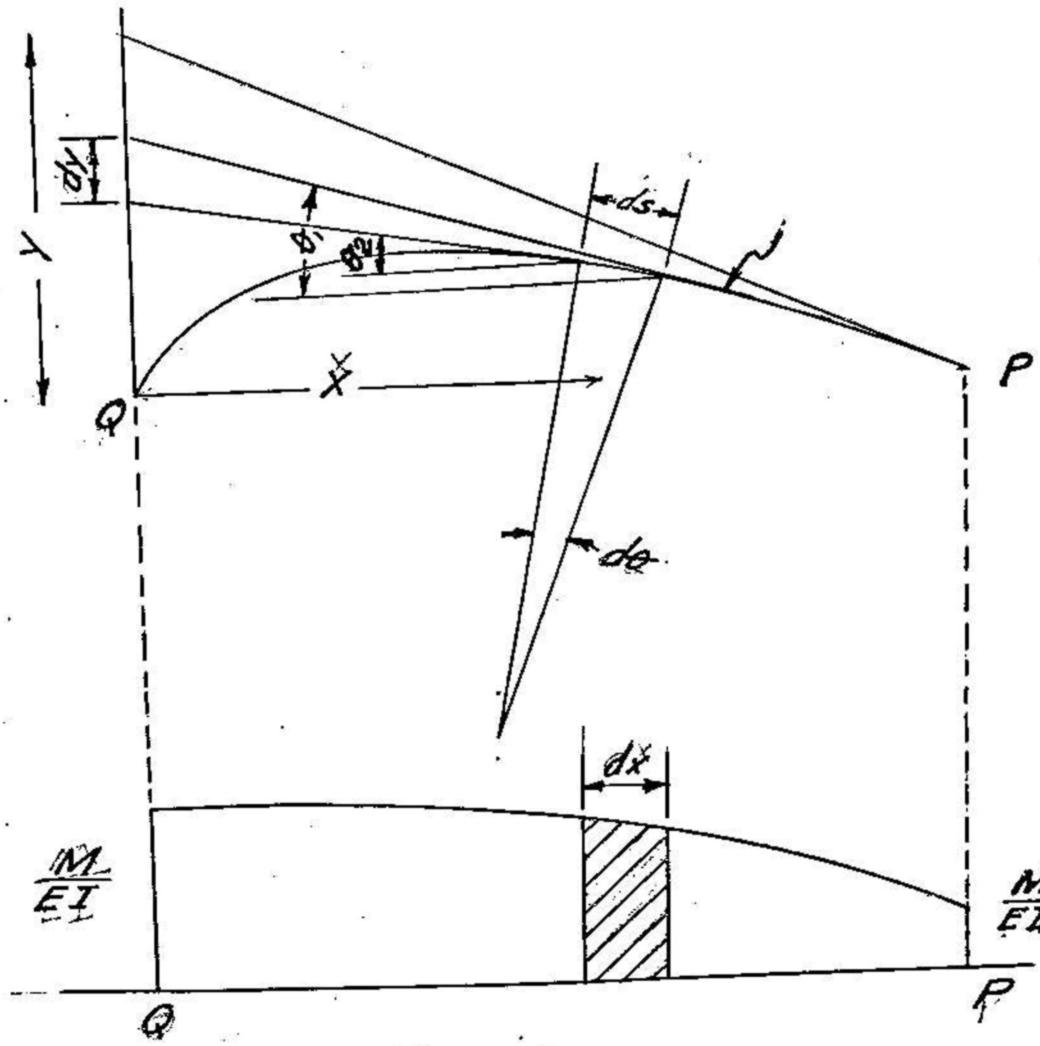


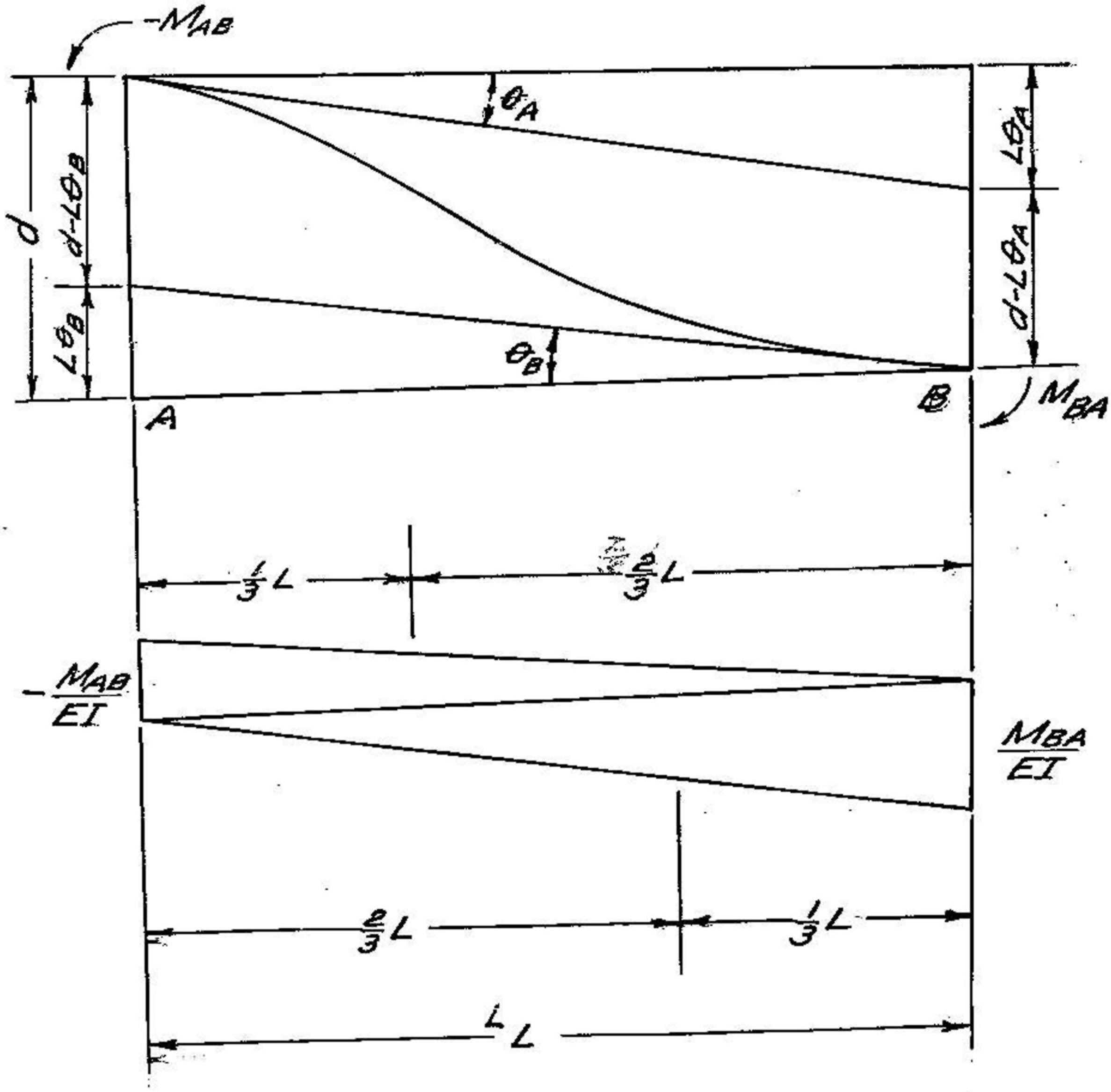
Figure 4

W.M. Wilson, F.E. Richart, Camillo Weiss, Analysis of Statically Indeterminate Structures by the Slope Deflection Method, Bull. No. 108, Eng. Exp. Sta., Univ. of Ill. 11

figure 3. The deformation at a distance c from the neutral axis is given by c and the unit deformation is $\frac{c\,d\theta}{ds}$. Since E = $\frac{\text{stress}}{\text{strain}}$ we have E $\frac{\frac{Mc}{I}}{\frac{c\,d\theta}{ds}}$ or $d\theta = \frac{M}{EI} ds$. We can assume dx to equal ds without appreciable error. Now $\int \frac{M}{EI} dx$ is the area of the shaded portion in figure 3. Hence the area of the diagram between any two points P and Q is $\int_P^Q \frac{M}{EI} dx$ and the difference in slope of the tangent to the elastic curve is $(\theta_1 - \theta_x) = \int d\theta = \int_P^Q \frac{M}{EI} dx$.

For proof of 2 refer to figure 4. It was assumed that ds dx. We can see from the figure that dy x but d is equal to $\frac{M}{EI} dx$ hence $y = \int_P^Q \frac{M}{EI} x\,dx$ but $\int_P^Q \frac{M}{EI} dx$ is equal to the area of the M/EI diagram between the points P and Q and x is the distance from Q to the portion of the diagram between P and Q hence we can say that y is equal in magnitude to the statical moment of that portion of the M/EI diagram about the point Q.[1]

[1] W.M. Wilson, F. E. Richart, Camillo Weiss; Analysis.



-MAB
d
d-LθB
LθB
θA
LθA
d-LθA
θB
A
B
MBA
1/3 L
2/3 L
-MAB/EI
MBA/EI
2/3 L
1/3 L
L

Figure 5

For the development of the dundamental slope deflection equations refer to figure 5.

From proposition 1:[1]

$\theta_B - \theta_A = -(M_{AB}/EI)(\frac{1}{2}L) + (M_{BA}/EI)(\ L)$

$\theta_B - \theta_A = L/2EI(-M_{AB} + M_{BA})$ (1)

From proposition 2:

$d - L(\theta_A) = (1/3)(L)(M_{BA}/EI)(\frac{1}{2}L) - (2/3)(L)(M_{AB}/EI)(\frac{1}{2}L)$

$d - L(\theta_A) = (M_{BA}/6EI)(L^2) - 2(M_{AB}/6EI)(L^2)$ (2)

Transposing (1) and multiplying (2) by $\frac{1}{L}$

$2EI(\theta_B - \theta_A) = L(-M_{AB} + M_{BA})$ (A)

$(6EI(d-L\theta_A) = L^2(-2M_{AB} + M_{BA}))\ 1/L$ (B)

Subtracting (B) from (A) we get

$2EI(\theta_B - \theta_A) - 6EId/L - 6EI\theta_A = M_{AB}L$ (A)

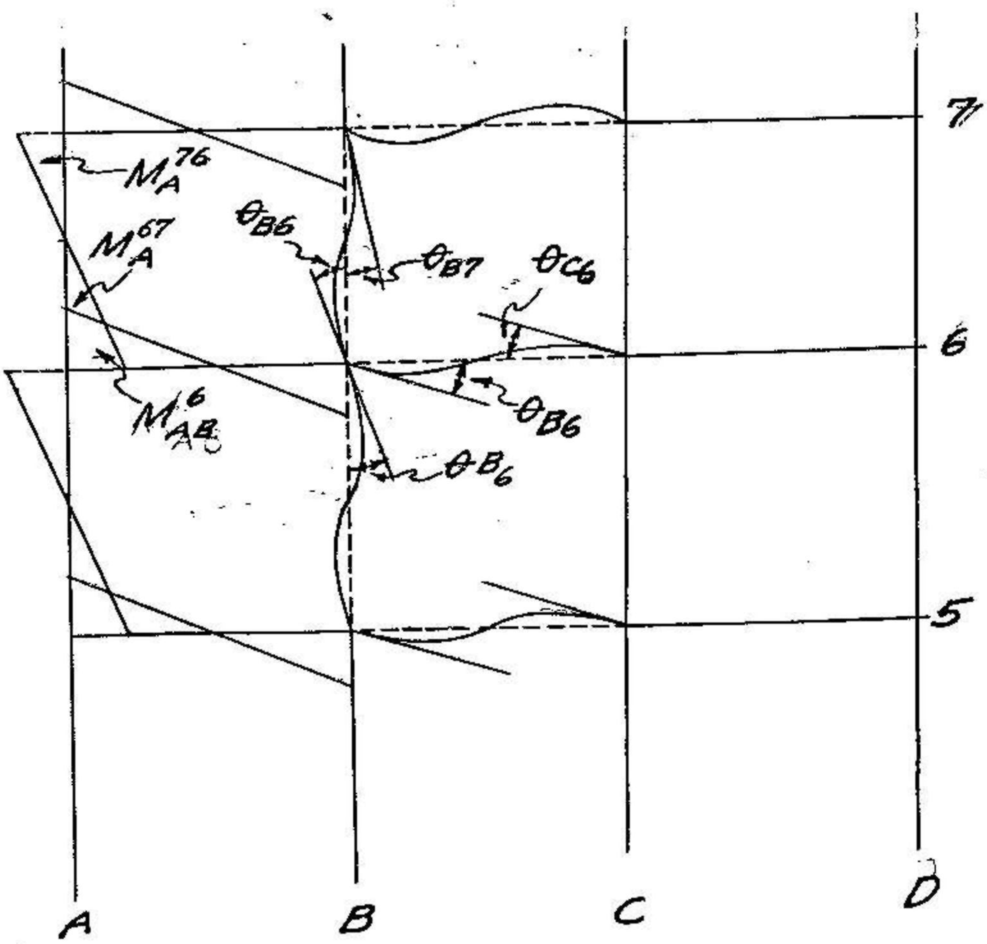
Let $R = d/L$ and $K = I/L$

$4EI\theta_A + 2EI\theta_B - 6EKd = M_{AB}L$

$M_{AB} = 4EK\theta_A + 2EK\theta_B - 6EKR$

$M_{AB} = 2EK(2\theta_A + \theta_B - 3R)$

<u>of Statically Indeterminate Structures by the Slope Deflection Method</u>, Bull. No. 108, Eng. Exp. Sta., Univ. of Ill., 10-15.



