Economic Study of a Braced Multi-Story Steel Frame

JAMES B. WILLIAMS AND THEODORE V. GALAMBOS

A BRACED FRAME is a steel skeleton in which the forces due to lateral load and the sway-induced second order moments are resisted by shear walls or bracing specifically designed for this purpose. Rigid framing in such frames is not designed to resist lateral forces and serves only to decrease the size of the beams.

It is intended in this paper to describe the design and the cost analysis of a braced steel frame with both rigid framing and simple framing, and to determine how much the economy of using continuous girders is offset by the added cost of heavier columns and moment connections required for the girders.

THE PROBLEM

Shown in Fig. 1 is a typical floor plan for a 20-story building. This plan was taken at random as a practical architectural plan which could be encountered by a structural engineer. It was assumed to be typical for each floor, and the structure was designed as a braced frame for up to 34 floors using both simple framing and rigid framing. Costs were then determined and compared.

Lateral forces in the frame example are resisted by wind bracing located at each end of the building. This bracing combined with the frame forms a "wind" truss. In this paper these wind trusses are denoted by the term *wind bent*. These wind bents resist all lateral forces.

The typical braced bent consists of the interior spans of 28′-4″ and 36′-2″ and occurs nine times across the building (neglecting the special framing in the vicinity of the elevators).

It is intended to design a typical wind bent and a typical braced bent. The wind bent is designed for the 34-story frame. The braced bent is designed on an accumulative basis where the weights and costs are determined from the roof down through the building, thus giving design and cost data for frames of two stories, four stories, six stories, etc.

James B. Williams is Structural Engineer, Bank Building and Equipment Corporation of America, St. Louis, Mo.
Theodore V. Galambos is Professor of Civil Engineering, Washington University, St. Louis, Mo.

Floors are designed with a 4½-in. slab on composite beams spanning between bents. Floor loading consists of a combined live load and partition load of 80 psf. Roof construction consists of steel roof deck on steel joists spanning between bents. A 20 psf roof load is used.

DESIGN CRITERIA

A36 steel is used for girders. Columns are designed considering both A36 steel and A440 or A441 steels. Composite design is considered as an alternate in the design of girders. A slab thickness of 4½ in. is used for both alternates.

Working stress design is used in schemes using simple framing in accordance with the AISC *Manual of Steel Construction*.[1] For rigid frame schemes, plastic design is

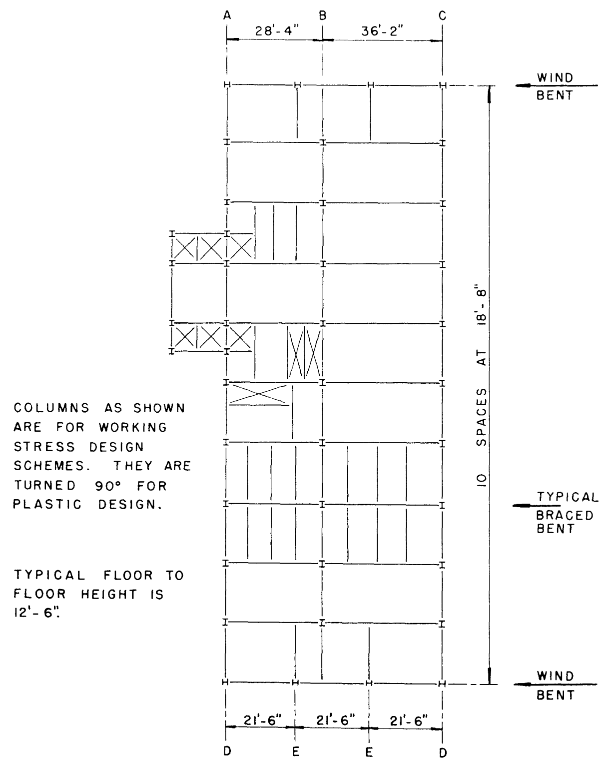

Fig. 1. Typical floor plan

used as recommended in the lecture notes and design aids booklet prepared for a special conference on Plastic Design of Multi-Story Frames held at Lehigh University in August of 1965.[2, 3]

By combining simple or rigid framing, A36 steel or high strength steel columns, and composite or non-composite girders, there are eight possible design schemes. These schemes are denoted in various tables or figures by the following abbreviations:

Simple Framing Using Working Stress Design:

- W_n A36 columns, non-composite girders
- W_{nh} A440 columns, non-composite girders
- W_c A36 columns, composite girders
- W_{ch} A440 columns, composite girders

Rigid Framing Using Plastic Design:

- P_n A36 columns, non-composite girders
- P_{nh} A441 columns, non-composite girders
- P_c A36 columns, composite girders
- P_{ch} A441 columns, composite girders

TYPICAL BRACED BENT

Columns are assumed to be spliced at alternate floors. This gives lengths of 25′-0″, a reasonable length for handling and ease of erection. The effective column length factor K is taken as 1.00 for columns in a braced frame.

