DESIGN OF HIGH-RISE BUILDINGS

by

Fazlur R. Khan

Presented at

A SYMPOSIUM ON STEEL

Sponsors

ASCE
AISC Chicago Fabricators
University of Illinois
at Chicago Circle

Fall 1965
Chicago, Illinois
INTRODUCTION

The continuing economic prosperity and population increase in the urban areas point toward a future with increased activity in high-rise construction of residential and office buildings. However, construction of high-rise buildings can be economically attractive only if the structural engineers can have comprehensive understanding of the structural behaviors of various systems on one hand and the practical sense of the construction problems on the other.

Steel building construction is only about 100 years old and an enormous amount of progress has been made since the beginning. The use of cast-iron and wrought-iron has long been replaced by high strength steel. Fabrication and erection techniques have been considerably improved and mechanized. Yet while the strengths of steel have gone up, the Modulus of Elasticity remains the same as it was just when iron was first found by man. It is left to the structural engineer to compromise the modern high strengths of steel with its "pre-historic" modulus elasticity on one hand and the not-so-advanced fabrication and erection techniques with the increased labor cost on the other.

The purpose of this paper is to briefly discuss the various aspects of a multi-story structure with particular reference to the latest AISC Code and point out the interpretations of the theory and practice involved in each case that may lead to more efficient multi-story structures.
FACTORS CONTROLLING DESIGN OF HIGH-RISE BUILDINGS

It has been a frequent question as to what height or number of stories of a building makes a building high-rise. Arbitrary definitions of a high-rise building have in some cases included even a two-story building. From the structural engineers point of view a high rise building is one whose structural analysis and design is affected by one or more of the following factors:

a. Height-to-width ratio

b. Forces and moments due to lateral loads

c. Sway under lateral loads

2a. Height-to-width ratio:

The most significant factor that affects the design of multi-story frames is the height-to-width ratio. Assuming that bay sizes remain constant the increase of number of stories will reach a point above which any further increase of story will require additional amount of steel in columns and beams beyond what would otherwise be required only to support the gravity loads. Figure 1 illustrates this point where arbitrarily the eighth story is shown as the point of departure from the gravity load requirements. From the structural point of view this building up to 8 stories could hardly be called a high-rise structure.

Figure 2 shows the same structure as in Figure 1 except that two more bays have been added. It is quite reasonable to say that the critical
number of stories has changed from eight to say 12. In actual practice when similar cases are compared, it becomes evident that the height-to-width ratio is probably the most significant factor that affects the choice of the type of structural system of a high-rise building.

From experience, it has been observed that the practical limit of height-to-width ratio for conventional steel structures is about 7.

2b. Forces and Moments Due to Lateral Loads

All building frames are subjected to some form of lateral loads. Normally the lateral load is caused by wind. In certain areas the earthquake forces may be the controlling design factor. The AISC Code in Section 1.5.6 specifically allows 33% increase in all relevent allowable stresses for combination of seismic or wind and gravity loads. For proper combinations of bay size, number of bays and number of stories, it is possible to design the entire frame only for gravity loads and automatically satisfy the wind load condition. In actual buildings such a controlled design is hardly possible and therefore some "premium" has to be paid for the height of the building.

In checking for the combined stresses in any member a frequent misinterpretation is made. The general form of the equation used for checking combined axial and bending stresses is -

\[
\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \quad (1)
\]
For checking for wind load forces and moments Equation (1) is sometimes written in the form

\[
\frac{fa}{F_a} + \frac{fawx}{F_{awx}} + \frac{fbx}{F_{bx}} + \frac{fby}{F_{by}} + \frac{fbwx}{F_{bwx}} \leq 1.33 \quad (2)
\]

For wind acting in the direction of Y-axis similar equation is used. Although for very preliminary check Equ. 2 may be used, for final design one must remember that the code allows an increase of 33% in the allowable stresses only but does not allow a flat increase of the right hand side of the equation to 1.33. The reason for the discrepancy arises from the fact that the new AISC Code has the controlling equation written in the form

\[
\frac{fa}{F_a} + \frac{fb \cdot \ell}{(1 - \frac{fa}{F_{fa}})F_b} \leq 1.0 \quad (3)
\]

Therefore when checking for combination of gravity and wind stresses the proper form of the equation should be -

\[
\frac{.75 \cdot fa}{F_a} + \frac{fb \cdot \ell}{(1 - .75 \cdot \frac{fa}{F_{fa}})1.33 \cdot F_b} \leq 1.0 \quad (4)
\]

2c. Sway Under Lateral Loads

The most critical factor affecting the efficiency and the economy of high-rise steel frames is the lateral sway. Although opinions vary as to what should be the acceptable limitation for sway there is a general agreement that for normal frames, the maximum sway must be limited to some reasonable value. There are mainly two reasons why lateral movement needs to be controlled -
(i) Large movements can cause partitions and other fixtures to be damaged.

