Plastic Design of a 14-Story Apartment Building

WILLIAM A. BENNETT

THE SUBJECT of this paper is the plastic design of the steel frame of the 14-story Binghamton Apartment Tower in Binghamton, N. Y. (Fig. 1). The design method used was that advanced by the authors of the Lehigh University lecture notes entitled *Plastic Design of Multi-Story Frames.*

The Lehigh notes provide analytical methods for both braced and unbraced frames. The unbraced frame method was used for this project because architectural considerations prevented the use of X-bracing or shear walls.

Sample calculations taken from the actual design sheets are included in an Appendix to illustrate the design methods and techniques described.

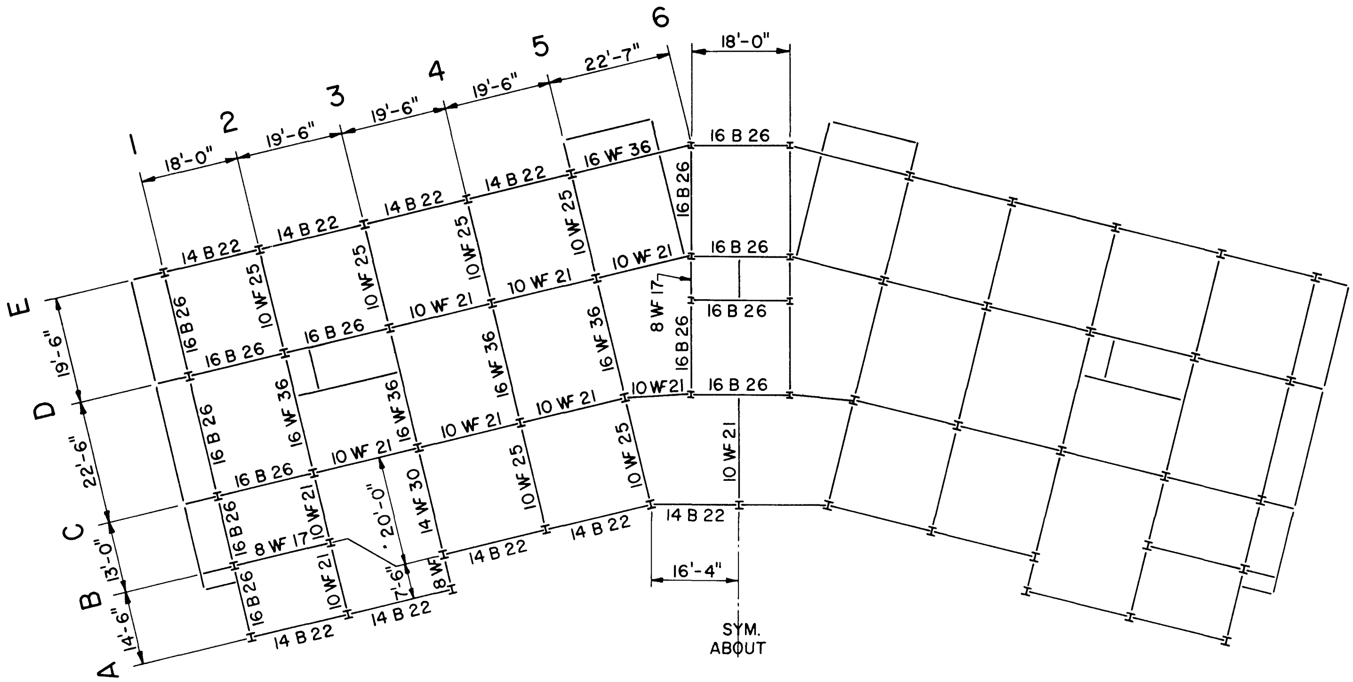
Figure 1

William A. Bennett is Structural Engineer, George E. Yurchison, Architect, Rochester, N. Y.