M_A^{76}
M_A^{67}
M_{AB}^{6}
θ_{B6}
θ_{B7}
θ_{C6}
θ_{B6}
θB_6
7
6
5
A
B
C
D

Figure 6

In the case that d = 0, R = 0 and we have

$$M_{AB} = 2EK(2\theta_A + \theta_B)$$

For the nomenclature used in the application of the slope deflection method refer to figure 6. Considering the columns of the sixth story we have:[1]

$$\Sigma(M_A^{67} + M_A^{76} + M_B^{67} + M_B^{76}) + W_6h = 0$$

Substituting the fundamental equation with the proper sub-subscripts:

$$2(2EK_{A_6}(2\theta_{A_6} + \theta_{A_5} - 3R_6) + 2EK_{A_6}(2\theta_{A_5} + \theta_{A_6} - 3R_6)$$
$$+ 2EK\ \ (2\theta_{B_6} + \theta_{B_5} - 3R_6) + 2EK_{B_6}(2\theta_{B_6} + \theta_{B_6} - 3R_6))$$
$$+ W_6h = 0$$

If we let $N = 2\Sigma I/h$, we have:

$$2K_{A_6}\theta_{A_5} + 2K_{B_6}\theta_{B_5} - N_6R_6 + 2K_{A_6}\theta_{A_6} + 2K_{B_6}\theta_{B_6} = - W_6h/6E \quad (1)$$

Considering point A_6 as a free body we have:

$$M_A^{67} + M_A^{65} + M_6^{AB} = 0$$

Substituting the fundamental equation with the proper subscripts:

$$2EK_{A_7}(2\theta_{A_6} + \theta_{A_7} - 3R_7) + 2EK_{A_6}(2\theta_{A_6} + \theta_{A_5} - 3R_6)$$
$$+ 2EK_{a_6}(2\theta_{A_6} + \theta_{B_6}) = 0$$

1 W.M. Wilson and G.A. Maney, Wind Stresses in the Steel Frames of Office Buildings, Bull. No. 80, Eng. Exp. Sta., Univ. of Ill. 16-17.

Letting $J_{A_6} = 2 \sum ((I/h) + (I/L))$ of all members that intersect at A_6:

$$K_{A_6}\theta_{A_5} - 3K_{A_6}R_5 + J_{A_6}\theta_{A_6} + K_{a_6}\theta_{B_6} - 3K_{A_7}R_7 + K_{A_7}\theta_7 = 0 \quad (2)$$

Considering point B of the sixth story we have:

$$M_6^{AB} + M_6^{BB} + M_B^{67} + M_B^{65} = 0$$

Substituting

$$2EK_{a_6}(2\theta_{B_6} + \theta_{A_6}) + 2EK_{b_6}(2\theta_{B_6} + \theta_{B_6}) + 2EK_{B_7}(2\theta_{B_6} + \theta_{B_7} - 3R_7) + 2EK_{B_6}(2\theta_{B_6} + \theta_{B_5} - 3R_6) = 0$$

Collecting we have:

$$2\theta_{B_6}(K_{a_6} + K_{b_6} + K_{B_7} + K_{B_6}) + \theta_{A_6}K_{a_6} + \theta_{B_6}K_{b_6} + \theta_{B_7}K_{B_7} + \theta_{B5}K_{B6} - 3R_7K_{B7} - 3R_6K_{B6} = 0 \quad (3)$$

For each story equations similar to (1), (2) and (3) can be set up by properly changing the subscripts. For each unbent there can be as many equations written as there are unknowns (O_A, O_B and R). There are three unknowns for each story. It is possible to solve for the unknowns but it is much simpler to use the numerical value of the coeffecients which necessitates a prelimenary design. If a member is inadequate, it must be changed and the entire calculations repeated. Obviously such a method is not practical for design work. However, the slope deflection method is used as a comparison for approximate methods.

APPROXIMATE METHOD OF W.M. WILSON AND G.A. MANEY[1]

This method is based upon the slope deflection method but additional assumptions are made enabling the stresses to be determined for any story independently of the others. The assumptions made in addition to those of the slope deflection method are:

> 1. The change in the slope at the top of a column in the story above and in the story below the one in which the stresses are to be determined, are equal to the change in slope at the top of the corresponding column in the latter story.
> 2. The ratio of the deflection to the length of the columns in the story above the one in which the stresses are to be determined, is equal to the ratio of the deflection to the length of the columns in the latter story.

Considering the previous equation (1), (2) and (3) it is assumed that θ_{A_5} and θ_{A_7} are equal to θ_{A_6} and that θ_{B_5} and θ_{B_7} are equal to θ_{B_6} and that R_7 is equal to R_6. Substituting in equations (1), (2) and (3) we have:

$$\theta_{A_5} = \theta_{A_7} = \theta_{A_6}$$

$$\theta_{B_5} = \theta_{B_7} = \theta_{B_6}$$

$$R_6 = R_7$$

$$- N_6 R_6 + 4K_{A_6}\theta_{A_6} + 4K_{B_6}\theta_{B_6} = W_6 h/6E \qquad (1)$$

1 W.M. Wilson and G.A. Maney, Wind Stresses in the Steel Frames of Office Buildings, Bull. No. 80, Eng. Exp. Sta. Univ. of Ill. 25.

$$- 3(K_{A_6} + K_{A_7})R_6 + (K_{A_6} + J_{A_6} + K_{A_6})\theta_{A_6} + K_{a_6}\theta_{B_6} = 0 \qquad \text{(II)}$$

$$- 3(K_{B_6} + K_{B_7})R_6 + K_{a_6}\theta_{A_6} + (K_{B_6} + J_{B_6} + K_{b_6} + K_{B_7})\theta_{B_6} = 0 \qquad \text{(III)}$$

These equations have been written for the sixth story and by changing the subscripts may be written for any other story. The K's and J's are known (from preliminary design) which gives us the equations in three unknowns which can be solved. W.M. Wilson and G.A. Maney have constructed a series of diagrams for columns and girders in terms of K ratios and percentage of wind moment which may be used instead of solving the above equations. The chief objection to this method is the need of a fairly accurate preliminary design. With other approximate methods this is not necessary and probably accounts for their popularity.

ROSS'S METHOD[1]

This method is based upon the slope deflection method also, and the results agree very closely. For an ideal bent the following are desirable:

1. The points on contraflexure of all members are at their midpoints.
2. All the direct stress is taken by the outside columns.

1 Albert Ward Ross, Jr. and Clyde T. Morris, "The Design of Tall Building Frames to Resist Wind", reprint from Proceedings, American Society of Civil Engineers, 1935.

The fundamental equation of the slope deflection method is:

$$M_{AB} = 2EK(2\theta_A + \theta_B - 3R)$$

Setting up equations in accordance with the conditions for an ideal bent: (refer to figure 7)

$$M_{a_1} = 2EK_{a_1}(3\theta) \; ; \; M_{b_1} = 2EK_{b_1}(3\theta) \; ; \; M_{c_1} = 2EK_{c_1}(3\theta)$$

$$M_{A_1} = 2EK_{A_1}(3\theta - 3R) \; ; \; M_{B_1} = 2EK_{B_1}(3\theta - 3R)$$

$$M_{C_1} = 2EK_{C_1}(3\theta - 3R) \; ; \; M_{A_0} = 2EK_{A_0}(3\theta - 3R)$$

$$M_{B_0} = 2EK_{B_0}(3\theta - 3R) \; ; \; M_{C_0} = 2EK_{C_0}(3\theta - 3R)$$

and

$$M_{a_1} = M_{A_1} + M_{A_0} \; ; \; M_{a_1} + M_{b_1} = M_{B_1} + M_{B_0}$$

$$M_{b_1} + M_{c_1} = M_{C_1} + M_{C_0}$$

Substituting:

$$2EK_{a_1}(3\theta) = 2EK_{A_1}(3\theta - 3R) + 2EK_{A_0}(3\theta - 3R)$$

$$2EK_{a_1}(30) + 2EK_{b_1}(3\theta) = 2EK_{B_1}(30 - 3R) + 2EK_{B_0}(3\theta - 3R)$$

$$2EK_{b_1}(3\theta) + 2EK_{c_1}(3\theta) = 2EK_{C_1}(3\theta - 3R) + 2EK_{C_0}(3\theta - 3R)$$

Eliminating the θ's and the R's and transposing:

$$K_{a_1}/(K_{A_1} + K_{A_0}) = (K_{a_1} + K_{b_1})/(K_{B_1} + K_{B_0}) =$$

$(K_{b_1} + K_{e_1})/(K_{C_1} + K_{C_0})$

From the above it is seen that the K's of a column above and below a floor must be proportional to the K's of the girders connected to it.

From the fundamental equation the moments in the girders with equal θ's is $M = 2EK(3\theta)$ and for columns $M = 2EK(3\theta - 3R)$. Since the entire direct stress caused by the wind is taken by the outside columns the stress on the inside columns is zero and the shears in the girders are equal at any floor level. The θ's are equal hence the point of contraflexure is at the center of the columns and girders. For the moment in the girders.

$$M = \text{shear}(L/2) = 2EK(3\theta)$$

$$\text{shear} = 4EK(3\theta)/L$$

Since E and θ are constant it follows that K/L is the variable. As pointed out the shear is constant for any floor hence K must be proportional to L. Therefore, the ideal bent would be proportioned as follows:

1. The sum of the K's of a column above and below a floor, must be proportional to the sum of the K's of the girders directly connected to it.
2. The K's of a girder must be proportional to their lengths.

Obviously, it is impossible to strictly comply with the above in practice as the dead and live loads of floors, walls, etc. are not proportional to the length of the supporting

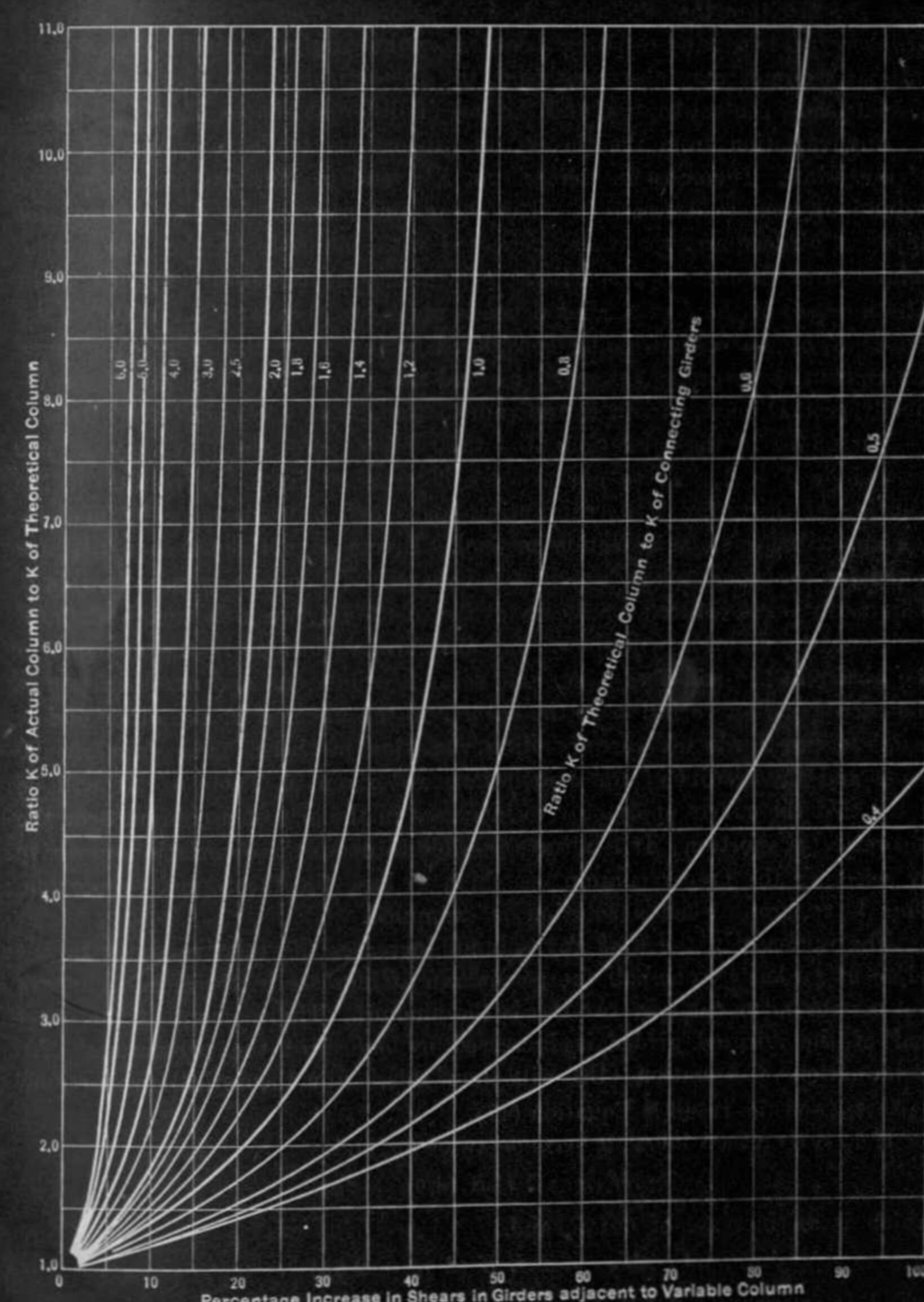


FIG. 7.—VARIATIONS IN MOMENTS AND SHEARS.

member. As the K's vary from the theoretical relationship the O's are affected. To enable the designer to approximate this change with relation to girder shear Mr. Ross made a diagram from data obtained by the solution of thirty-five bents of varying spans and proportions calculated by the approximate slope deflection method. The diagram has been photostated and is Figure 8. For a clearer idea of the effect of a change of θ upon the moment; if θ_B of M_{AB} $2EK(2\theta_A + \theta_B)$ (where $\theta_A = \theta_B$) increases 100% we have:

$$M_{AB} = 2EK(4O_B + O_B)$$

which is an increase in M_{AB} of 66-2/3 per cent.

and for the effect of a change of θ upon the shear:

$$\text{shear} = 4EK(3O_B)/L = 12EKO_B/L$$

for a 100% increase in θ

$$\text{shear} = 2EK(4O_B)/L + 2EK(5\theta_B)/L = 18EKO_B/L$$

which is an increase in shear of 50%.