Live loads and live load reductions are taken as recommended in the *National Building Code*.[4]

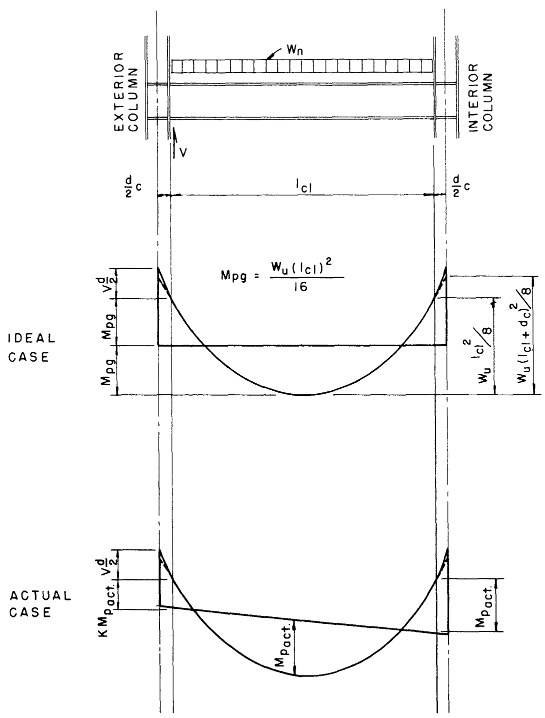

Fig. 2. Critical girder moments by plastic design

Working Stress Design—Girders are designed simply by $M = Wl/8$ using A36 steel. Column design is governed by buckling about the weak axis. Girders are framed into the column webs in order to make any moment due to girder reactions negligible. It is assumed that such moments are more critical to the exterior column design than the increased Kl/r distance resulting from not being able to brace the columns on their weak axis in the plane of the exterior wall

Plastic Design—The braced bent does not resist lateral forces. Gravity loads govern the design; for this the load factor is taken as 1.70.

Clear distances between columns are used rather than centerline distances in the calculation of the girder moments. This increases the column moments by $Vd/2$ where d is the depth of the column and V is the end shear. This is found to save approximately 16 percent of girder weight on the 28′-4″ span and 11 percent on the 36′-2″ span. The column moment requirements are increased somewhat, but not enough to make a critical difference in design, particularly in the lower stories where the column moment capacities are much more than adequate.

Girder sections chosen are the lighest shapes available that satisfy moment requirements. Often the sections chosen have a much greater capacity than needed. It is advantageous in the design of exterior columns to determine a reduced moment capacity required due to the greater capacities at the interior column and at the point of critical positive moment. The moment required at the exterior column is denoted as KM_p, where K is determined from Ref. 5. (See Fig. 2.)

The factor K is not so conveniently obtained for composite girders, but can be calculated from the following equations by trial and error:

$$x_{cr} = \frac{L}{2} - (1 - K)\frac{M_p}{w_u L}$$

$$\frac{w_u x_{cr}}{2}(L - x_{cr}) = KM_p + \frac{(1 - K)}{L} x_{cr} M_p + M_{p\,cb}$$

where

- x_{cr} = location of maximum positive moment
- L = clear span
- M_p = plastic moment of steel section above
- w_u = factored uniform linear load
- $M_{p\,cb}$ = plastic moment of composite section

Bracing of the compression flange of the girders must be provided at a maximum distance of $L_{cr} = 65r_y$ (A36 steel) where r_y is the radius of gyration of the girder about the y-axis.[2] This is a problem only in the negative moment regions where the compression flange is not continually supported by the slab. In this case it is found that the spacing of the first interior beam is decreased slightly, but not enough to change the beam designs. A plate stiffener is welded between the bottom of the beam and the top of the girder bottom flange. (See Fig. 3.)

Columns must be of sufficient size to resist axial loads and provide at least enough moment capacity to resist the required plastic moment in the girder at exterior columns or the difference in required plastic moments at interior columns. Both the columns above and below the girder are chosen so that their total resisting moment is equal or greater than the applied girder moments.

The maximum resisting moment of a column, M_{pc}, is determined from tables in Ref. 3 for various axial force ratios, P/P_y. The actual moment capacity is likely to be less than M_{pc} due to column instability. The percentage of M_{pc} which a column is capable of developing is determined from moment-rotation curves for columns of double symmetrical curvature for various slenderness ratios.[3]

Checkerboard loading will have no appreciable effect on the design of exterior columns. It may, however, affect the interior columns and has been considered in this design. Such loading in the vicinity of a column will tend to cause single curvature rather than the double curvature caused by full loading.