DESCRIPTION OF BUILDING

The Binghamton Apartment Tower is designed as a luxury apartment building with variable size apartments of one, two and three bedroom units. It will be fully air conditioned and provide balconies for the larger units. The building plan is V-shaped (Fig. 2) and provides a large open parking deck as well as basement parking for a total of 148 cars. An enclosed swimming pool will be provided adjacent to the parking deck. The building will be constructed on a prime site overlooking the Susquehanna River to the south, within the city limits. There are fourteen floors with two centrally located high speed elevators; one of which is large enough to be used as a service elevator. Two stairways are provided; one in each wing. There are ten apartment units per floor with each floor having its own storage and service facilities. The number of units chosen per floor represents the maximum which can be serviced by one employee, thereby creating a more efficient service operation. An important factor in the building design was a zoning requirement limiting the height of the building to 140 ft.

Because dead weight is costly, a lightweight concrete floor plank is to be used for the floors, supported on open web joists, with no topping in the living rooms, bedrooms and halls. Carpeting is to be placed directly on the plank in these areas. A plus factor in the use of floor planks in New York State exists if the plank can be erected immediately after final bolting of each tier, thereby eliminating the extra expense of providing temporary wood planking. Interior partitions are 4- and 6-in. masonry with dry wall, painted finish. Exterior walls are steel stud insulated panels with a brick veneer. This keeps the desired brick exterior while maintaining a relatively light wall which can be rapidly built. This type of construction is in keeping with office policy for work where speed of construction is important.



PLAN – 9TH FLOOR

Figure 2

ARCHITECTURAL CONSIDERATIONS

The main design requirement was to squeeze 14 stories in an overall height which would normally accommodate only 13 stories. The floor dimensions were limited by the site and apartment area requirements, so that little or no space was available in plan for vertical bracing. Also, the need for a flexible apartment layout meant that partitions would not line up with transverse column lines. For these reasons a moment resistant frame was analyzed keeping beam depths to an economical minimum while maintaining required frame stiffness. This resulted in a 9 ft-4 in. story height above the eighth floor and a 10 ft-0 in. story height from the eighth floor to the second floor. In order to maintain a uniform 8 ft-2 in. ceiling in the exterior bays, floor beams for the upper floors were limited to a 10-in. depth in the perimeter bays (Fig. 3). These beams were contained in the ceiling space and carry negligible gravity loads, so that their total moment capacity is available for wind and sway resistance. Interior beams are 14 in. and 16 in. in depth, giving a ceiling height of 7 ft-6 in.

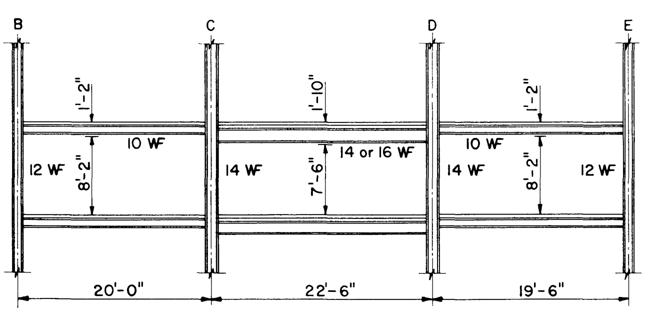


Figure 3

For the lower floors the transverse beams are kept at a uniform depth of 16 in. The overall tonnage including balcony framing, main entrance canopy, parking deck and miscellaneous steel is 1,100 tons of structural steel and bar joists. Of this total, structural steel amounts to 7.1 lbs per sq ft and bar joists 2.6 lbs per sq ft. The total cost of steel is estimated at $400,000.

STRUCTURAL ANALYSIS

Through the application of a semigraphical method, the total lateral deformation behavior of a story can be determined. The members selected from a preliminary design will then be adequate if the load-deformation behavior of the story is satisfactory. This method takes into account the additional moment due to the $P\Delta$ effect, as well as the wind moment. (In a given column the axial force, P, displaced by an amount Δ, produces an additional overturning moment commonly referred to as the $P\Delta$ moment. This moment has been found to cause significant reductions in frame strength and must be accounted for in the analysis.) The ultimate load deflection index used is $\Delta/h = 0.02$, where h equals the story height. An initial graphical solution suited to any