For practical design Mr. Ross suggests the following routine:[1]

> 1. Calcualte the wind shear in each story from the assumed wind loads.
> 2. Calcualte the external wind moments, in the outside columns, assuming all of it to

[1] Albert Ward Ross Jr. and Clyde T. Morris, "The Design of Tall Building Frames to Resist Wind", reprint from Proceedings, May 1928, American Society of Civil Engineers, 1417.

be there and that the points of contraflexure of the columns are at their midpoints.

3. Calculate the column shears and moments, assuming that all the girders shears are equal and that the contraflexure points are at the middle of the members.

4. Calculate the dead and live load stresses in the columns.

5. Proportion the columns to carry the dead and live load from Item (4), together with the wind bending and direct stresses from Items (2) and (3).

6. With the girder shears from Item (3), calculate the wind moments in the girders and design the key girder at each floor for its wind moment combined with its dead and live load moment. The key girder is usually the shortest one. (Note that the maximum wind moment and and maximum dead and live moments do not occur at the same point.)

7. Proportion the other girders by their relative K's, which are proportional to the girder lengths taken directly from the dimensions of the bent. The sizes thus determined should be checked to see that the allowed unit stresses are not exceeded for dead and live load.

The preliminary design is complete but the members are not "Theoretically proportioned" hence it is necessary to determine the effect of this differance on the moments.

8. The relative theoretically proportioned K's of the columns are determined by adding the K's of the adjacent girders.

9. Determine the ratio of the sum of the actual K's above and below each floor to the theoretically proportioned K's of the columns. The one with the least ratio will then be termed theoretically proportioned.

10. Determine the ratio of the sum of the actual K's of the "Theoretically proportioned"

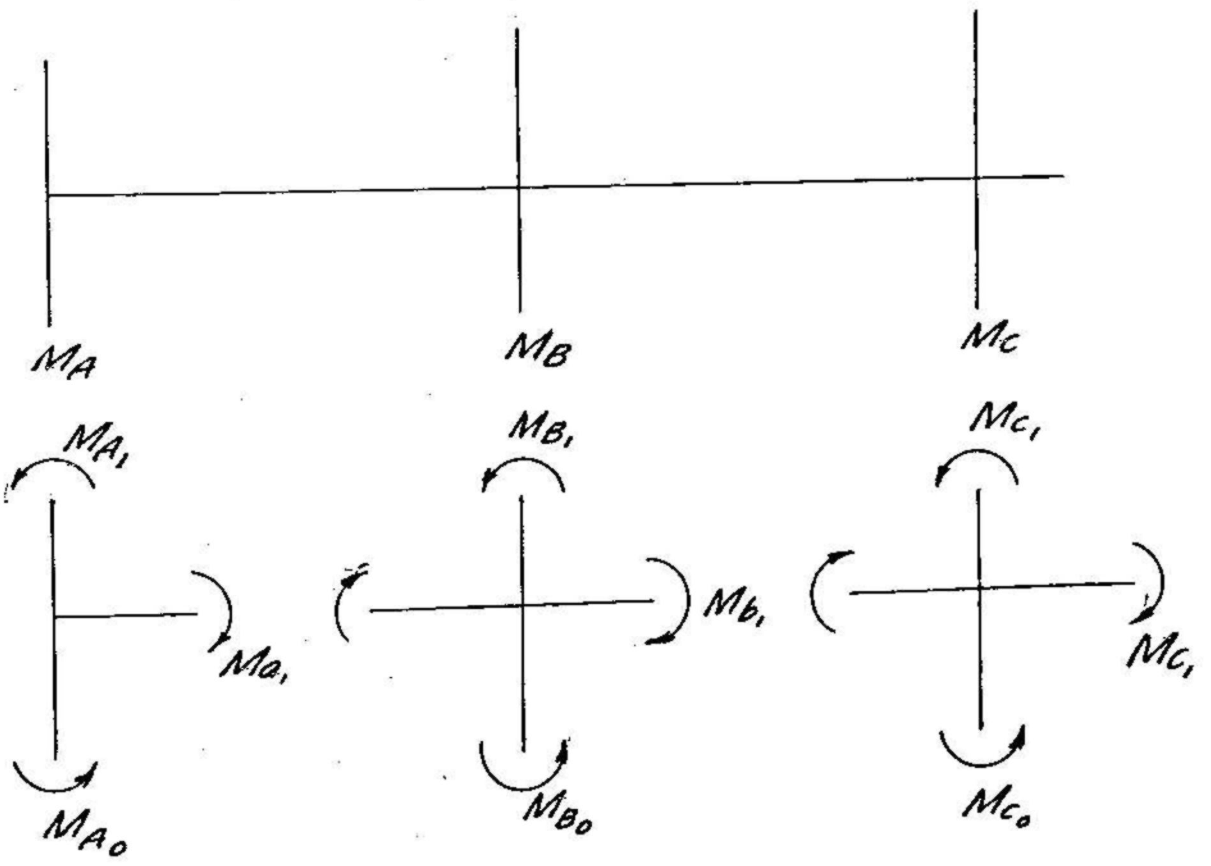


Figure 7

column above and below each floor, to the sum of the K's of the connecting girders.

11. With the ratios of the actual column K's to the theoretically column K's as determined for the columns which vary from theoretical proportions in item (9), and the ratios of the theoretical column K's to the K's of the adjacent girders, as determined in item (10) the relative variation in the shears and moments in the girders can be determined from figure 8.

12. For equal O's the relative moments in the girders are proportional to the girder K's.

13. Combining the relative moments found in items (11) and (12), the resulting proportionate moments for the girders are found.

14. From the relative moments found in item (13) the actual girder moments may be found, because the sum of the moments at the ends of all the girders at a floor is equal to shear in the story above times half its story height, plus the shear in the story below times half its story height. (This is approximate because the contraflexure points in the columns may not be at the mid-height.)

15. The moments in the columns may now be found by assuming that the ratio of the moment above a floor to that below is the same as the ratio of the story shears multiplied by the story heights. From figure 7, it then follows:

$$M_A^{12} = \frac{M_1^{AB}}{1 + \frac{S_0 L_0}{S_1 L_1}} \; ; \quad M_B^{12} = \frac{M_1^{BA} + M_1^{BC}}{1 + \frac{S_0 L_0}{S_1 L_1}} \; ; \quad M_C^{12} = \frac{M_1^{CB} + M_1^{CC}}{1 + \frac{S_0 L_0}{S_1 L_1}}$$

$$M_A^{21} = \frac{M_2^{AB}}{1 + \frac{S_2 L_2}{S_1 L_1}} \; ; \quad M_B^{21} = \frac{M_2^{BA} + M_2^{BC}}{1 + \frac{S_2 L_2}{S_1 L_1}} \; ; \quad M_C^{21} = \frac{M_2^{CB} + M_2^{CC}}{1 + \frac{S_2 L_2}{S_1 L_1}}$$

16. The girder shears are determined from the equation,

$$S_{AB} = \frac{M_{AB} + M_{BA}}{L}$$

17. The basement story column moments must be calculated from equation......

$$M_{01} - M_{10} = -2EK\theta_1$$

For the development of the last expression in the above we have for the moments at the top and bottom of the basement column, assuming that the connection to the foundation is rigid enough to prevent deflection:

$$M_{01} = 2EK(2\theta_0 + \theta_1 - 3R)$$

$$M_{10} = 2EK(2\theta_1 + \theta_0 - 3R)$$

By subtracting we get the desired expression since $\theta_0 = 0$.

CHAPTER III

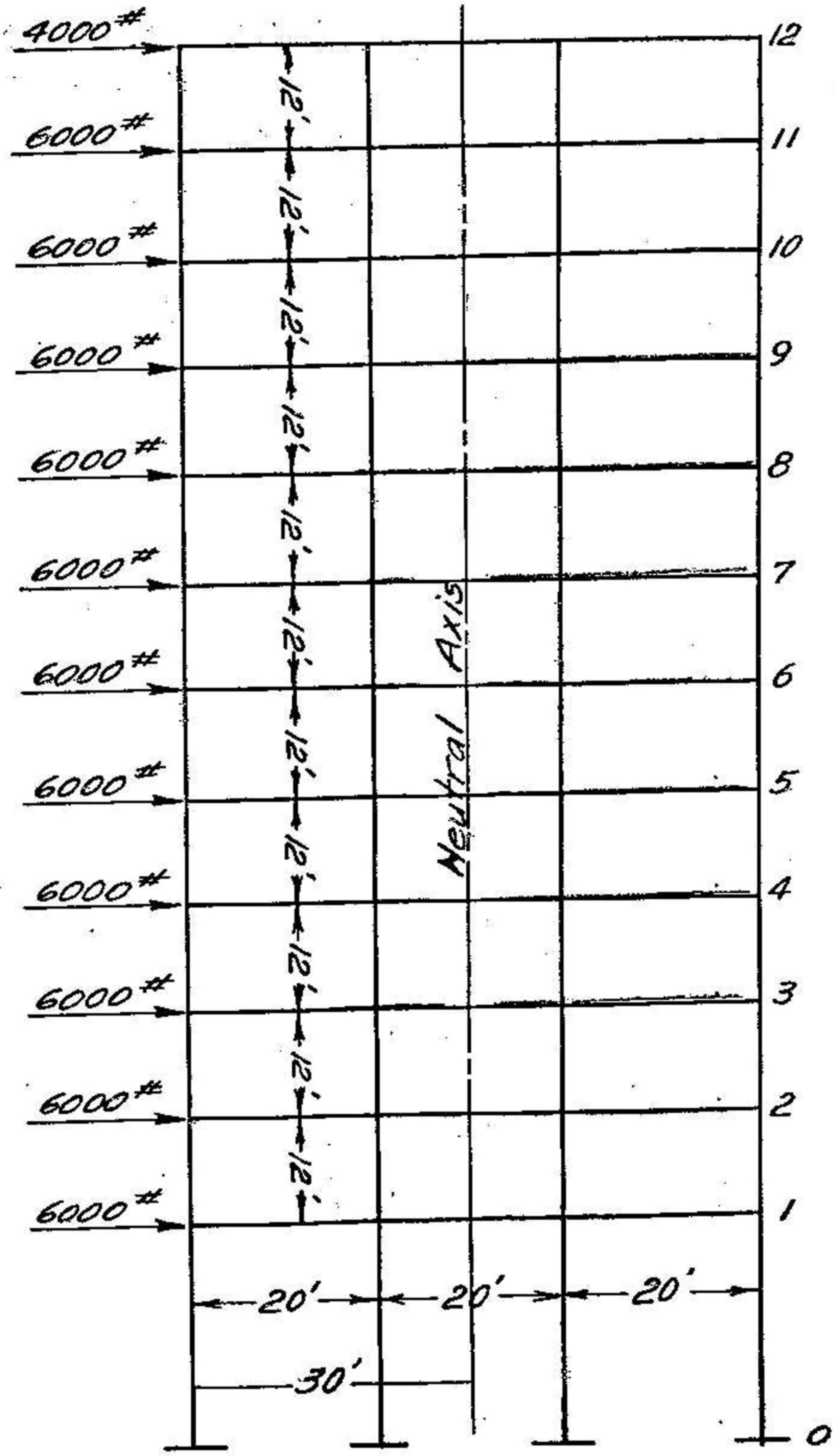
APPROXIMATE METHODS OF CALCULATING WIND STRESSES

FLEMING'S METHOD NO. I - CANTILEVER METHOD

The most important of the approximate methods are known as Fleming's methods I, II, II-A and III. They were given by him in an artical "Wind Bracing Without Diagonals for Steelframe Office Buildings" in Engineering News, Vol. LXIX of March 13, 1913 and became known by his name although as he points out they are not altogether original on his part.[1] In 1930 Mr. Fleming published "Wind Stresses in Buildings" which included methods I and II-A. In the later work method II-A was referred to as II. It is likely that more buildings have been designed by these four methods than with all other methods combined.

The greatest value of these methods is that they can be worked rapidly which is of the utmost importance when the actual cost in delay is taken into comsideration. Buildings are expensive, especially the extremely tall ones, and the interest on the financial outlay is a direct loss to the buyer until the investment is paying returns. Speed is necessary from the time the prospective

1 G.A. Hool and W. S. Kinne, Stresses in Framed Structures, 451.


4000#
6000#
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6000#
6000#
6000#
6000#
6000#
6000#
6000#
12
11
10
9
8
7
6
5
4
3
2
1
0
12'
Neutral Axis
20'
20'
20'
30'

Figure 9

building has been decided upon as representing a good investment.

The fundamental assumptions of all the Fleming methods are:[1]

> 1. A bent of a frame acts as a cantilever.
> 2. The point of contraflexure of each column is at mid-height of the story.
> 3. The point of contraflexure of each girder is at its mid-length.
> 4. The direct stress in a column is directly proportional to the distance from the column to the neutral axis of the bent.
> 5. The wind load is resisted entirely by the steel frame.