The column sections as previously selected for full loading were checked for checkerboard loading. To be satisfactory, at least one rotation needed to be found at each joint at which the plastic moment of the longer span could be resisted by the girder from the shorter span and the columns above and below the joint.

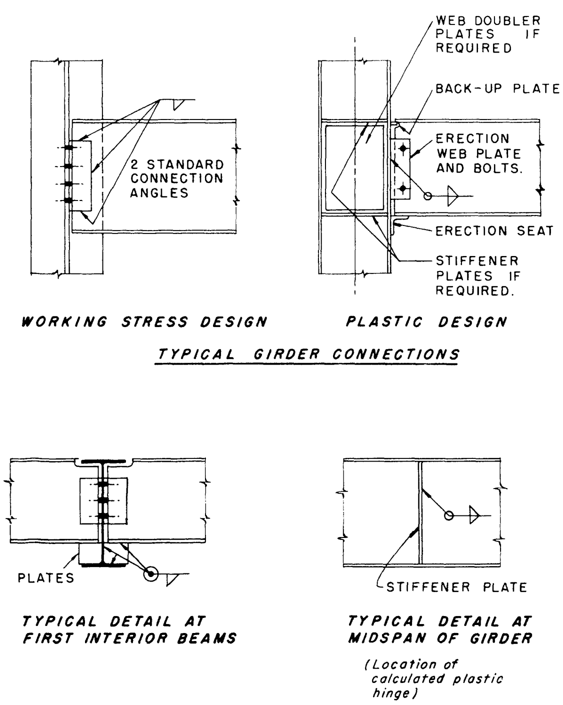

Fig. 3. Typical details

In this frame example, the effect of checkerboard loading was not found to be critical except in the upper floors. The only column size to change is the A36 column at the 34th floor. All other columns remained the same as determined by full loading.

Rigid beam-to-column connections are achieved by full penetration butt welds made in the field after the girders are erected with the aid of small seat angles, which also serve as back-up plates for welding, and two erection bolts through the girder web into a plate shop-welded to the column. (See Fig. 3 for various connection details.)

Plate stiffeners and web doubler plates, if required, are shop welded to the columns. The requirements for these plates were determined by the formulae for shear, web crippling and buckling, and tension distortion of the column flanges as presented in Chapter 5 of Ref. 2.

Cost Calculations—The erected cost of steel varies because of many factors. Location affects labor costs and cost of freight. The amount of profit may vary, depending upon how greatly the bidders want the job.

However, the major considerations in an economic comparison are relative costs of the alternates considered. In this report, relative cost differences in the following areas are taken into account:

1) High strength steel versus A36 steel as used in columns
2) Extra cost of shear connectors in place on composite beams
3) Field welded connections on the rigid frame versus conventional bolted connections used with simple framing
4) Extra fabrication costs of beams by plastic design

Costs are calculated for the various frame elements by the use of unit costs in terms of cents per pound of erected steel.

One very good rough estimate of unit costs is suggested as follows:[6] A reasonable cost for steel in place is 18¢ per pound. Of this amount, 10¢ is for material, the remainder for labor. High strength steel may be estimated 1½ to 2¢ more per pound, closer to 2¢ for the heavier shapes. Extra costs for rigid connections will increase the unit cost approximately 10 percent.

A further attempt has been made to determine unit costs in more detail. The unit cost has been subdivided into material, installation common to all alternates, and extra installation due to specific alternates.

Material cost consists primarily of the mill base price with added extras for section, length, cutting, quantity and freight. A chemical extra is added for high strength steel.[7] Added to this are various shop operations such as drafting, shop fabrication, and shop paint.[8]

Installation costs common to all members include items such as the following:[8]

1) Unloading at the job site
2) Erection and plumbing
3) Crane and minor erection equipment
4) Field paint
5) Overhead and profit, approximately 25 percent

No allowance has been made for any extra erection costs on a multi-story frame. This is an error common to all alternates and might amount to $15 per ton.[8] Also, direct costs of column-splices and base-plates are neglected but are assumed as part of the erection cost.

The installation cost of high strength steel columns is arbitrarily increased by 5 percent since less weight is handled with essentially the same amount of labor.

Simple framing connections are figured as standard connection angles shop welded to beams and field bolted to columns.[1] The cost of this type of connection might be reduced slightly by an economical designer, but with spans of the magnitude in the frame example, very little saving is anticipated from revising these standard connections.