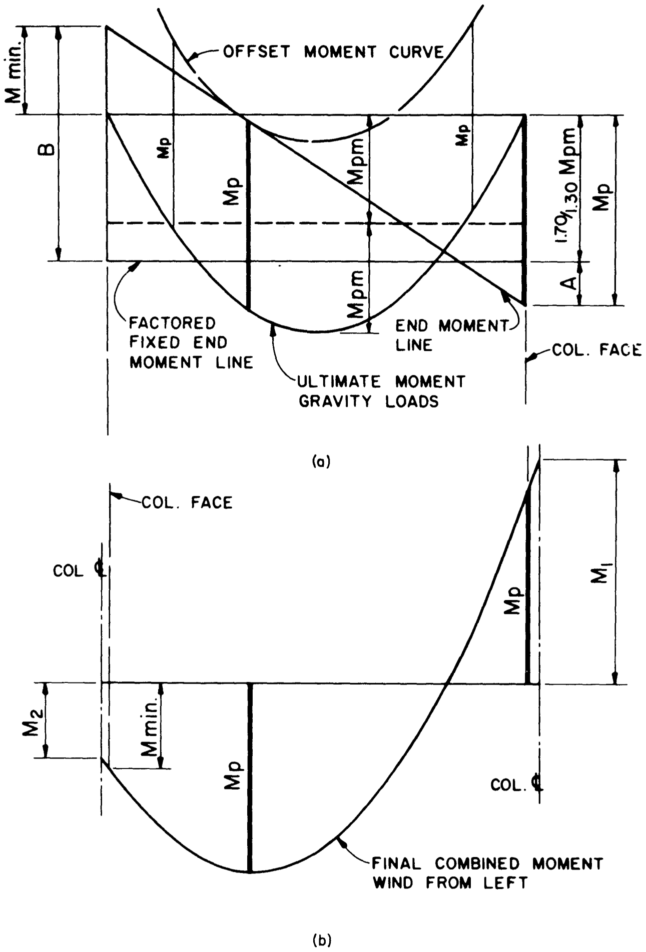


Figure 4

type of gravity loads and any load factors can be constructed to determine the maximum reserve capacity (over the amount required for gravity loads) for a beam (see Fig. 4). This capacity can be used to resist wind and $P\Delta$ moments at each level.

In Fig. 4a the minimum plastic moment, M_{pm}, is the plastic moment required for gravity loads only (L.F. = 1.70) and is used for the no-drift condition. Since for the condition of wind-plus-gravity load a load factor of 1.3 is appropriate, a beam adequate for the gravity-load-only condition would be adequate for a factored end moment of $1.70/1.30 \times M_{pm}$ when resisting wind-plus-gravity load. A beam with a greater M_p is chosen and the end moment line constructed so that at no point in the span is this value exceeded. This yields values of A and B (measured from the factored fixed end moment points) which would be reached at the beginning of mechanism action. When these values are reached, uninhibited lateral movement would occur. These values are used to determine the maximum value of the restraining functions M_{r1} and M_{r2} in a given subassemblage by dividing by the limiting column moment M_{pc}.

The slope deflection equations are used in defining the restraining function. In simple terms this function can be written $M_p = f\theta M_{pc}$, where the value for f is taken as KEI/L and nondimensionalized by dividing by M_{pc}. Before the first plastic hinge forms in the girder, the value for K is 6. When this hinge forms, the stiffness of the girder is reduced and the value of K becomes 3. After the last hinge forms, the restraint becomes zero and the load-deflection relationship of the subassemblage becomes parallel to the rigid plastic line.

In order to use this method, values for the available column moment M_{pc} must be obtained. These values, along with charts showing the moment-rotation curves of beam-columns and curves showing the lateral load vs. deflection relationships for various ratios of P/P_y and h/r_x of the column, are presented in the design aids booklet which accompanies the Lehigh University lecture notes. The basis for these charts can be found in the development of the column deflection curve concept, which can give the complete load-deflection history for a particular column. By overlaying the appropriate chart, the restraining functions can be plotted graphically and the lateral deflection of the subassemblage can be checked against the assumed maximum.