Consider the reactions of the columns to be proportional to their distance from the neutral axis of the building. The neutral axis is determined by the areas of the columns. In Figure 9 the neutral axis of the building is in the center since the building is symetrical. The wind loads are as shown in the figure. Since the reactions are assumed to be proportional to the distance from the neutral axis and X is the common factor of shear, the reactions in the columns will be 10X for B and C and 30X for A and D. Hence, for the eighth story we have:

$$6(6000) + 18(6000) + 30(6000) + 42(4000) = 10X(10) + 30X(30) + 30X(30) + 10X(10)$$

$$2000X = 492000$$

$$X = 246$$

1 Robins Fleming, Wind Stresses in Buildings, 103

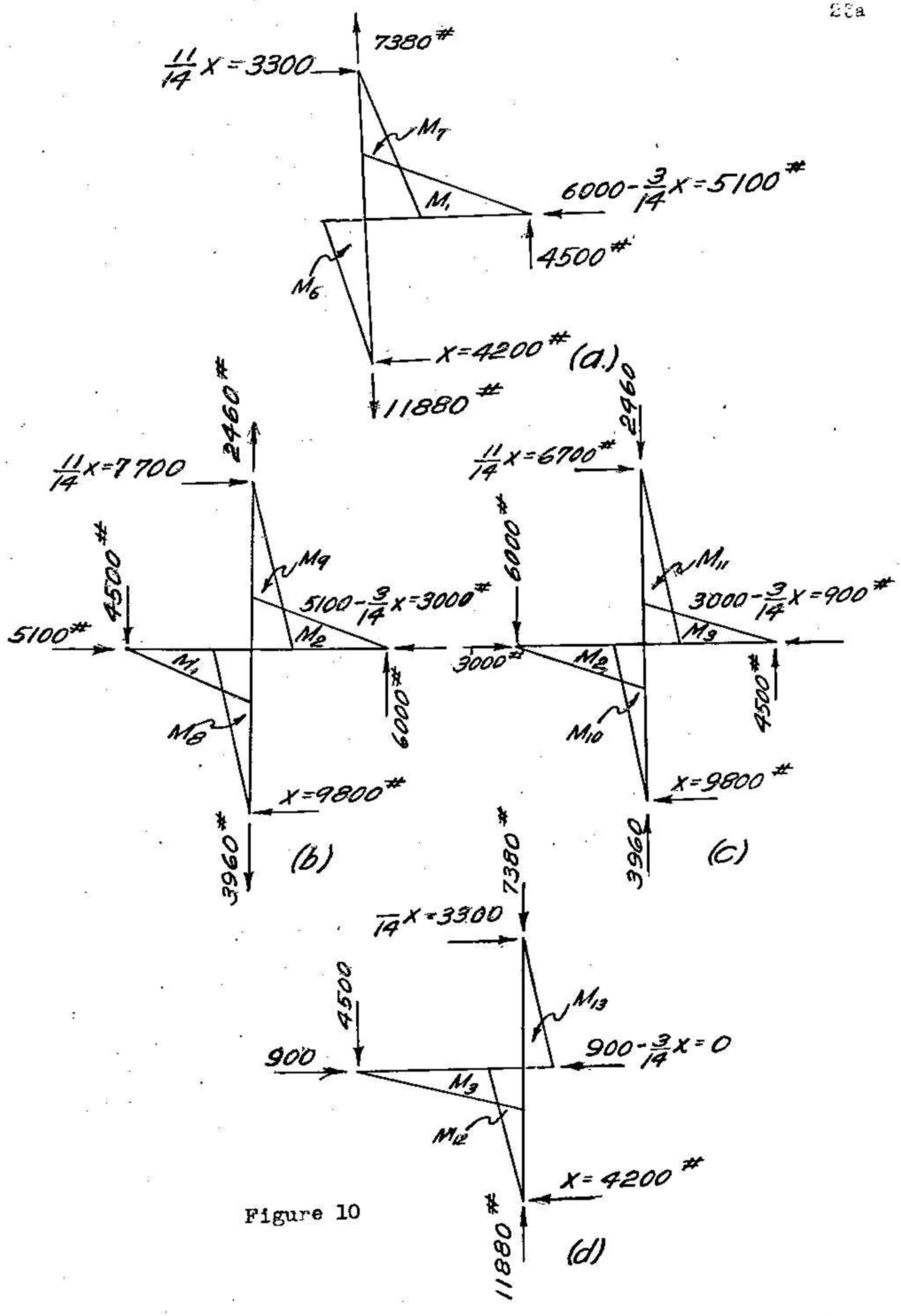

7380#
$\frac{11}{14}$X = 3300
M7
M1
6000 - $\frac{3}{14}$X = 5100#
4500#
M6
X = 4200#
(a)
11880#
2460#
$\frac{11}{14}$X = 7700
4500#
M9
5100 - $\frac{3}{14}$X = 3000#
5100#
M2
M1
M8
6000#
X = 9800#
3960#
(b)
2960
$\frac{11}{14}$X = 6700#
6000#
M11
3000 - $\frac{3}{14}$X = 900#
M3
3000#
M2
M10
4500#
X = 9800#
3960
(c)
7380#
$\frac{ }{14}$X = 3300
4500
M13
900 - $\frac{3}{14}$X = 0
900
M3
M12
X = 4200#
11880#
(d)

Figure 10

10X = 2460

30X = 7380

For the seventh story we have:

6(6000) + 18(6000) + 30(6000) + 42(6000) + 54(4000)

= 10X(10) + 30X(30) + 30X(30) + 10X(10)

2000X = 792000

X = 396

10X = 3960

30X = 11880

Now the difference in tension between the seventh and eighth stories in column A is taken up by shear on the girder. Considering this joint as a free body we have as shown in figure 10, a.

11880 - 7380 = 45000

The shear in the eighth story is 22000 and in the seventh story 28000 with 6000 acting at the joint. We then have as increments of shear in the seventh story X, (6/28)X and (22/28)X which act as shown in Figure 10. Thus taking moments around the point where X is applied.

(3/14)X(6) + (11/14)X(12) = 4500(10)

X = 45000/10.7 = 4200

(3/14)X = 900#

(11/14)X = 3300#

The moments in the girder and column are:

M_1 = 45000(10) = 45000#'

M_7 = 3300(6) = 19800#'

$M_6 = 4200(6) = 25200^{\#'}$

For the joint about B at the eighth story (Figure 10, b) we have a difference in tnesion in column B of:

$3960 - 2460 = 1500^{\#}$

This must be added to the shear in the girder ab since girder bc must carry in shear the difference in tension of column A and B.

$1500 + 4500 = 6000^{\#}$

Treating the joint as a free body and taking moments about the point where X is applied:

$(11/14)X(12) + (3/14)X(6) = 4500(10) + 6000(10)$

$X = 105000/10.7 = 9800$

$(3/14)X = 2100$

$(11/14)X = 7700$

$M_1 = 4500(10) = 45000^{\#'}$

$M_2 = 6000(10) = 60000^{\#'}$

$M_8 = 9800(6) = 58800^{\#'}$

$M_9 = 7700 \quad = 46200^{\#'}$

Considering the joint about C at the eighth story (Figure 10, c) we notice that on this side of the neutral axis the columns are in compression. Hence, the total shear on girder cd is:

$6000 - (3960 - 2460) = 4500$

The free body sketch is as shown and the equation is the same as for joint B.

$$(11/14)X(12) + (3/14)X(6) = 6000(10) + 4500(10)$$

$$X = 105000/10.7 = 9800$$

$$X = 105000/10.7 = 9800^{\#}$$

$$(3/14)X = 2100^{\#}$$

$$(11/14)X = 7700^{\#}$$

The moments are:

$$M_2 = 6000(10) = 60000^{\#'}$$

$$M_3 = 4500(10) = 45000^{\#'}$$

$$M_{10} = 9800(6) = 58800^{\#'}$$

$$M_{11} = 7700(6) = 46200^{\#'}$$

Joint D (Figure 10, d) is similar to joint A. The moment equation where X is applied is:

$$(11/14)X(12) + (3/14)X(6) = 4500(10)$$

$$X = 45000/10.7 = 4200$$

$$(3/14)X = 900^{\#}$$

$$(11/14)X = 3300^{\#}$$

$$M_3 = 4500(10) = 45000^{\#'}$$

$$M_{12} = 4200(6) = 25200^{\#'}$$

$$M_{13} = 3300(6) = 19800^{\#'}$$

It is interesting to note that the stress in girders ab, bc, cd decreases from 6000 to zero. This is true for any bent and gives the designer a check on his computations at each step.

In the preceding axample it was assumed that the columns were equally spaced. Suppose the dimensions of the

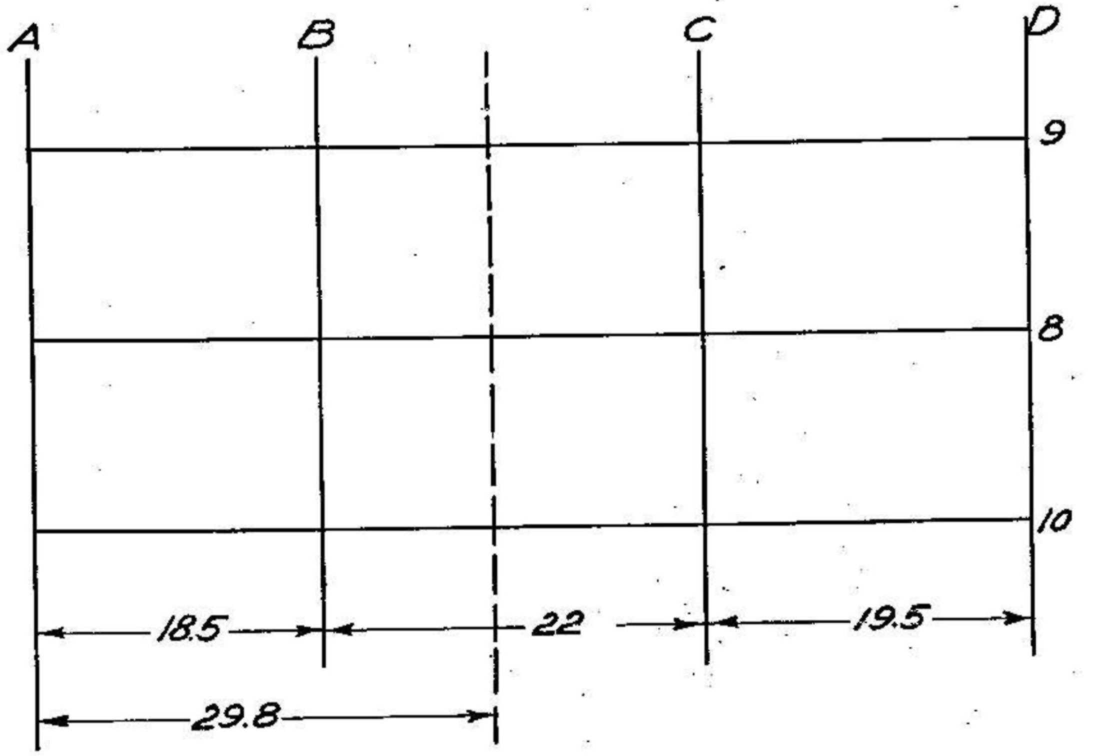


Figure 11

bent to be as shown in Figure 11 and that the sectional area of column A is 1A, column B is 2.5A, column C is 2.3A and column D is 1.1A. These relations are easily determined in practice from the dead and live loads as in Chapter I. Letting Y equal the distance of the neutral axis from the left we have:

$$1A(0) + 2.5A(18.5) + 2.3A(40.5) + 1.1A(60)$$

$$= (1A + 2.5A + 2.30A + 1.1A)Y$$

$$Y = (0 + 46.3 + 93.2 + 66)/(1 + 2.50 + 2.30 + 1.1)$$

$$= 205.5/6.90 = 29.8$$

For the determination of the forces on the columns, we have the moment due to the wind at the eighth story of 492000#' and that at the seventh story of 792000#' from the previous example. Letting F equal the stress in column A, we have from the assumption that the stresses in the columns are proportional to their distance from the neutral axis:

$$2.50(11.3)F/29.8 = \text{stress in column B}$$

$$2.30(10.7)F/29.8 = \text{stress in column C}$$

$$1.10(30.2)F/29.8 = \text{stress in column D}$$

Then:

$$492000 = F(29.8) + (2.50(11.3)F/29.8)11.3 +$$

$$(2.3(10.7F/29.8))10.7 + (1.1(30.2)F/29.8)30.2$$

$$492000 = 29.8F + 10.7F + 8.83F + 33.7F$$

$$83.0F = 492000$$

$$F = 5930$$

For the eighth story the stresses in the columns are:

Column A = 5930# tension

Column B = .948(5930) = 5620# tension

Column C = .825(5930) = 4900# compression

Column D = 1.11(5930) = 6610# compression

the seventh story is computed in the same manner:

$$83.0F = 792000$$

$$F = 9550$$

Column A = 9550# tension

Column B = .948(9550) = 9050# tension

Column C = .825(9550) = 7880# compression

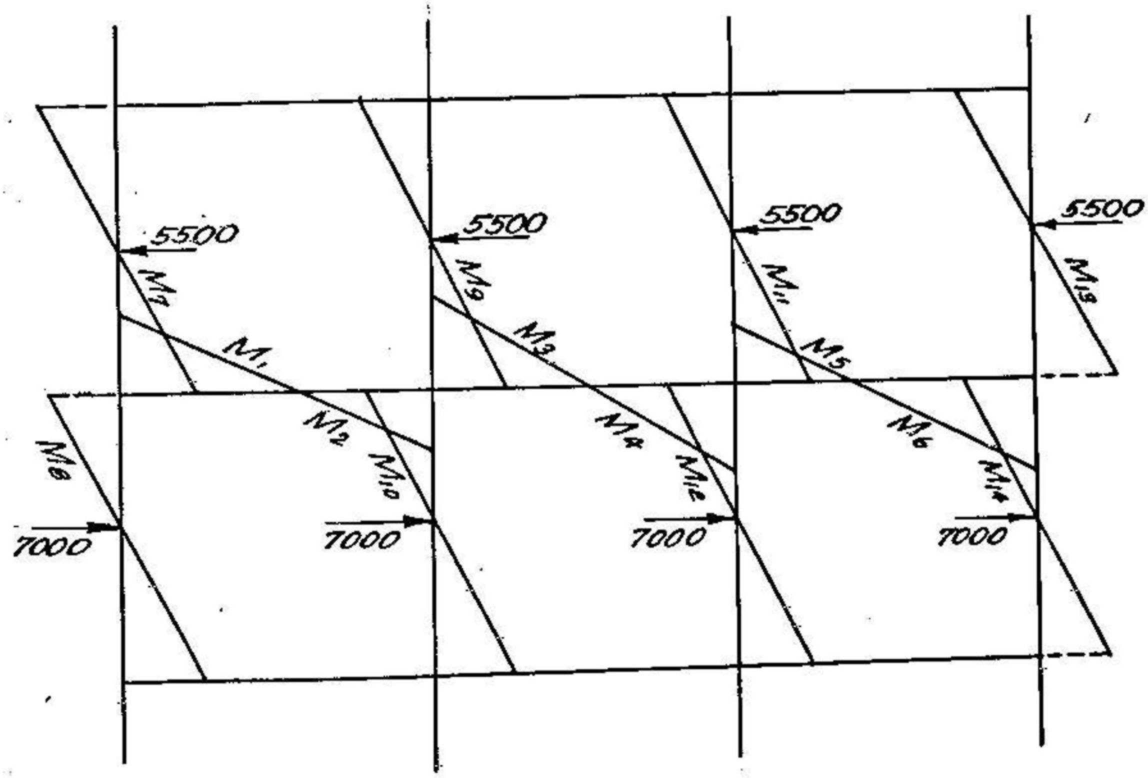
Column D = 1.11(9550) = 10600# compression

All computations made with the slide rule.