Rigid connections are figured as shop welded to as great an extent as possible. The cost of shop welding is calculated as 45 cents per linear foot of $\frac{3}{16}$ or $\frac{1}{4}$-in. fillet weld.[7] Full penetration butt welds of the girder flanges to the columns are made in the field. Field welding is assumed to cost $8.25 per hour,[7, 8] with the time required for making one girder connection as 20 minutes.[7]

Extra costs to beams by plastic design consist of stiffener plates shop welded to the girder web at the calculated location of maximum positive moment. Also, stiffener plates are field welded between the bottom of the first interior beam at each end of each girder span and the top of the bottom flange of the girders. (See Fig. 3.)

TYPICAL WIND BENT

In a high-rise office building, it is usually architecturally desirable to have as much open floor area as possible. It is not usually very satisfactory to have wind bents in the middle of an "open" office area or a corridor. This generally limits the wind bracing to the outside walls.

In this building example, which is quite long and narrow, bracing the end walls becomes quite a problem, particularly since the plan chosen was for a 20-story building and 14 stories were arbitrarily added. Bracing the end walls results in a maximum of three braced bays of 21′-6″, taking five bays of wind. The building height-to-braced bay ratio is 425/64.5 = 6.6. Multiplying this ratio by 5 bays of load gives a relative moment-to-depth ratio of 33.0.

As an alternative, it may be possible to brace selected interior bents. It would seem impractical to have any bracing in the 36′-2″ span, the logical location of a central corridor. Similarly bracing the 28′-4″ span at every bent would require fixed partitions at 18′-8″ o.c., also architecturally undesirable. Bracing the 28′-4″ span at alternate bents is architecturally less objectionable. However, the resulting moment-to-depth ratio is 2 bays $\times$ 425/28.3 = 30.0. This is hardly an improvement over bracing only the end walls and is architecturally less desirable. On this basis, it was decided to locate all the wind bracing in the end bents.

Design Criteria—Plastic design of the bent members is governed by either gravity load alone times a load factor of 1.70 or the combined wind and gravity load times a load factor of 1.30.

Wind loads are assumed to be concentrated loads applied at each floor. X-bracing is used. Only the tension brace is considered as effective. For determination of forces in the bracing and deflections, all joints are assumed pinned. Frame resistance is neglected. The bracing force in the tension diagonal is calculated from the cumulative wind load plus the column loads acting on the sway angle ρ.

The sway angle ρ is defined as the ratio of lateral deflection between floors to the floor-to-floor height. In standard practice an allowable value of ρ is often taken as 0.002, although values of 0.004 to 0.005 have been used with satisfactory results.[9] In this example frame, the allowable value of ρ at working loads was arbitrarily assumed as 0.003. It is assumed that this higher value is justified due to the construction materials and architectural functions of the building. For plastic design this value is multiplied by the load factor 1.30; for load calculations $\rho = 0.004$.

In preliminary designs, deflection was found to govern the wind bent design and it was desirable to brace all three bays for the full height of the building. Each bay was assumed to share equally in lateral load distribution.

Design by Deflection Criteria—It was found from preliminary designs that the exterior columns contribute from 60 to 90 percent of the deflection at the roof. Obviously, the best way to limit sway deflections was to use much larger sections than needed for strength in the exterior columns, particularly in the lower floors.

Stresses in all the members designed by this deflection criteria are quite low. Whether the wind bent is designed by plastic design or working stress design thus makes no difference except in the design of the girders.

Except in the top few floors, girders designed by plastic design have a much greater moment capacity than required. The required girder moments are quite small compared to the axial load due to wind. Not too many floors below the roof it would seem reasonable to use simple beam connections and still have sufficient moment capacity. Working stress design results in deeper beams, having approximately an equal or slightly greater area than those determined by plastic design. These is also a slight saving in connection costs.

In a deflection analysis in which any frame resistance is neglected (all joints are assumed pinned) there is no advantage in using plastic design with moment connections when the design or the girders is governed by lateral axial loads acting on the wind bent.

SUMMARY AND CONCLUSIONS

It is intended in this paper not only to compare the weights of the various design schemes, but also to allow for the higher costs of the "lighter" schemes in order to give a more realistic economic comparison.

The design program is separated into two categories: (1) design of a typical braced bay resisting no lateral forces and (2) the design of a typical wind bent. The wind bent of the given frame example is a rather special case and is discussed first.

Typical Wind Bent—Deflection is the governing design criteria of the wind bent. The building height-to-depth ratio is quite large and architectural considerations limit wind bracing to the exterior wall at each end.

The bent is designed as a truss with all joints pinned. Any advantage of using rigid framing is largely eliminated by neglecting any frame resistance. In the preliminary bent, the contribution of the rigid frame was found to be very small. This resistance may not be quite as negligible in the final design in which the bent members are considerably larger.