Once the restraining characteristics of the beam are determined and plotted graphically the lateral deflection of the subassemblage can be checked against the assumed maximum. Figure 4b shows the final combined moment diagram plotted to the column center line.

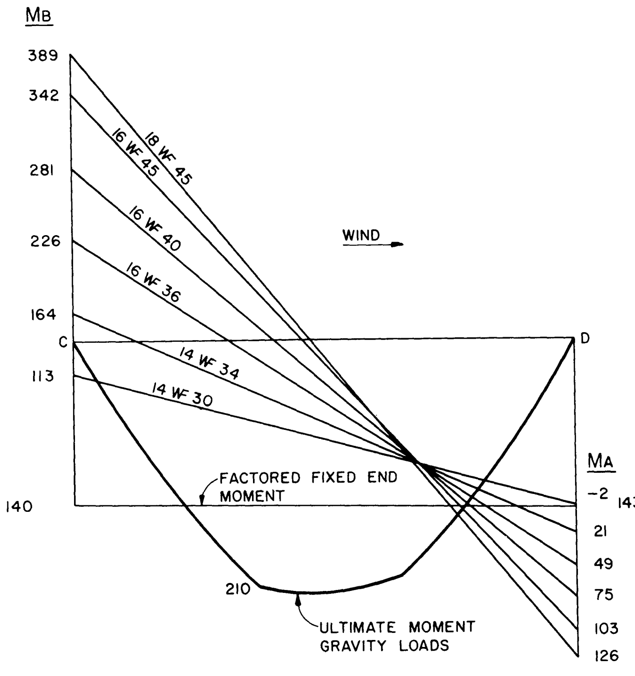


Figure 5

It is possible by this method to determine whether hinges will form at the girder ends or at the column ends depending on whether there is a strong column, weak girder condition or the opposite. The greater the number of plastic hinges which can be developed, the more economical the design will be. It is also possible to limit the number of plastic hinges consistent with design requirements. Initial drift conditions due to unbalanced gravity loads may be large enough to require that the resulting shear be added or subtracted to the shear due to wind and the $P\Delta$ effect in computing story moments. For this frame initial drift conditions were not large and therefore disregarded.

Figure 5 represents moment characteristics of a number of beams to facilitate the economical selection of sizes required for different levels at Line 3. The values of M_A and M_B are required in determining the restraining function. The sum of these values also represents the total resisting end moment. The summation of all beam end moments for a particular line must be equal to or greater than the overturning moment at the level in question. Figure 6 shows the three beams used to resist overturning at Line 3, Level 9. The overturning moment at Level 9 is 612 kip-ft. The summation of beam moments for the beams selected from Fig. 6 is 735 kip-ft. Figure 6c shows the girder restraining characteristics and the location of the hinge points for the various subassemblages. Since the last plastic hinge forms at a deflection index less than the maximum assumed ($\Delta/h = 0.02$) in computing the $P\Delta$ moment, the design is satisfactory.

Figure 6d shows the plastic hinge points in the beams and columns as well as the available moment capacity in the columns. The column moments shown are values at the top of the column in question. It is conservatively assumed that the column moment available at the bottom of the column just above the level in question is the same. Therefore the total column resisting moment is two times the sum of the values shown, or 1,026 kip-ft. The final check is that for shear. This can be determined by scaling the values of $Q_h/2M_{pc}$ for the various subassemblages on the graph (Fig. 6c) at the deflection index for the last hinge point to form. The sum of the various values of Q must be greater than the factored external lateral force. In this example the factored wind force is 43.3 kips.

The design in this example is largely controlled by the unbalanced moment condition existing at column **D** for frame buckling (load factor = 1.70). For more balanced loading in spans **CD** and **DE,** a 14W68 column with a 14W34 beam for span **CD** could have been selected, these sizes being determined by frame instability (load factor = 1.30 with wind). In referring to Fig. 2 it can be seen that there are four bays in Line 3. However, because bay **AB** is very short, the beam size was kept proportionately small and therefore was disregarded in the frame analysis. An interesting fact illustrated in Fig. 6c for beam **DE,** which has almost negligible gravity load, is the close proximity of plastic points at each end of the beam (points 2 and 4). For a condition of no gravity load on transverse beams, these points would form one common point with one restraining function. For this building, stability in the longitudinal direction is achieved by the bracing effect of the two-stair tower and the elevator shaft masonry walls. Out-of-plane deformations are prevented at each joint by rigid connections for the longitudinal beams. The shape of the building also contributes to the stability in the longitudinal direction.