METHOD II; METHOD OF EQUAL SHEARS[1]

In addition to the assumptions of method II the total horizontal shear is regarded as being distributed among the columns proportional to their moments of inertia. The compression and tension on the columns is calculated by the bsy and since the stresses on the two middle columns are equal but of opposite sign, it is assumed that

1 G.A. Hool and W.S. Kinne, Stresses in Framed Structures, 457-458.



5500
5500
5500
5500
M7
M9
M11
M13
M1
M3
M5
M2
M4
M6
M8
M10
M12
M14
7000
7000
7000
7000

Figure 12

the outside columns take all the stress.

For the seventh story the stresses are:

$6(6000/3) + 18(6000/3) + 30(6000/3) + 42(6000/3) +$

$54(4000/3) = 20X$

$$60X = 492000$$

$$X = 8200$$

The stresses in the outside columns of the seventh story are:

$6(6000/3) + 18(6000/3) + 30(6000/3) + 42(6000/3) +$

$42(4000/3) = 20X$

$$60X = 792000$$

$$X = 132000$$

The total horizontal shear in the eighth story is 22000 or 5500 per column and in the seventh story is 28000 total or 7000 per column. The moments are:

(refer to Figure 12)

$M_1 = 5500(6) + 7000(6) = 75000^{\#'}$

$M_2 = 5500(6) + 7000(6) + (13200 - 8200)20$

$M_2 = 75000 + 100000 = - 25000^{\#'}$

$M_3 = 2(5500(6) + 7000(6)) - (13200 - 8200)20$

$M_3 = 150000 - 100000 = 50000^{\#'}$

$M_4 = 2(5500(6) - 7000(6)) - (13200 - 8200)40$

$M_4 = 150000 - 200000 = -50000^{\#'}$

$M_5 = 3(5500(6) - 7000(6)) - (13200 - 8200)40$

$M_5 = 225000 - 200000 = -25000^{\#'}$

$M_6 \quad 3(5500(6) - 7000(6)) - (13200 - 8200)60$

$M_6 = 225000 - 300000 = -75000^{\#\prime}$

The column moments are:

$M_8 = M_{10} = M_{12} = M_{14} = -7000(6) = -42000^{\#\prime}$

$M_7 = M_9 = M_{11} = M_{13} = 5500(6) = 30000^{\#\prime}$

The compression in the floor girders from the wind is:

$$ab = 6000 - 1500 = 4500^{\#}$$

$$bc = 4500 - 1500 = 3000^{\#}$$

$$cd = 3000 - 1500 = 1500^{\#}$$

In the case of a bent with unevenly spaced columns it is assumed that the horizontal shear is distributed among the columns according to the span length of the bays. This is readily seen; since, if the assumption were not made, the stresses from adjacent bays upon an interior column would be equal and hence the interior column would be stressed. By taking the wind loading proportional to the span length of the bays the stresses upon an interior column are equal in amount but opposite in sign and their algebraic sum is then zero.[1]

METHOD II-A, PORTAL METHOD

In this method the structure is regarded as being a series of portals each independent of the other.

[1] G.A. Hool and W.S. Kinne, Stresses in Framed Structures. 457-458.

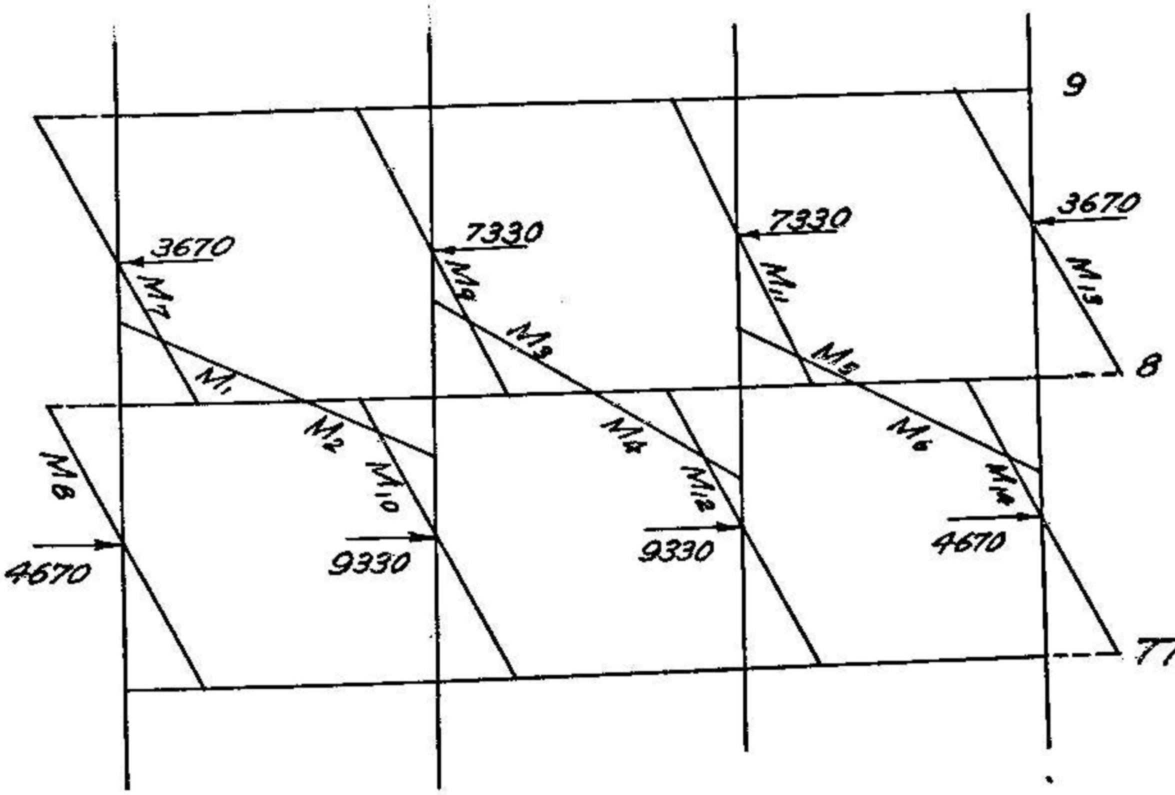


Figure 13

Thus, the total horizontal shear is divided by the number of bays and the shear on an outside column is half that any other. For the eighth story refer to figure 13. The shears in the eighth story are:

22000/3 = 7330#

Column A = 3670#

Column B = 7330#

Column C = 7330#

Column D = 3670#

For the seventh story the shears are:

28000/3 = 9330

Column A = 4670#

Column B = 9330#

Column C = 9330#

Column D = 4670#

The compression and tension in the outside columns due to the wind are for the seventh story:

6(6000) + 18(6000) + 30(6000) + 42(6000) + 54(4000) = 60X

60X = 792000

X = 13200# compression and tension

The outside column stresses in the eighth story are:

6(6000) + 18(6000) + 30(6000) + 42(4000) = 60X

60X = 492000

X = 82000# compression and tension

The bending moments in the girders are:

$M_1 = 3670(6) + 4670(6) = 50000^{\#'}$

$M_2 = 3670(6) + 4670(6) - (13200 - 8200)20 = -50000^{\#'}$

$M_3 = 3670(6) + 4670(6) + 7330(6) + 9330(6)$

$- (13200 - 8200)20 = 50000^{\#'}$

$M_4 = 3670(6) + 4670(6) + 7330(6) + 9330(6)$

$- (13200 - 8200)40 = -50000^{\#'}$

$M_5 = 3670(6) + 4670(6) + 2(7330)6 + 2(9330)6$

$- (13200 - 8200)40 = 50000^{\#'}$

$M_6 = 3670(6) + 4670(6) + 2(7330)6 + 2(9330)6$

$- (13200 - 8200)60 = -50000^{\#'}$

The moments in the columns are:

$M_7 = M_{I3} = 3670(6) = 22000^{\#'}$

$M_9 = M_{11} = 7330(6) = 44000^{\#'}$

$M_8 = M_{14} = 4670(6) = 25000^{\#'}$

$M_{10} = M_{12} = 9330(6) = 56000^{\#'}$

The compressions in the girders are:

$ab = 6000 - 1000 = 5000^{\#}$

$bc = 5000 - 2000 = 3000^{\#}$

$cd = 3000 - 2000 = 1000^{\#}$

If the bays are unequally spaced as in Figure 11, it is assumed that the shears in the columns are proportional to the moments of inertia of the columns. Letting the respective moments of inertia be II, 2.5I, 2.3I and 1.II we have: (Note that the results from this ratio of moments of inertia of columns is not comparable to the results in method I which had the same ratio of areas because the moments of inertia of sections are not necessarily proportional to the area.)

$$IX + 2.5X + 2.3X + 1.1X = 22000$$

$$6.9X = 22000$$

$$X = 3190$$

Shears in columns of the eighth story are:

Column A = 1.0X = 3190#

Column B = 2.5X = 7980#

Column C = 2.3X = 7340#

Column D = 1.1X = 3510#

Shears in columns of seventh story are:

$$6.9X = 28000$$

$$X = 4060$$

Column A = 1.0X = 4060#

Column B = 2.5X = 10100#

Column C = 2.3X = 9340#

Column D = 1.1X = 4460#

The moments in the girders and columns are computed in a similar manner to those of the previous example.

METHOD III, CONTINUOUS PORTAL METHOD

The tension and compression in the columns is distributed as in method I. For the seventh story:

$$6(6000) + 18(6000) + 30(6000) + 42(6000) + 54(6000)$$

$$= 10X(10) + 30X(30) + 30X(30) + 10X(10)$$

$$2000X = 792000$$

$$X = 396^{\#}$$

$$10X = 3960^{\#}$$

$$30X = 11880^{\#}$$

For the eighth story:

$$6(6000) + 18(6000) + 30(6000) + 42(4000) =$$

$$10X(10) + 30X(30) + 30X(30) + 10X(10)$$

$$2000X = 492000$$

$$X = 246^{\#}$$

$$10X = 2460^{\#}$$

$$30X = 7380^{\#}$$

The shears in the girders is assumed to be as in method I:

$$\text{Shear in ab} = 11880 - 7380 = 4500^{\#}$$

$$\text{Shear in bc} = 3960 - (2460 - 4500) = 6000^{\#}$$

$$\text{Shear in cd} = 6000 - (3960 - 2460) = 4500^{\#}$$

The horizontal shear is assumed to be evenly distributed among the columns as in Figure 12. The shear in the eighth story is 22000 or 5500 per column and in the seventh story 28000 or 7000 per column. The moments in the girders are:

$$M_1 = 5500(6) + 7000(6) = 75000^{\#\prime}$$

$$M_2 = 5500(6) + 7000(6) - (11880 - 7380)20 = -15000^{\#\prime}$$

$$M_3 = 2(5500)6 + 2(7000)6 - (11880 - 7380)20 = 60000^{\#\prime}$$

$M_4 = 2(5500)6 + 2(7000)6 - (3960 - 2460)20$

$- \quad (11880 - 7380)40 = -60000^{\#\prime}$

$M_5 = 3(5500)6 + 3(7000)6 - (3960 - 2460)20$

$- (11880 - 7380)40 = 15000\ '$

$M_6 \quad 3(5500)6 - 3(7000)6 - (3960 - 2460)40$

$- (11880 - 7380)60 - 3960 - 2460)20 \quad -75000\ '$

The moments in the columns are:

$M_7 = M_9 = M_{11} = M_{13} = 5500(6) = 33000^{\#\prime}$

$M_8 = M_{10} = M_{12} = M_{14} = 7000(6) = 42000^{\#\prime}$

The compressions in the floor girders are:

$ab = 6000 - 1500 = 4500^{\#}$

$bc = 4500 - 1500 = 3000^{\#}$

$cd = 3000 - 1500 = 1500^{\#}$

If the columns are not evenly spaced but are similar to Figure 11, the horizontal shear is taken proportional to the moments of inertia of each column as in method II-A and the compression and tension in the columns to vary as the sectional area and distance from the neutral axis as in method I.[1]

1 G.A. Hool and W.S. Kinne, Stresses in Framed Structures, 462-464.

CHAPTER V

DESIGN DETAILS

No matter how carefully the wind, live and dead stresses are determined, the finished structure is no better than its details. The detailer has no set rule to go by and must on occasion tax his ingenuity to the utmost to meet the many conditions of satisfactory design.

For combined loading the A.I.S.C. Standard Specifications read:[1]

> Members subject to both direct and bending stresses shall be so proportioned that the greatest combined stresses shall not exceed the allowed limits.