(ii) Large movements, if frequently perceptible, can cause discomfort to the occupants.

The problem of partition cracking etc. can be eliminated by proper details, although in some cases such details may be too expensive to be justified. On the other hand, the problem of human discomfort cannot be easily corrected and therefore in taller buildings must be resolved before finalizing the structure.

The allowable sway of a building is commonly expressed as the ratio of maximum computed lateral movement at the top of the building to the total height of the building. This ratio in existing tall buildings ranges from 1/200 to 1/800. A number of tall steel buildings built within the last twenty years in New York and Chicago fall between the 1/550 to 1/650 range. In view of the fact that these buildings have total heights between 500 and 900 feet and have not been known for any particular trouble from excessive sway, the writer is of the opinion that for preliminary design of high-rise steel buildings the sway ratio may be taken as 1/600.

It should be pointed out that in using a sway ratio such as 1/600, the engineer should recognize the two components of the total deflection, viz, the horizontal wracking at each story due to column bending and the vertical wracking due to anial deformation of the columns commonly called as "column shortening". The proportions of the horizontal
wracking to vertical wracking at each floor as illustrated in Fig. 3 will
determine whether one of the following factors will be more critical in
checking partition details.

(i) horizontal control joints
(ii) vertical control joints
(iii) control joints in both direction.

With the increasing number of slender and tall buildings being built
recently, engineers are more and more concerned about human reaction to
large lateral movements of these buildings under storms. To date, no
known study has been made to establish the various ranges of perception
levels for lateral movements of buildings. Some basic research on this
subject is greatly needed in order to establish a rational design criterion
for tall buildings. The writer is currently conducting a series of experi-
ments on human perception of linear movements and is also measuring
actual movements in one building. It is hoped that the results of these tests
will be published at a later date.

ELASTIC DESIGN VS. PLASTIC DESIGN

Plastic design of steel structures is an established method of achieving an
economic and safe structure. Considerable amount of research has been
done on rigid frames to determine the additional reserve strength of such
frames before collapse. In low-rise structures, where lateral movement is
seldom a concern, the limit design enables a considerable amount of
saving of steel. Take for instance, a part of the multi-story frames as shown in Fig. 4. For a normal elastic design the members of the frame would be sized as shown in 4a. For limit design the moment diagram can be as shown in Fig. 4B. and the sizes of members, using the plastic section modulus tables of the new AISC may be as shown on the same Figure. It is interesting to note that as much as 14 percent steel could be saved by the limit design approach in this particular case.

Now let us consider that the column and beam sizes as obtained by the simple ultimate design are used in the 20 story building as shown in Fig. 5a. and is subjected to the same lateral shear and vertical forces. It has already been shown that from the stress point of view the members are adequate. However, a quick check of lateral deflection shows that the story sway (wracking only) is \(1/210\) the story height. The equation commonly used by the writer for checking wracking component of sway for preliminary design is derived from certain simplifications of Maney-Goldberg method and is expressed as:

\[
E \psi = \frac{M_n}{\Sigma K_{c,n}} + \frac{M_n}{\Sigma K_{g,n}} \quad \cdots \cdots \cdots (3)
\]

where:
- \(\psi\) = sway ratio such as 1/600
- \(E\) = modulus of elasticity in K/in
- \(M_n\) = \(V_n \times h_n\) = total wracking moment in Ft. Kips
- \(\Sigma K_{c,n}\) = Sum of column stiffness \(I_c/h\) at \(n^{th}\) story
- \(\Sigma K_{g,n}\) = Sum of girder stiffness \(I_g/l\) at \(n^{th}\) level.
The terms are further illustrated in Fig. 6. For the example problem the left hand side of the Equation (3) will be \(30 \times 10^3/600 = 50\) and the right hand side of the equation will be 143.5. It is evident that the sway will be more than 1/600. Study of Equation (3) will readily indicate that the major portion of the sway (about 90% in most cases) is due to the girder flexibility. Therefore, the most efficient way of controlling sway is to increase the stiffness of the girder. To do so in the example problem, the girder stiffness should be increased to 46 in.\(^3\) and the resulting section modulus increases to 1600 in.\(^3\). The resulting stresses in the girder will reduce to 38% of the portal frame design purely for strength.