Neither composite construction nor high strength steel was considered for the wind bents.

Girders by plastic design are found to weigh 7½ percent less than those by working stress design, although they are found to cost 2 percent more. This small difference in cost is negligible when considering that the girders are a small portion of the wind bent compared to the columns and bracing members which are the same by either plastic or working stress design; therefore no cost comparison is made for the wind bent.

Typical Braced Bay—The major interest of this paper is in the design of the braced bay. A comparison is made between the various design schemes for bent heights ranging from two to 34 stories. Tables 1 through 4 show member sizes designed for the different schemes.

As one would expect, columns in the plastic design schemes are generally heavier than comparable working stress columns. However, in lower stories where girder moments are small in relation to column loads, many plastically designed columns are the same or even lighter.

Cumulative weights and costs for the braced bay for varying frame heights are illustrated in Figs. 4 and 5.

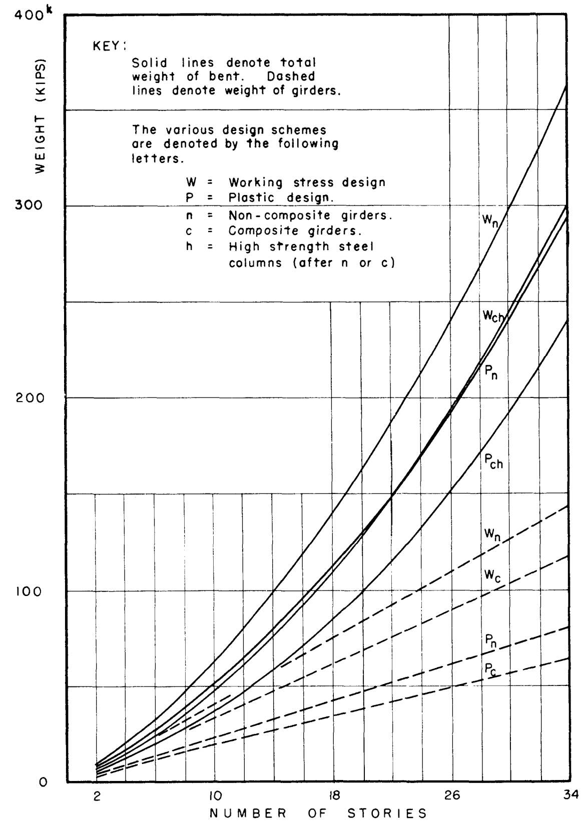

Fig. 4. Cumulative weights for typical braced bay

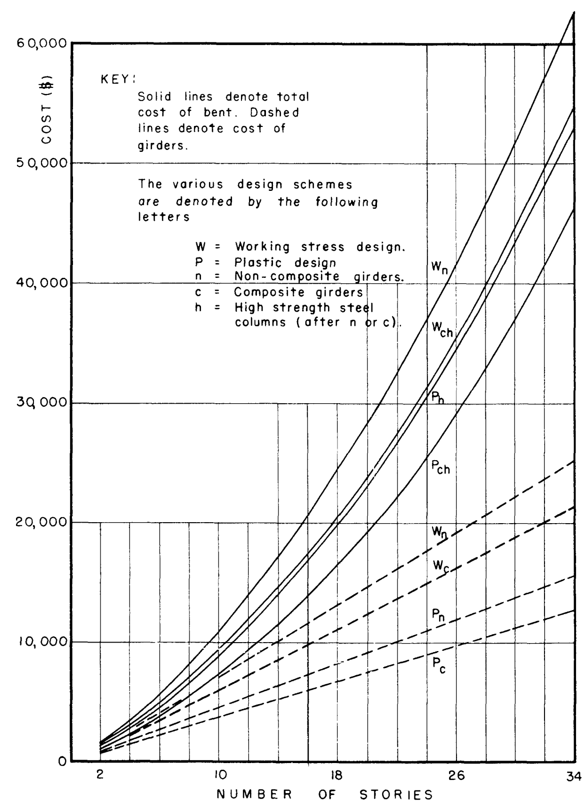

Fig. 5. Cumulative costs for typical braced bay

For clarity, only the two extreme cases of each main design classification, plastic or working stress, are shown. The dashed lines denote the portion of each scheme consisting of girders.