This means that the sum of the direct stress and extreme fiber stress due to bending shall not exceed a value commensurate with that for a member of the gieen L/r ratio. The A.I.S.C. column formula allows 15000 per square inch up to 60 L/r and beyo d 60 L/r the permissible unit stress is $S = \frac{18000}{1 - (L^2/1800r^2)}$.[2] The bending stress is determined by S Mc/I which is added to the stress on the member the sum of which must not exceed that given above for

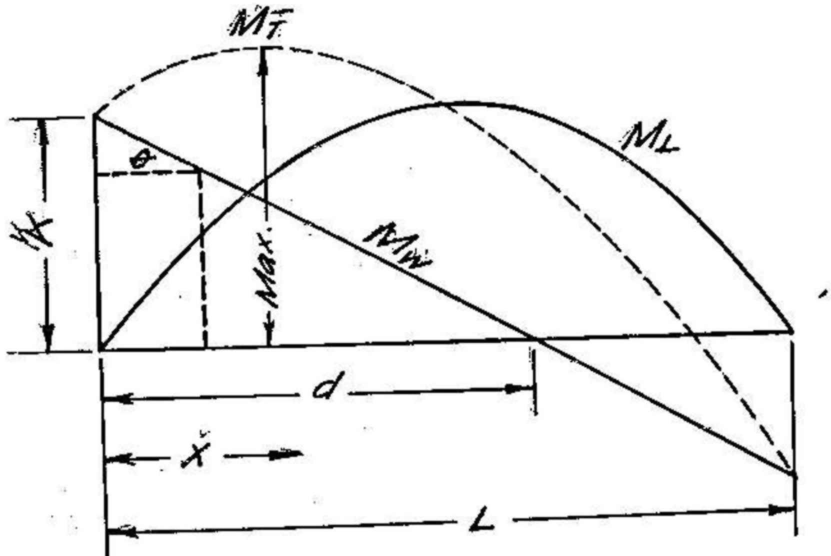
1 American Institute of Steel Construction Inc. Steel Construction, 11 and 9.

2 Ibid.

columns. Obviously, for design purposes a tentative column must be selected and finally chosen by trial and error. It is permissible to increase the allowable unit stresses when used for wind load alone. These increased unit stresses will be taken up in detail later in the chapter.

Girders are selected in accordance with their section modulus as was done in chapter I and checked as already explained if they are subject to a direct stress. When standard rolled sections are too small, girders must be built up. This is also true for columns. Listed in the different handbooks are the section moduli for various built up girders, but the designer may frequently have conditions which warrant special design.

The moment that the girder must resist is known, and the architectural conditions often limit the depth; hence, the maximum depth is known as is the span. The required moment of inertia may be approximated fairly close by I Mc/S. It is well to increase the moment of inertia by one or two per cent, depending upon the depth, size of rivots to be used and bending moment. The rivot holes shift the neutral axis off center and the moment of inertia of the finished girder must be checked to see that it is no less than that required.



$$M_L = \frac{WLX}{2} - \frac{WX^2}{2} \text{ (Dead + live load moment)}$$

$$M_W = K - x\tan\theta \text{ (Wind load moment)}$$

$$\tan\theta = \frac{K}{d}$$

$$M_W = K - \frac{KX}{d}$$

$$M_T = M_W + M_L = \frac{WLX}{2} - \frac{WX^2}{2} + K - \frac{KX}{d}$$

$$\frac{dM_T}{dx} = \frac{WL}{2} - WX - \frac{K}{d}, \quad X = \frac{L}{2} - \frac{K}{Wd}$$

$$M_T = \frac{WL}{2}\left(\frac{L}{2} - \frac{K}{Wd}\right) - \frac{W}{2}\left(\frac{L}{2} - \frac{K}{Wd}\right)^2 + K - \frac{K}{d}\left(\frac{L}{2} - \frac{K}{Wd}\right)$$

$$M_T = \left(\frac{WL}{2} - \frac{K}{d}\right)\left(\frac{L}{2} - \frac{K}{Wd}\right) - \frac{W}{2}\left(\frac{L}{2} - \frac{K}{Wd}\right)^2$$

Figure 14

There are two moment diagrams that affect the girder; namely, the live and dead load which is a parabola and the wind load which is two triangles. (see Figure 14) The amount that the wind load affects the design of the girder depends upon tbe position of the point of contraflexure. It is well to investigate the resultant moment diagram obtained by adding the two diagrams superimposed. It is noticed that the resultant moment diagram is greater on one side of the span than on the other, but when the wind reverses, the diagrams are reversed. Beneath the diagram is the development of two formulas which may be used to determine the point at which the resulta t moment diagram is a maximum and the amount. Built up girders may be of the box or "I" type. The method of calculation for each type is the same and an example of the latter is given.

Assume total load to be 90000#

Let h = 14"

Let L = 20'   assuming negligible wind moment

t = thickness of web = 14/60 = .233 (In accordance with A.I.S.C. specifications)[1] Use 3/8" plate

$I = Mc/S = 45000(12)(1/8)(7)/18000 = 525 in^4$

I_p = moment of inertia of web = $(1/12)bd^3$

$= (1/12)(3/8)(2740) = 85.5 in.^4$

Required moment of inertia of four angles about neutral

1 American Institute of Steel Construction Inc., Steel Construction, 11.

axis = 525 - 85.5 = 439

Increasing 1.15% gives 505

I - 4"x3"x1/2" angle has I = 2.42, A = 3.25, X = .83

The 4" leg is used as the outer most fiber of the girder glange and X is the distance from this leg to the neutral axis of the angle.

$I_n = I_A - d^2A = 2.42 - (7 - .83)^2 3.25 = 128$

$4(128) = 512$

Use 4 - 4"x3"x1/2" angles

Spacing stiffners in accordance with A.I.S.C. specifications.[1]

$85t(18000(A/V) - 1)^{\frac{1}{2}} = 85(3/8)(18000(14)(3/8)/45000 - 1)^{\frac{1}{2}}$

= 136"   Use 5' spacing for stiffners

Selection of stiffner angles:

$V/S = 45000/15000 = 3 \text{ in}^2$

Use 2 - 3"x3"x1/2" angles for stiffners in pairs A = 2.75.

I = 2.22, X = .93

$I_1 = I_2 - d^2A = 2.22 - (2.75)(.75 - .93)^2/2 = 4.16$

d = 14"   2(I) = 2(416) = 8.32

15000 per square inch is permissible if L/r is less than 60.

$L/r = 14/(8.32/5.50)^{\frac{1}{2}} = 11.4$ original assumption was correct.

Rivets for stiffners:

Bearing value of 3/4" rivets on 3/8" plate = 8440#

1 American Institute of Steel Construction Inc., <u>Steel Construction</u>, 11.

Shearing value of 3/4" rivets = 11930#

45000/8440 = 5.33

The resulting spacing of 6 - 3/4" rivets would be too close. Use two pairs of stiffner angles at each end of the girder with four rivets in each pair. Five feet out from the end the shear in the girder is: 45000/2 = 22500#

22500/8440 = 2.67

Use 4 rivets for inside stiffners

1/2" fillers should be used between stiffners and web.

Computing spacing of 3/4" flange rivets at entervals of 3'. $v_1 = V_1Q/bI = 45000Q/(3/8)525$

Q = statical moment of flange angle areas about neutral axis of girder = 2(3.25)(7 - .83) = 39.4

v_1 = 45000(39.4)/.375(525) = 9000#

Using the same unit stresses as before:

8440/9000 = .938

This value is less than 3 times the diameter of the rivet and by the A.I.S.C. specifications can not be used.[1] We have the choice of choosing larger angles or increasing the web thickness. The smallest angle which will permit two rows of rivets is 5". Hence, it is more economical to increase the size of the plate.

Bearing of 3/4" rivets on 5/8" plate = 11930#

Shearing of 3/4" rivets = 11930#

1 American Institute of Steel Construction, <u>Steel Construction</u>, 12-13.

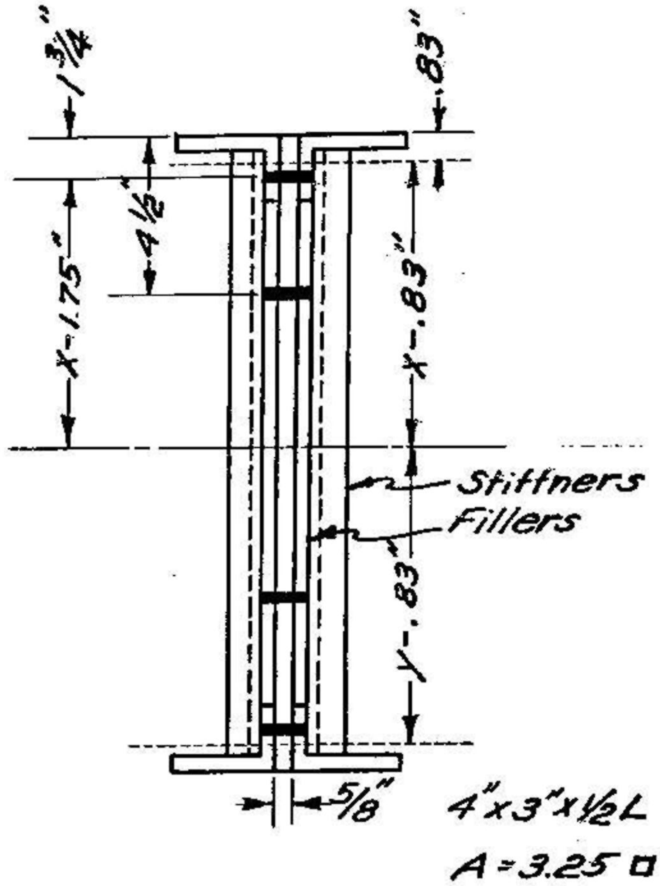


Figure 15

$v_1 = 45000(39.4)/(5/8)(525) = 5560^{\#}$ $11930/5560 = 2.14''$

Space rivets $2\frac{1}{4}''$ c-c for first 3'.

$45000 - 3(4500) = 3150$

$v_2 = 315000(39.4)/(5/8)525 = 3910$

$$11900/3910 = 3.04''$$

Space rivets 3" c-c for balance of distance.

The assumed neutral axis is not the true, since the rivet holes on the tension side of the neutral axis are deducted from the gross area and shift it from the center. Determining the true neutral axis: (refer to Figure 15).

$$6.5(y - .83) + (5/8)(y)(y/2) = 6.5(x - .83) + (5/8)(x)(x/2) - (3/4)(1\tfrac{5}{8})(x - 1.75) - (3/4)(5/8)(x - 4\tfrac{1}{2}) \quad \text{(I)}$$

$$x + y = 14 \quad \text{(II)}$$

Solving these two equations we get:

$$x = 7.36$$

$$y = 6.64$$

Checking the moment of inertia of the girder:

$I(\text{for angles}) = 2.42 - (6.64 - .83)^2 3.25 = 109\ in^4$

$I(\text{for web}) = (5/8)(6.64)^3/3 = 60.8\ in^4$

$\text{Total } I = 2(2(109) - 60.8) = 558\ in^4$

$\text{Required } I = 525\ in^4$

The member is designed for a moment of inertia of 525 and even though it was found that the actual moment of inertia is 558 the load should not exceed 90000 because the flange rivets were computed for the first value.

JOINT DESIGN

In all details where the only stresses are wind it is permissible to increase the unit stresses. Robins Fleming recommends the following unit stresses:[1]

Tension, rolled steel net section	-- 24000
Compression incolumns, gross section (with a maximum of 20000)	---- 24000 - 70l/r
Shear on gross area of webs of beams and girders where height between flanges is not more than 60 times the thickness of web(or where webs have stiffners)	----- 16000
Shear on gross area of web when h/t exceeds 60 (without stiffners)	20000 - 70h/t
Shear in power driven rivets	-- 18000
Shear in hand driven rivets	-- 13500
Bearing upon power driven rivets	-- 32000
Bearing upon hand driven rivets or rough bolts	-- 27000
Rivets in direct axial tension	-- 16000

In the case of combined live, dead and wind stresses none of the above unit stresses are to be used if the resulting section is smaller for the live and dead loads than it would be if the ordinary values were used.