From the simple example, two significant conclusions should be drawn.

(i) The most efficient ways of controlling sway in a rigid frame is to increase the girder stiffness for normal buildings.

(ii) Where sway is a controlling factor in the design high strength steel should be limited only to columns and only if it is evident that the contribution to total sway due to column moments is small.

On the basis of the design of a number of high-rise steel frames the writer is of the opinion that except for certain criteria for seismic loads the normal sway limitation makes it inefficient to use high strength steel in the girders. Until the metallurgists find a way to increase the modulus of elasticity of steel, the high strength steel as well as ultimate design methods will be limited to the secondary floor beams of a conventional high rise building.
JOINTS AND CONNECTIONS

The proper design and detailing of the joints in a high-rise frame is necessary for two reasons -

(a) The entire design assumption of a frame may be jeopardized by incompatible joint details.

(b) Joints detailed without regard to the latest shop fabrication and field erection practices may make an otherwise light structure into an extremely expensive one.

In designing joints, the basic assumptions of the frame analysis must be kept in mind. If the analysis is based on full-fixed connections, it is imperative that the joints are designed such that the elastic properties of the joint elements are compatible with the members connecting at the joint. If the analysis is made on the basis of rigid connections and the joints are detailed for "semi-rigid" behavior, a variety of unforeseen effects may develop in the actual structures including increased sway and possible problems of column or frame stability.

One common form of semi-rigid connection is made by connecting the flanges through smaller size plates as shown in Fig. 7. In designing such a semi-rigid connection, the engineer should make it certain that the following two items are not overlooked -

(a) The possibility of brittle fracture as a result of biaxial stress condition in the plane of the connecting plate.
(b) The possibility of drastic reduction of the effective stiffness of the beam.

Experience has shown that the proportions of the connecting plate in Fig. 7 should be such that the unrestrained length of the plate should be greater than the width of the plate. In other words, the ratio L/B should be greater than 1.0. Tests have indicated that for proportion less than 1.0 there may be a possibility of brittle fracture without undergoing plastic deformation.

It is obvious that the stiffness of a beam is contributed more by its section properties near the ends than near the mid span. This can be readily illustrated by the Column-Analogy method of determining stiffness. Figure 8 the analogous columns for three different types of beam profile. From basic principle of column-analogy and its application on the three beams in Fig. 8, it is readily seen that the actual moment of inertia at the ends is the most effective in increasing stiffness and at the mid span it is the least effective. Two extreme cases would be hinges (zero I) at both ends which would make the effective stiffness zero for lateral movement and the hinge at center which would have no influence on the beam stiffness, if under wind load the point of contra flexure is assumed to be at mid-span.

In rigid multi-story frames where the actual maximum stress in the girders under combined gravity and wind load is less than half the allowable stress and where such structures are not subjected to earthquake loads,
the beam-to-column connection can be simplified as shown in Fig. 9. Although the effect of such connection is negligible on the sway characteristics of the frame, its effect on reducing the cost of erection can be significant.

The success of a steel frame in terms of over-all economy largely depends on the choice of the "optimum" joint. Optimum joint in this case is the type of joint that will satisfy the basic design requirements and yet will have the least effort in shop fabrication and field erection. The optimum joint for the same type of structure may be different under different combinations of location, shop practices and available field erection techniques and manpower. It is for this reason the engineer should consider a number of possible alternates and leave it to the contractor to choose the most economic one.

OPENINGS IN BEAMS AND GIRDERs

A building is not a structure only. It needs heating, ventilation, plumbing and electricity as the basic necessities of the modern life. Therefore, space for the ducts, the pipe, the cables and such other items must be provided. Obviously to keep the floor to floor height to a minimum, openings in beams and girders have to be provided wherever possible. Lack of proper coordination between the various engineering departments may lead to considerably more number of beam-openings than are actually needed. Although not having enough openings can cause later embarrassment the overdesign on the other hand is wasting money. In some projects
judicious planning in this matter has saved as much as $3.00 per ton of steel, which in a large project is not a small amount of money.