Table 1. Summary of Braced Bay Girder Designs

Span	Working Stress Design				Plastic Design			
	Non-composite girders	Unit cost (¢/Lb)	Composite girders	Unit cost (¢/Lb)	Non-composite girders	Unit cost (¢/Lb)	Composite girders	Unit cost (¢/Lb)
Roof:								
28′-4″ span	14WF30	18.30	—	—	14B17.2	23.25	—	—
36′-2″ span	18WF45	17.50	—	—	16B26	20.45	—	—
Typical floor:								
28′-4″ span	21WF55	17.75	18WF45	18.50	16B31	20.35	16B26	21.45
							14B22	21.75
36′-2″ span	24WF76	17.35	21WF62	17.75	18WF45	18.60	16WF36	19.55

Table 2. Summary of Braced Bay Column Designs.
Column A: Exterior Column on 28′-4″ Span

Floor	Working Stress Design		Plastic Design			
			Non-composite		Composite	
	A36	A440	A36	A441	A36	A441
34						
33	8WF24	8WF24	8WF35	8WF28	8WF24	8WF20
32						
31	8WF31	8WF24	10WF45	8WF40	8WF28	8WF20
30						
29	10WF45	8WF40	12WF53	10WF45	10WF45	8WF40
28						
27	10WF66	10WF49	14WF68	10WF66	10WF60	10WF45
26						
25	12WF79	12WF65	14WF78	10WF66	12WF79	Same as
24						non-
23	12WF99	12WF72	14WF111	10WF72	12WF99	com-
22					Same as	posite
21	14WF111	12WF85	14WF111	10WF89	non-	
20					com-	
19	14WF127	12WF99	14WF127	12WF92	posite	
18						
17	14WF142	14WF103	14WF142	12WF106		
16						
15	14WF158	14WF119	14WF158	14WF127		
14						
13	14WF176	14WF127	14WF176	14WF127		
12						
11	14WF193	14WF136	14WF184	14WF136		
10						
9	14WF202	14WF158	14WF202	14WF167		
8						
7	14WF219	14WF176	14WF219	14WF176		
6						
5	14WF237	14WF184	14WF237	14WF184		
4						
3	14WF246	14WF202	14WF246	14WF202		
2						
1	14WF264	14WF211	14WF264	14WF228		

Figures 6, 7, and 8 compare all schemes to a basic scheme which utilizes: (1) working stress design, (2) non-composite girders, and (3) A36 steel columns. These charts illustrate the relationships of cost and weight of the girders, columns and entire frame.

The use of composite construction is found to be more economical than non-composite construction in all schemes. Nearly all of the savings are in the girders, but there is also a considerable savings in the relative weight and cost of columns by plastic design, particularly in the higher floors. Due to the greater positive moment capacity of composite girders, required negative moment capacities are not as great. Subsequently, some column sizes were reduced.

In working stress schemes where columns are not required to resist moment, A440 steel columns are found to be more economical in frames of four or more floors. In plastic design schemes, A441 steel columns are more economical for all frames, even two-story frames.

Economy is certainly indicated with the use of plastic design. At all floors, every plastic design scheme is more economical than the comparable working stress design. In fact, the least economical plastic scheme is less expensive than the most economical working stress scheme.

The same economy may not result on a small job where field welding is not readily available.

Unit costs are calculated for the braced bay at various story heights. The results are tabulated in Fig. 9.

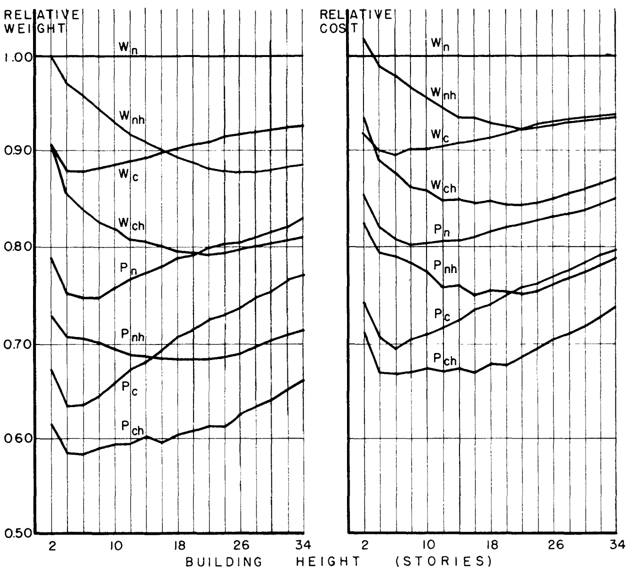

Fig. 6. Relative comparisons of weights and costs of typical braced bay design schemes