As said before, there is no set rule that the designer can follow and his decision as to the type of joint to be used depends upon the particular conditions

1 Robins Fleming, Wind Stresses in Buildings, 120.

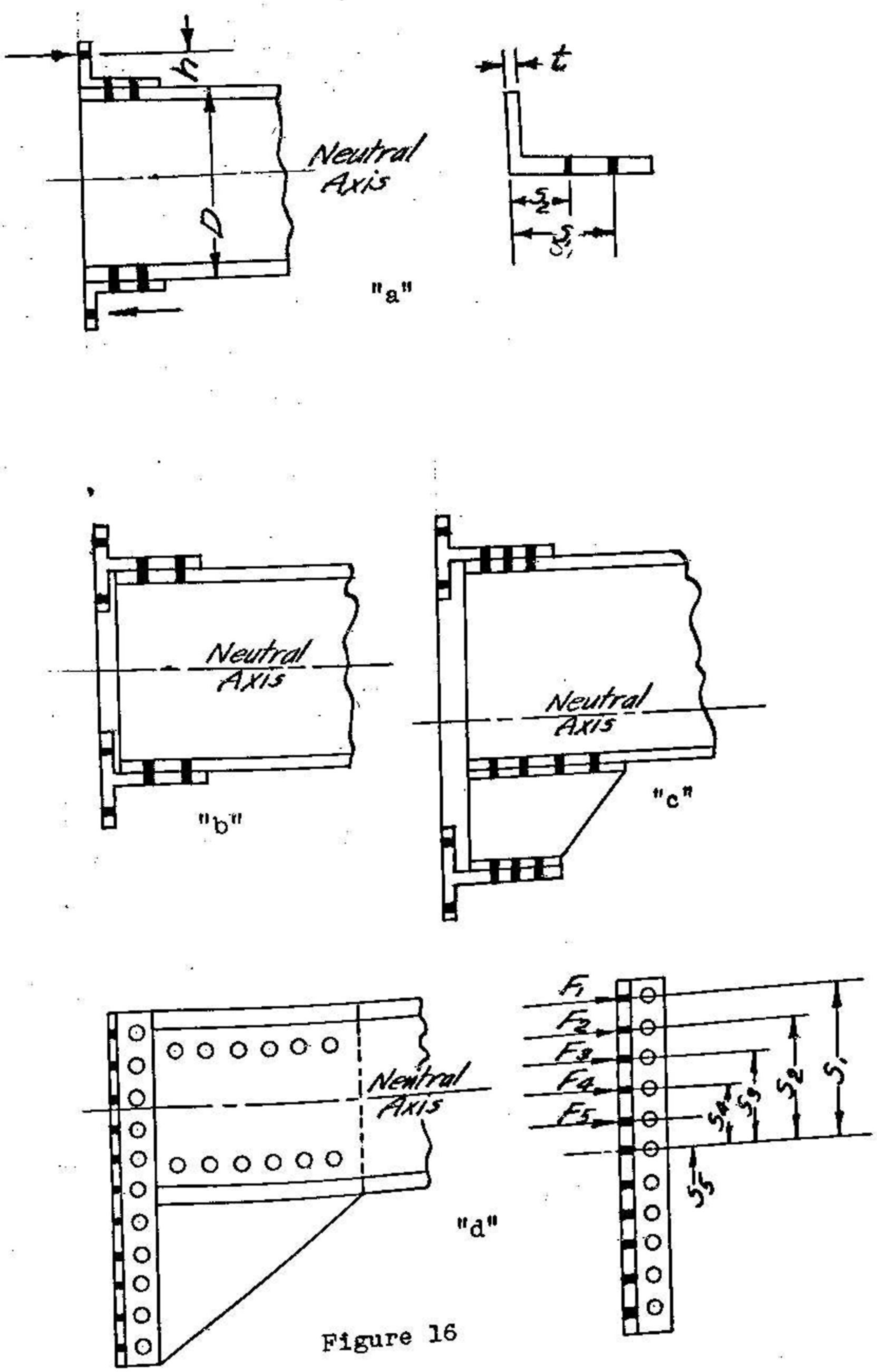

h
t
Neutral Axis
D
s_2
s_1
"a"
Neutral Axis
"b"
Neutral Axis
"c"
Neutral Axis
"d"
F_1
F_2
F_3
F_4
F_5
s_1
s_2
s_3
s_4
s_5

Figure 16

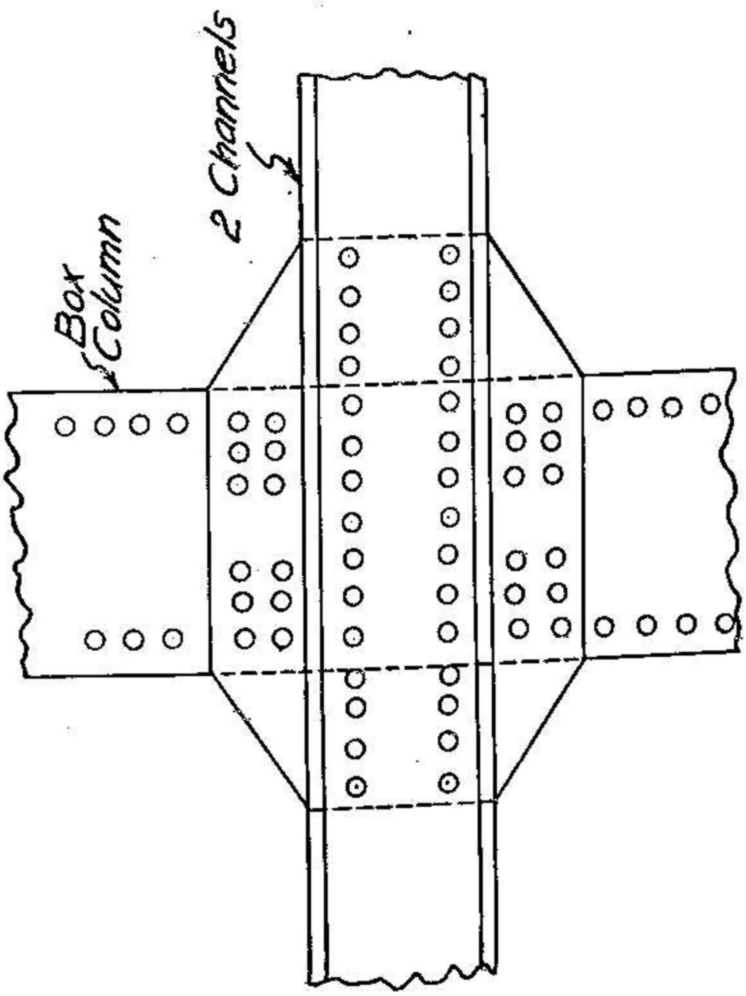


Figure 1 b (Continued)

Figure 16

confronting him. Figure 16 shows several types of joints in common use.

For type "a" (Figure 16) two angles are riveted to the beam and column as shown. The moment is resisted as indicated by arrows. The stress in the top rivets in tension is:

$$F = M(D - 2h)$$

M = wind moment

D and h are as shown in figure 16 "a"

It is noticed that there is no moment from the dead and live load on the joint. The number and diameter of the required rivets for the leg of the angle adjacent to the column is:

$$Nd^2 = 4F/(3.1416)(16000)$$

Usually the diameter of the rivets is assumed and thus the number can be found. The thickness of the angle is:

$$t = (3.1416)(d^2)h/b$$

b = width of angle

Fleming suggests that the point of contraflexure of the angle leg may fall at a point half way between the root of the angle and the center of the rivet.[1] In such a case the above becomes:

$$t = (3.1416)d^2h/2b$$

The rivets must be checked for shear due to the live and dead load. For this purpose it is better to consider

1 Robins Fleming, Wind Stresses in Buildings, 131.

only the bottom angle since it is undesirable to have rivets in tension for live and dead loads which would be the case if the top angle were to take a portion of the load. For bearing on the rivets in the bottom angle adjacent to the column Ndt(16000) must be equal to or greater than half the total dead and live load on the girder and for shear the rivets in the bottom angle $N(d^2/4)1000$ must be equal to or greater than half the total live and dead load. It has been assumed in both cases that the rivets will be hand driven. The number and diameter of rivets in the leg of each angle adjacent to the girder with respect to shear is:

$$N = M/(3.1416)(D)(4500)d^2$$

These rivets are also subject to direct axial tension. The number and diameter with respect to direct axial tension is: (refer to Figure 16 "a")

$$F = M/(D - 2h)(s_1 - s_1/s_2)$$

$$Nd^2 = 4F/(16000)(3.1416)$$

$$M = \text{wind moment on joint}$$

To protect against bearing stresses t'Nd(32000) should be equal to or less than M/D. The thickness of the leg of the angle adjacent to the girder should be the same as that adjacent to the column.

It is not necessary to go through all these steps in actual design after one or two joints have been completed. Many times designing for one stress automatically takes care of another. This is particularly true of shearing and bearing.

Obviously, the controlling factor in joints of the angle type is the thickness of the legs. Sometimes a split "I" beam is used as in Figure 16 "b". The calculations are similar to those for the angle type. When either of the above types are inadequate for the wind moment, extensions may be used as shown in Figure 16 "c" Again the calculations are similar.

When the girder is composed of two rolled or built up channels a gusset joint may be used similar to Figure 16 "d". The size and number of the rivets through the webs of the channels and the gusset plate are given by:

$$Nd = M/p(3350))^{\frac{1}{2}} \quad \text{for hand driven rivets}$$

$$Nd = M/p(4500))^{\frac{1}{2}} \quad \text{for shop rivets}$$

The above is given with respect to shear. For bearing the following holds:

$$Nd = M/tp(27000) \text{ for hand driven rivets}$$

$$Nd = M/tp(32000) \text{ for shop rivets}$$

In both cases t is the thickness of the gusset plate provided it is less than the sum of the thickness of the webs of the channel and p is the depth of the rivet lines.

For the rivets connecting the angles to the column:

let n number of rivets

let s_1, s_2, s_3, s_4 -------------- s_n be the respective distances from rivets to the neutral axis

let F_1, F_2, F_3, F_4 -------------- F_n be the permissible direct axial stress for one rivet.

The outer most rivet on the tension side of the neutral is stresses to the allowed limit. The other rivets are stressed proportional to their distance from the neutral axis. Obviously the rivets on the compression side are not stressed. Considering the rivets on the tension side:

$$M/2 \quad F_1 s_1 + F_1(s_2/s_1)s_2 + F_1(s_3/s_1)s_3 + F_1(s_4/s_1)s_4$$

$$------------ + F_1(s_n/s_1)s_n$$

M = wind moment

For the rivets connecting the angles to the gusset plate we have the same equation except that F_1 is the permissible shearing or bearing stress; the latter depending upon the thickness of the gusset plate if it is less than the sum of the web thickness of the two channels. The thickness of the leg of the angle adjacent to the column must be determined:

Let p' = pitch of rivets

Let y = distance from center of rivet to extreme edge of angle

$$Mc = SI$$

$$F_1h(t/4) = 24000(t^3)(y + p'/2)12$$

$$t = (F_1h/8000(y - p'/2))^{\frac{1}{2}}$$

F_1 = permissible axial tension of rivet

h = distance from center of rivet to edge of leg of angle adjacent to column.

The above calcualtions need not be repeated for the leg of the angle adjacent to the gusset plate; since, the moment at the root of the angle is the same.

For continuous girders of channels or built up channels gusset joints may be used similar to Figure 16 "e". To determine the number of rivets connecting the channels to the gusset plate, the amount of moment that the channels will stand must be determined. This value is deducted from the sum of the two maximum wind moments. For the dead and live loads the channels act as continuous beams; hence, there will be a negative moment which must be cared for. Usually the channel sections are adequate to properly care for this moment. However, if it is found that the section is greater than is required, advantage of the fact should be used in designing for the wind moment. The columns are so designed that they will resist the wind moments thrown upon them. It may be necessary in some cases to design for each wind moment separately, due to architectural considerations, withoug adding the two together. In such a case the number of rivets will not be the same on each side of the column but will be proportional to the moment on each span of the girder. The number and diameter of rivets required for shear is:

$Nd^2 = M/(3.1416)(p)(3350)$ for hand driven rivets

M = moment

p = effective depth of rivet rows

For bearing:

$Nd = M/pt(27000)$

t = thickness of gusset plate

In practically all joints it will be found that the shear and bearing upon the rivets due to the live and dead loads are small in comparison to the wind stresses when the moment is large enough to warrant the use of a gusset joint.

Obviously, for a detailer to design each joint is impractical so far as speed is concerned. All the possible joints that can be made from tandard sections are designed and put in tabular form. The joints are then selected from the depth of the girder and the section modulus. For joints, the section modulus is a quantity which when multiplied by the value of one rivet in shear, direct axial tension or bearing, whichever might be the predominating factor for a particular type of joint, gives the moment in foot pounds or inch pounds.

Column splices should be designed so that the moment of inertia is commensurate with that of the column section. If this is not practical, it should be designed to resist the moment expected to fall upon it. As a rule columns are spliced a few feet above the girders because of the resulting advantages in construction. This distance depends, of course, upon the joint which must be cleared. The number of rivets required is determined in a manner similar to that for the flange rivets of the girder previously given. The moment of inertia of the splice plates is found from the formula given in mechanics $I = I_0 + Ad^2$. The use of this formula was illustrated in the girder design. The ends of columns for splicing are usually milled; and if they are of different section, a plate is laid between them to insure full bearing for each column. No general equation for splicing is given ecause of the number of built up column sections that may be made.

CHAPTER IV

EARTHQUAKE STRESSES

Considerable work has been done towards a solution of the problem of earthquake stresses in buildings, but much still remains before all conditions can be fully determined. Of recent years the study of seismograph records have done much to further the work. At least it is possible at the present time to make estimations that approach the truth with enough accuracy to warrant using them for design purposes.

The movement of the earth during an earthquake is not confined to any one plane. This has been ably demonstrated by Professor Sekiya who took the records of seismographs recording movements in two planes and solved for the resulant or absolute movement. He then took stiff copper wire and bent it to correspond with the absolute motion for a given time, distance and direction.[1] The appearance of the wire was similar to a quantity of solder wire that had been bunched together in the hand.

The waves as registered on a seismograph are not sine waves although for purposes of estimating the acceleration of the earth in any one plane they are assumed to be such. The amplitude is, of course, not constant; hence when

1 Robins Fleming, Wind Stresses in Buildings, 160

estimating the acceleration, the maximum amplitude and shortest period should be used since:

$$^{1}A \quad \frac{4\pi^2 \text{amp}}{\text{period}^2} \qquad A = \text{acceleration}$$

It is true that a wave of the largest amplitude may not be the most destructive as a glance at the above equation will show. According to Dr. Percy Byerly of the University of California there are three groups of waves which may be recorded:[2]

> As recorded on the seismograph, three groups of waves are to be distinguished. The first to arrive are composed largely of the compressional, rarefactional or longitudinal type; they have traveled through the earth by a curved path with a dip deeper than a chord. Their velocity is about 5.6 km. per second. The waves of the second group are of the shear or transverse type; and have traveled a similar path to the waves of the first group, but with a lesser speed, a cut 3.2 km. per second. The first two groups are termed the first and second preliminaries.
>
> The third group of waves appears on the seismogram after the second preliminaries.and usually carry the maximum energy at distant stations. They have traveled along the earth's surface and are therefore referred to as surface waves. Within these surface waves are at least two separate types, and possibly more. The motion in the first type seems to be traverse to the path and largely in the horizontal plane. The velocity is about 4.4 km. per second. The second kind of of surface waves has a vertical component, and they have a velocity of about

1 Earthquake - Resistant Construction I - Data of Design, Henry D. Dewell, Eng. News-Record, April 26, 1928, vol. 100, 653.