The structural implications of openings in beams should be briefly discussed. While small openings do not affect the basic elastic properties of the beams, large openings should be analyzed for possible combinations of loads. Recent tests indicate that even in larger openings certain simple details can be effectively used. Figure 10 shows the simpler reinforcing details around openings as compared to the conventional and more expensive details.

One point of caution should be mentioned. Frequently large openings are checked only for the normal uniform load moments and shears. This can sometimes lead to serious problems if unsymmetrical heavy loads are applied to the beam in its actual use. It is therefore advisable to use some reasonable criteria for unsymmetrical loads when designing the reinforcements for large openings. Fig. 11 shows the types of unsymmetrical loads that may be encountered in a multi-story frame.

As pointed out earlier, where girders are designed primarily to limit lateral sway of the frame the most effective portion of the beam is at the two ends. In such cases a logical conclusion is to use a deeper beam section near the two ends and a shallower section at the center as shown in Fig. 12. In some manner similar to this innovation is one of the traditional details used where smaller beam sections are added at the top and bottom flanges primarily to provide a rigid beam-to-column connection (Fig. 13).
For longer spans of beams the use of trusses is quite efficient in providing both stiffness as well as enough openings for pipes and ducts (Fig. 14). However, in that case care should be taken not to overlook the effect of the deformations of the diagonal members in reducing the stiffness of the effective beam.

**CHOICE OF THE OPTIMUM STRUCTURAL SYSTEM**

The successful structure is the result of the joint effort of the Architect, the Structural Engineer and the Mechanical Engineer. A structure using the least amount of steel may lead to a very expensive finished building because no consideration was given to any of the non-structural factors that affect the finished building. On the other hand, a structure that is designed only to fit the aesthetic 'visions' of the architect and the needs of all other trades is bound to contribute to the inferiority and excessive cost of the finished building. Somewhere between these two extremes lies the optimum structural system for any particular project and it is left up to the alertness and active participation of the engineer to develop the optimum system.

Experience has shown that most building structures may be classified into six broad categories -

(a) Simply-connected structures

(b) Semi-rigid structures

(c) Rigid-frame structures
(d) Structures with shear-trusses

(e) Structures with interacting frame and shear-trusses

(f) Rigid-box type structures
   (i) Closely-spaces exterior columns
   (ii) Closely-space diagonals as exterior walls
   (iii) Optimum column-diagonal-spandrel for exterior walls.

Simply connected will be most economical for building up to about six stories with masonry in-filled exterior walls.

Semi-rigid structural frame would normally prove efficient for up to about 15 story frames with some help from in-filled walls.

Rigid-frames are economically used up to about 40 stories although its economy gradually reduces after about 20 stories.

Vertical shear-truss type structures are efficient up to about 40 stories although in reality the remaining frame almost always contributes to the lateral stiffness (Fig. 15).

Structures built with shear-trusses to interact with rigid frames are efficient up to about 60 stories (Fig. 16).

For buildings over 60 stories, the optimum structural system is the rigid-box type structure where all the exterior wall elements of the structure are made to act together like the walls of a tube. This can be achieved by closely spaced exterior columns and relatively stiff spandrels (Fig. 17) or by closely spaced diagonal members on the exterior wall (Fig. 18) or by tying all exterior columns simply by adding the minimum
number of diagonals in the plane of the exterior walls (Fig. 19). Buildings using all these three systems have been built or are in the process of being built. It appears from information on some of these buildings that the last of the three systems results in the least material and least cost. Fig. 20 illustrates a comparative height study of the six types of high-rise structures discussed above.

CONCLUSION

Factors affecting the efficiency of high-rise structures have been discussed. A number of simple guide lines are included.

The optimum structure for a given project is the one that is least tortuous and evolves into the form most natural for the loads imposed on it.

* * *
FIG. 1

Steel wt. in lb/ft² of floor

No. of stories

Actual buildings

Total steel

Col. steel

50
40
30
20
10
20
40
60
80
100
V_{TOT} = 121 kN

FIG. 4
FIG. 4a

FIG 4b
<table>
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<tr>
<th>b</th>
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<th>n</th>
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</tr>
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**FIG. 6**

**FIG. 7**
V- UNIFORM LOAD

V- WIND LOAD

V- CONC. LOAD

FIG. 11
\[ I_{(\text{nominal})} \approx A_T \cdot r_c^2 + A_B \cdot r_B^2 \]

\[ I_{(\text{effective})} \approx 0.65 - 0.85 \cdot I_{(\text{nom})} \]