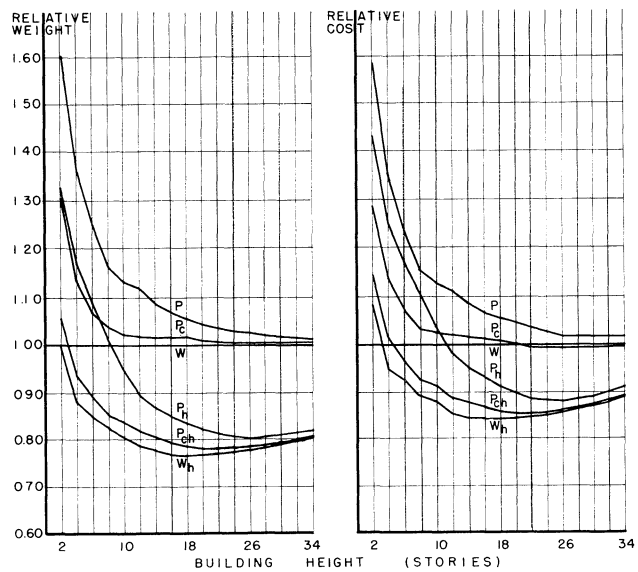

Fig. 8. Relative comparisons, columns only

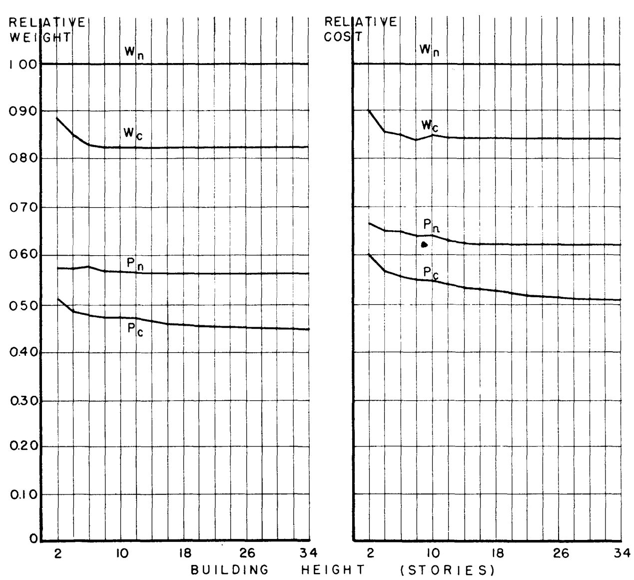

Fig. 7. Relative comparisons, girders only

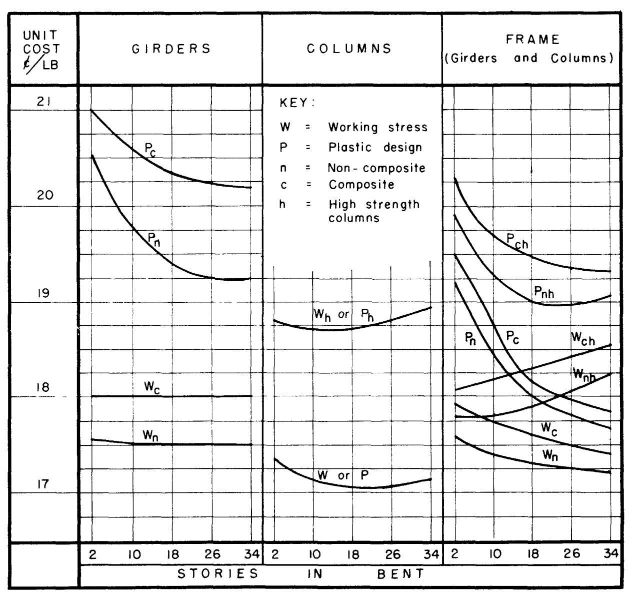

Fig. 9. Unit costs for the typical braced bay

Costs of connections are included in girder unit costs. Unit costs for girders by working stress design are nearly constant for all frame heights. Unit costs for girders by plastic design are somewhat less in the lower floors because of the elimination of stiffener plates and web doubler plates at the heavier columns where webs and flanges are of sufficient thickness to resist girder forces.

Composite girders are from $\frac{1}{2}$ to 1 cent per pound more than non-composite girders for both types of framing. The extra cost of rigid framing required for plastic design increases the unit price of girders 2 to 3 cents per pound.

Unit costs for columns do not vary for either plastic or working stress design since connection costs are reflected in girder costs. High strength steel increases the unit cost of columns about $1\frac{1}{2}$ to $1\frac{3}{4}$ cents per pound.

Bent unit costs, combining both girders and columns, indicate higher unit costs for plastic design ranging from about 2 cents per pound more for a two-story frame to about $\frac{1}{2}$ or 1 cent more for a 34-story bent.

Conclusions—No conclusion is made in regard to the relative merits of plastic or working stress design in the wind bents. The typical wind bent is designed to satisfy deflection criteria for the 34-story bent and no significant difference results in the use of either type of design.