2 Ibid., 650.

> 3.9 km. per second. On some records the so-called "maximum portion" arrives as something definitely new with a velocity around 3.3 km. per second. Often this portion builds up gradually and appears as a part of the previous type. The maximum portion has a larger vertical component, and the waves resemble, or are, surface waves.
>
> Following the maximum portion the record dies out in a long series of oscillations, which are often referred to as the coda.
>
> In nearly all earthquakes the second preliminaries and the maximum portion or principal portion are usually inseparable if not identical ...
>
> Many observers have noticed waves in the alluvium at the time of a severe earthquake. These are obviously a secondary effect. Their velocity is quite small relative to those of the waves above described. Observers relate seeing them approach and bracing themselves for the movement. These waves may be very effective in damaging buildings on alluvium. These waves are not recorded on seismographs, since (1) whenever possible seismograph stations are located on rock, (2) such extreme motion would disable ordinary seismographs built for delicate work.

The above is interesting in that it gives us another way in which an earthquake may affect a building. There is a motion at right angles to the direction of propagation known as amplitude. Both are non-coplanner. There is also the effect of the wave as a body when it strikes the building in the path of propagation. This is particularly true of the first of the groups mentioned above. Dr. Bailey Willis, past-president of the Seismological Society of America gathered evidence in Santa Barbara which seemed to him to cause the collapse of certain types of buildings at the first shock.[1]

The destruction caused by an earthquake is due primarily to the movement in a horizontal plane. Hence, the primary data for design is to approximate the maximum acceleration as given at the first of the chapter. If seismograph records are not available, a rough estimate can be made from overturned objects.[2] Newton's first law may be used:

$$F/A = W/g$$

F may be determined from the moment causing overturning, W may be closely approximate from the dimensions and density of the object and A may then be determined. Obviously, the results of the above should be compared with an object that did not overturn so that a maximum and minimum value of the acceleration may be had. The true value would be somewhere between these two.

If the assumption is made that a structure is perfectly rigid, then the shear and moment at any horizontal plane may be determined from Newton's equation as given above. The force that the steel frame work must overcome acts at the center of gravity of the portion of the building above

1 Henry D. Dewell, Earthquake-Resistant Construction I - Data of Design, Eng. News-Record Vol. 100, 651.

2 Ibid, 653.

the plane. For purposes of calculation it is not necessary to determine the exact center of gravity. Suppose for an inside bent similar to figure 2 we have:

Wt. of outside walls = 20(12)(100) = 24000 #

" " partitions = 12(20)(25) = 6000 #

" " floors:

live = 20(20)(100) = 40000 #

dead = 20(20)(25) = 6000 #

Assume total weight of roof to equal

weight of floors = 46000 #

Approximate weight of roof girders 3 @ 600 = 1800 #

" " " columns A and D 1500 = 3000 #

" " " " B and C 1200 = 2400 #

The total weight above plane x-y is 221000

$F = (W/g)A$ letting $A = .10g$

$F = (221000/g).10g = 22100$ #

The force of 22100 is the sum of the forces acting at the centers of gravity of the walls, floors, partitions and members, but we have obtained the total force as shear on the plane x-y. If we repeat the procedure for a plane passing through each story, we will find that each shear is approximately the sum of those of the stories above. The relation will be exact if the girder and column sections increase proportionally from top to bottom. Unlike the action of the wind against the side of a building, the total shear due to an earthquake for a given plane through a story is distributed among the columns. In some of the previous approximate methods this condition was assumed but in this case it is strictly true. The distribution of the shear is proportional to the weight of the part of the building which is acting against the column. Just how the weight divides itself is a matter for conjecture. A logical assumption would be according to the weight that the column supports. In the design of the Mitsue Bank Building it was assumed that the horizontal shears on the columns were equal in the same plane.[1] If we assume that the points of contraflexure for columns are at the midpoints, an approximate solution is obvious. Method II-A is probably the most adaptable because of the similarity of the assumption that the columns take horizontal wind shear proportionally to the bay widths which is also approximately true of the distribution of weight among the columns. It is to be noticed that the points of contraflexure for girders will not be at the midpoints of the span, due to the fact that the weight of the outside walls upsets slightly the original

[1] John W. Pickworth and Walter H. Weiskopff, Tokyo Bank Building Designed to Resist Earthquakes, Eng. News-Record vol. 98, 1012.

assumption made for the method when used in connection with wind stresses.

The value of .10g for the horizontal acceleration as used in the above is usually recommended. In made and marsh land accelerations as high as 10 to 12 feet per second were estimated in the San Francisco earthquake.[1] This value is extremely high and as in the case of extreme winds can not be economically taken care of. The Tokyo building regulations use a value for the acceleration of .10g, but the combined dead and live load, and earthquake load must not exceed a working stress of 16000 per square inch or other usual working stresses as stated in the Japanese building code.[2] It was stated in a previous chapter that the allowable unit stresses for wind may be increased due to the fact wind will not always have the velocity assumed in the design. Likewise the unit stresses for earthquakes should also be increased and should be greater than those for wind because of the possibility of occurrence.

Two general principles of design are used. The first endeavors to make as independant as possible the motion of the ground and the building. Small buildings in Japan have actually been constructed which have rollers between

1 Henry D. Dewell, Earthquake-Resistant Construction I - Data on design, Eng. News-Record, April 26, 1928, vol. 100, 652.

2 Ibid.

the foundation and superstructure. It has been noted that the most destructive waves are on the surface of the ground and that a few feet down the destructiveness is decreased; hence, it has been proposed that certain buildings continue into the ground and have a space between them and the surrounding earth connected only by lateral members designed to permit movement. It has also been suggested that slip joints be used at the points of contraflexure for the basement columns. The second system consists of rigidly connecting the foundation and superstructure together.[1]

The most notable example of earthquake building construction at the present time is the Mitsui Bank building at Tokyo, Japan, The consulting engineers who are responsible for sedign are John W. Pickworth and Walter H. Weiskopf. The building is designed for a hovizontal eccelleration of .10g.

The foundation consists of a solid reinforced concrete mat which distributes the vertical load evenly over the supporting ground. Tests show that 4000 pounds per square foot bearing upon the soil is permissible, but 2000 pounds is all that has been used.

In accordance with the Japanese Building code all column splices are made at the points of contraflexure.

1 Henry D. Dewell, Earthquake-Resistant Construction I-Data on Design, Eng. News-Record, April 26, 1928, Vol. 100, 654.

From the architectural requirements the first floor required rather long columns which resulted in correspondingly large bending moments. The detail used was four angles at each corner heavily latticed. This design is particularly adaptable in that the moment of inertia and radius of gyration are the same in two directions. That is, the general outline of the columns are square. In splicing columns it was required by the building code that there was no bearing between sections. Since the bending moments in all members were extremely large all girders were latticed.

In the previous solution for earthquake stresses it was assumed that the building was perfectly rigid which is not true. Omori observed that tall chimneys did not rupture at their base as would be expected but at a distance about two thirds of the height or one third the height from the base. It is reasonable to believe that such is the case for structures and some buildings in earthquake areas have additional bracing at the third points. Professor Milne says that the acceleration that will fracture a column firmly fixed at its foundation is:[2]

2 Robins Fleming, Wind Stresses in Tall Buildings, 182.

$$a = (g/6)(FAB)/fW$$

a = acceleration

F = the force of cohesion, or force per unit surface, which when gradually applied produces fracture.

A = area of base fractured

B = thickness of the column

f = height of center of gravity of column above the fractured base.

W = the weight of the portion broken off.

It is readily seen that to assume perfect rigidity and use a method of calculation similar to that given would result in extremely heavy design for buildings approaching a thousand feet in height. Obviously such a design would be impossible economically. Present engineering knowledge for such cases is hardly more than a guess. It is not unreasonable to say that if an earthquake such as struck San Francisco in 1906 were to occur in New York, many if not all the tall buildings would be so damaged that they would have to be torn down and rebuilt if they did not collapse.

CHAPTER V

CONCLUSIONS

Several methods for calculating wind stresses have been presented, but no one method has been recommended. It is not feasible to use the slope deflection method because of the time and work required. At best this method is primarily for review. For the approximate slope deflection method Ross claims that a change of member because of inadequacy to care for a moment tends to increase the same conditions. In the design of the American Insurance Building Ross says:[1]

> In order to determine the possibility of using the approximate slope deflection method in designing, the nineteenth floor of the American Insurance Building was selected and the stresses in the girders were determined for the bent as constructed, by the approximate slope deflection method. The first solution gave a decreased moment for the long center span and for the outside while the moment for the short span was considerably greater than that for which it was designed. The sections of the girders were reproportioned for these stresses and a second calculation of stresses made. The variation was in the same direction and more marked. A third solution showed the same tendency. In these solutions the sections of the columns were not changed, but it is evident that if the sections of the girders were changed as indicated by the stresses, the column moments would be greatly altered also.

1 Albert Ward Ross and Clyde T. Morris, The Design of Tall Building Frames to Resist Wind, Aver. Soc. of Civil Eng. 1410.

If the slope deflection method is accepted as a standard, Ross's method is probably the most exact for design purposes. However, due to the assumptions made for the slope deflection method it is not impertinent to question the accuracy of the method. Upon the assumptions made the method is exact, but the question arises as to the validity of the assumptions. Moliter evaluated the effect of the assumptions as follows:[1]

> 1. The connections between columns and beams are perfectly rigid. This may influence the resulting stresses by 30 to 50 per cent as indicated by strain gage measurements on riveted connections.
> 2. The change in lengths of members due to direct stress is negligible. This may involve errors of about 1 per cent.
> 3. The lengths of the members are the distances between intersections of their neutral axes. This may effect the stresses.
> 4. The deflection of a member due to internal shear is negligible. Deflections in beams are usually about 10 per cent greater when the web shear is considered.
> 5. The wind load is resisted entirely by the steel frame. If this were true, tall buildings would sway in the wind three or four times as much as they actually do. In other words the architectural clothing probably increases the rigidity of the frame from 300 to 400 per cent...

[1] Robins Fleming, Wind Stresses in Buildings, 102.

If the above approaches the truth, the value of requiring close agreement between slope deflection and approximate methods is questionable. Due to the fact that it is necessary to make similar and no less assumptions than are made for the slope deflection method it is valid to use this so-called exact method as a basis for comparison although it in itself may be in error.

Wilson and Maney compared the results of the slope deflection method, the approximate slope deflection and Fleming's methods I, II and III which were found to compare in about the order given.[1] Ross compared the results of his method with those obtained from the slope deflection method for the first eight stories of the American Insurance Building. In order to use the slope deflection method for a portion of the building without solving for the whole it was assumed that $\theta_8 = \theta_7$ and $R_7 = R_6$.[2] The results of Fleming's method No. I were compared and were found to be considerably in error. In this building it was necessary to omit some girders at the second and third floors in order to provide for a mezzanine floor. It may be that the errors in Fleming's

1 W.M. Wilson and G.A. Maney, Wind Stresses in the Steel Frames of Office Buildings, Bull. No. 80, Eng. Exp. Sta. Univ. of Ill., Tables 23 and 24, 79-80.

2 Albert Ward Ross and Clyde T. Morris, The Design of Tall Buildings to Resist Wind, Amer. Soc. of Civil Eng. 1402.

method were due to this irregularity. It is also true that the above assumption to make possible the solution of the first eight stories will affect the results of the slope deflection method.

It is seen that in methods II and III that the moments at the ends of a girder are not equal but are consistently greater at one end than at the other. Since the wind is as likely to blow from one direction as another, the girder connections and the girder itself is designed for the largest moment. The result is usually an over design for the girders while the moments in the columns vary in both directions from those found by the slope deflection method. Method III is applicable to bents of not more than three bays because the point of contraflexure will fall outside the span length of the girders.[1]

In the past all four of Fleming's methods have been used and it is likely that they will continue to be used in the future. Methods I and II-A have been somewhat preferred; method I for the many checks it affords the designer and method II-A for the speed with which it may

1 G.A. Hool and W.S. Kinne, Stresses in Framed Structures, 464.

be worked. The points of contraflexure for methods I and II-A are at the mid-points of girders and columns; hence, the moments to be designed for in the column and girder connections are not so great.

For purposes of design no one method may be recommended since all have their advantages. From the standpoint of accuracy the approximate methods to be used for design are Ross's and Fleming's methods No's. I and II with the approximate slope deflection and slope deflection for checks of peculiar or irregular dimensioned bents or portions of bents. It is noticed that methods II and III have been left out. It may or may not be significant that of the number of methods given by Fleming in Engineering News, March 13, 1913 and those of "Six Monographs on Wind Stresses" from articles by Fleming published in book form, he only gives what are classed in this paper as methods I and II-A in his recent book "Wind Stresses in Buildings" published in 1930.

The choice of methods depends upon the judgment of the engineer. Sometimes it may be feasible to use more than one method on a building. The final choice of methods depends upon the height of the building and the least width of the base, the type of the building and last, but by no means least the time allowed the engineer to complete the design. As Professor Burr says, "So long as the stresses found by one legitimate method of analysis are provided for, the stability of the structure is assured."[1]

It is readily seen that to design an extremely tall building to resist earthquake shock on the assumption that it is a rigid body is economically impossible. It is entirely possible that calculations upon this assumption are greater than is actually the case were the truth known. It is probable that adequate bracing at the third point is sufficient, but the stresses to be designed for are unknown. At the present time the surest measure of safety is to limit the height of the building to some conservative distance and assume that the structure is a rigid body. If during an earthquake a fissure occurs at the base of the building, the result is problematical. No amount of designing could resist the tremendous stresses resulting from such an occurrence. If the building rests upon a reinforced concrete mat as in the case of the Mitsui Bank building, the results may not be as bad as if each

1 Robins Fleming, Wind Stresses in Buildings, 117.

column rested upon a separate footing but even the least that might happen would necessitate expensive repairs if the fissure were very wide. However, any saving in lives resulting from earthquake "proof" construction is money and time well spent.

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