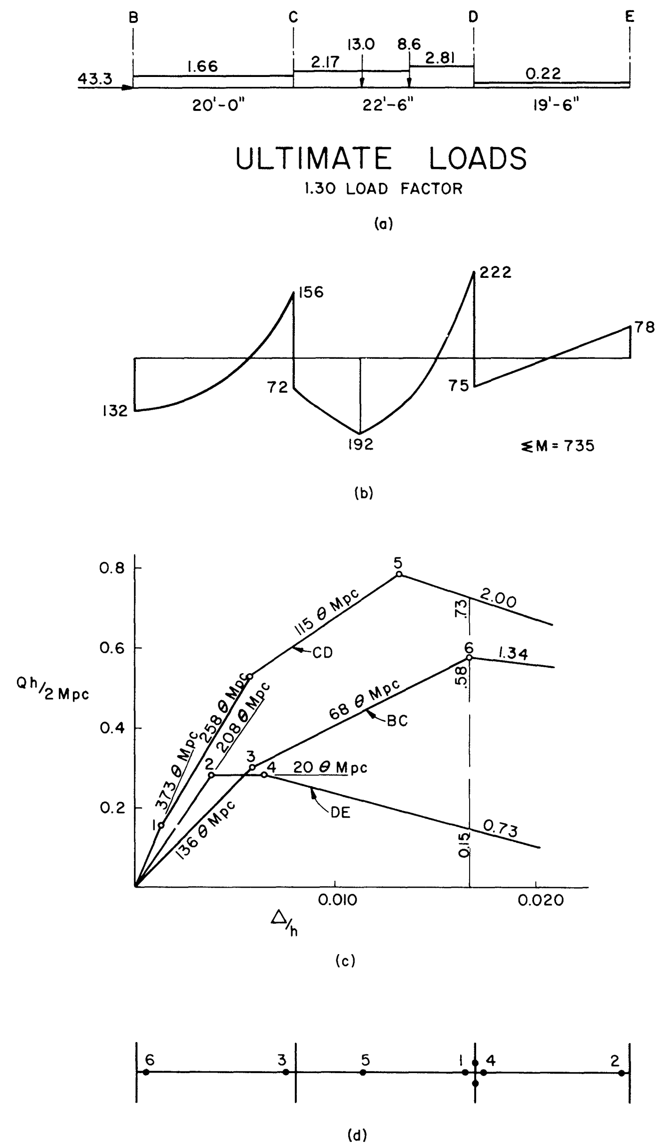


Figure 6

CONCLUSIONS

A complete analysis of all design factors was not intended in this article. It was assumed that frame buckling analysis for the condition where drift is minimal is easily handled. It would govern only the top few stories of the frame, and for these reasons was not discussed.

The writer feels that there are important advantages in the use of the plastic or ultimate strength method. After the designer has acquired the "feel" of the new method he can quickly and accurately determine frame requirements knowing that he has the most economical method available.

The designer has more freedom in evaluating how the frame will best meet varied architectural requirements. For instance, in the frame of the Binghamton Apartment Tower, beams for exterior bays were kept to a 10-in. depth and exterior columns to a 12-in. nominal depth. Since the interior bay consists of heavier 14-in columns with 16-in. beams, an unbalanced design bent existed with flexible exterior bays and stiff interior bays. The writer feels that the shear, moment and deflection requirements for this condition were best solved by the use of the plastic method.

The plastic design method also enables the designer to better understand the rotational and deflection characteristics of the beams and columns in frames where stability is important.

Significant weight savings were achieved by the use of plastic design for the project described in this paper. A design comparison with a first order elastic analysis using a horizontal drift limitation of 0.0025 shows a 3.5 percent savings in weight compared to the total steel tonnage in the building. This savings amounts to about 38 tons.