The typical braced bent is designed for two to 34 stories and weights and costs are tabulated on a cumulative basis. For the two-story bent, plastic design schemes have unit costs 10 to 12 percent higher than working stress schemes. This increase is only 3 or 4 percent for the 34-story bent. However, the savings in weight with plastic

Table 3. Summary of Braced Bay Column Designs.
Column B.: Interior Column

Floor	Working Stress Design		Plastic Design			
			Non-composite		Composite	
	A36	A440	A36	A441	A36	A441
34						
33	8WF24	8WF24	8WF35	8WF28	Same as non-composite	Same as non-composite
32						
31	8WF48	8WF40	8WF48	8WF48		
30						
29	10WF72	10WF54	10WF66	8WF58		
28						
27	12WF92	12WF72	12WF85	10WF72		
26						
25	14WF111	14WF87	14WF111	10WF89		
24						
23	14WF136	14WF103	14WF136	12WF99		
22						
21	14WF158	14WF119	14WF150	14WF127		
20						
19	14WF184	14WF136	14WF176	14WF127		
18						
17	14WF202	14WF167	14WF202	14WF167		
16						
15	14WF228	14WF184	14WF219	14WF176		
14						
13	14WF264	14WF202	14WF246	14WF193		
12						
11	14WF287	14WF237	14WF264	14WF211		
10						
9	14WF314	14WF264	14WF287	14WF264		
8						
7	14WF320	14WF287	14WF314	14WF287		
6						
5	14WF342	14WF314	14WF342	14WF287		
4						
3	14WF370	14WF320	14WF370	14WF314		
2						
1	14WF398	14WF342	14WF398	14WF342		

design more than offset the higher unit cost. In general, plastic design results in savings ranging from 12 to 20 percent.

Unit prices are also increased with the use of composite construction and high strength steel columns, but not enough to offset a savings in most cases of from 2 to 8 percent.

The most economical scheme for all bent heights was found to consist of 1) plastic design, 2) composite girders, and 3) high strength columns.

Comparing this to the least economical scheme consisting of 1) working stress design, 2) non-composite girders, and 3) A36 columns, it is found that for the various bent heights the savings in weight ranges from 35 to 40 percent. The corresponding savings in cost ranges from 26 to 33 percent. The obvious economic solution for the braced bents of this structure is the scheme where plastic design, composite construction and high-strength steel columns are used.

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This report is a condensation of a thesis presented by the first author in partial fulfillment for the Master of Science degree at Washington University of St. Louis, Mo. The thesis contains the detailed description of the design and the cost analysis.[10]

Table 4. Summary of Braced Bay Column Designs.
Column C: Exterior Column on 36′-2″ Span

Floor	Working Stress Design		Plastic Design			
			Non-composite		Composite	
	A36	A440	A36	A441	A36	A441
34	8WF24	8WF24	10WF45	8WF40	8WF35	8WF28
33						
32						
31	10WF39	8WF31	12WF50	10WF45	10WF45	8WF40
30						
29	12WF53	10WF45	14WF68	10WF66	12WF58	10WF45
28						
27	12WF72	12WF58	14WF84	10WF66	12WF79	10WF66
26						
25	12WF92	12WF72	14WF111	10WF77	12WF92	10WF72
24						
23	14WF111	12WF85	14WF119	10WF89	14WF111	10WF89
22						
21	14WF127	14WF95	14WF136	12WF99	14WF127	12WF92
20						
19	14WF142	14WF111	14WF136	12WF120	Same as non-composite	12WF106
18						
17	14WF167	14WF119	14WF167	14WF127		Same as non-composite
16						
15	14WF184	14WF136	14WF193	14WF136		
14						
13	14WF202	14WF158	14WF202	14WF158		
12						
11	14WF219	14WF176	14WF219	14WF176		
10						
9	14WF237	14WF193	14WF237	14WF193		
8						
7	14WF264	14WF202	14WF264	14WF202		
6						
5	14WF287	14WF237	14WF287	14WF237		
4						
3	14WF314	14WF264	14WF314	14WF264		
2						
1	14WF314	14WF287	14WF314	14WF287		

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Plastic Design of Multi-Story Frames—Lehigh Lecture Notes

Lehigh University reports that the Lecture Notes and Design Aids printed in limited quantities for the August, 1965 Summer Conference have been reprinted and are still available. Copies may be purchased from Lehigh University at a price of $5.50 per set, including mailing costs.

These lecture notes and design aids are usable both in the classroom and the design office. In addition to the theoretical discussions of plastic design for multi-story frames which lead to design recommendations, there are numerous design examples tabulated in a manner appropriate for office and/or computer usage.

Requests should be sent to:

Dr. George C. Driscoll, Jr.
Fritz Engineering Laboratory
Lehigh University
Bethlehem, Pa. 18015