LATERAL BRACING SYSTEMS IN HIGH RISE STEEL FRAMES

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ABSTRACT OF THESIS

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This thesis describes a study of lateral bracing systems in moderately sized steel frames. The objective of the study is to investigate the performance and economy of nine different bracing systems in a specified plane frame. An automated design program, which is described in two appendices, is used to size the members in the frames. The results show the relative economy and performance of the various bracing systems. Results concerning stiffening of frames to reduce wind drift, portal and cantilever approximate analysis methods and bracing performance during gravity loading are also presented and discussed.

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Chapter 1

INTRODUCTION

Choosing an efficient and acceptable lateral bracing system is necessary in the design of nearly all frames. Lateral loads arising from wind or seismic action must be transmitted to the building's foundation by the structural system. A large number of possible systems exist, but questions about their relative economies have proved difficult to answer. The "best" system will be determined by a number of factors, including labor and material costs, architectural considerations and the structure's location.

The objective of this investigation is to study the performance and relative efficiencies of several types of lateral bracing systems with the aid of a computer program for automated design. Each of nine different lateral bracing systems were placed in a frame and a standard steel section listed in the AISC Steel Construction Manual (2) was selected by the design program for each member in the frame. Studies of each system and comparisons of the systems are discussed in detail. The final design weight of each frame provides some measure of its relative economy. The techniques used to stiffen a frame to reduce wind drift are also explained and observed results are organized into general trends.

1.1 Literature Review

Studies of the performance and relative economies of lateral bracing systems conducted with the aid of an automated design program have received little attention in the technical literature. Although alternate structural systems are often compared by designers in the preliminary phases of building design, and computer-aided analysis and design tools are becoming increasingly available, this researcher could find no reports in the technical literature which dealt directly with the subject of this thesis. There is, however, some literature available on either the performance of lateral bracing systems or on automated design programs.

Condit (7) has provided an excellent article covering the historical aspect of wind bracing. He begins with a description of a medieval carpenter's knee braced timber frame. Then he traces the development of lateral bracing in cast iron and steel structures by the mid-nineteenth century French engineers, including Gustave Eiffel, designer of the Eiffel Tower. Finally, Condit looks at the lateral bracing system used in the Chase Manhattan Building, which was completed in 1961.

The optimization of overall structural steel building systems is discussed by Iyengar (14). A presentation of the basic types of building systems and their economical height ranges is shown. A brief, general technique for selecting an optimum structural system from several alternative systems is also explained. Brief discussions relating computer programs, shear lag, floor framing, and foundations to overall structural efficiency can also be found in this article.

Articles relating to the performance of specific types of lateral bracing proved to be scarce. Gaylord and Gaylord (10) and Merritt (17) have discussions about bracing types and general bracing theory. A general description of various framing systems and their performance is discussed by Scalzi (20). Taranath (22) has investigated a technique for finding the optimum placement for single or double outrigger trusses in a braced frame. Khan (16) discusses the present and future of tall buildings, relating structural systems to overall urban environment.

Several very good automated design computer codes are available. Generally, these codes perform tasks similar to program DESIGN1, the automated design program used in this thesis. However, the different codes tend to vary in their solution methods.

AUTOTIER, a program described by Agaskar (1), utilizes methods which are very similar to DESIGN1. AUTOTIER requires an initial input of design geometry, loadings, and other parameters. The program runs without interfacing with the design engineer, automatically making design decisions and arriving at a final set of fully stressed steel members. AUTOTIER differs from DESIGN1 in three significant ways: it considers finite joint sizes, it can work in all three dimensions, and it can consider five loading cases internally during the design. DESIGN1 must work in two dimensions and must interface with the programmer for each loading case.

STRUDL-II (STRUctural Design Language) (11), a component of the ICES (Integrated Civil Engineering System) project, has a steel design option. Using a computer language developed for structural problems, the user specifies the type of structural system, and its geometry, loading and initial member trial sizes. STRUDL then analyzes and checks the appropriate AISC code equations for the initial trial member sizes. The designer must then interface with the computer, indicating which members are to be redesigned by the program. After the redesign, the user orders a reanalysis and then another design check. This continues until a final set of adequate members is found. Many other options exist with STRUDL (12), including reinforced concrete design, nonlinear analysis, linear buckling analysis and dynamic analysis.

Another program with steel design capabilities is STAND (STructural ANalysis and Design) which was developed by S. L. Chu (6) at Sargent and Lundy Engineers. STAND is very similar to STRUDL in its use of a design language and its interfacing with the design engineer. STAND has considerably less capacity for analysis than STRUDL, but it has more design capabilities in rolled beam and column selection, composite beam design, plate girder design, and column base plate design.

Other automatic design programs similar to AUTOTIER, STRUDL-II and STAND exist. Smith and Wilson (21) present a computer technique for the automatic design of shell structures. Efforts are being made to combine optimization routines into automatic design programs. Kavlie and Moe (15) have developed an automated design optimization routine for grillages and frames. Their discussion briefly covers most of the major nonlinear programming techniques currently used, and places particular emphasis on the sequential unconstrained minimization technique (SUMT). Gallagher and Zienkiewicz (9) present the major techniques of structural optimization in greater detail.

Chapter 2

THE FRAME AND THE LATERAL BRACING SYSTEMS

Many types of lateral bracing systems can be used to resist the lateral loads imposed on a structure. The more conventional systems for moderately high structures (10 to 40 stories) can be placed into three broad classes: rigid frames, vertical trusses and combinations of the frames and trusses. In addition, framed tubular systems are becoming more common. Rigid frames rely on the member flexural stiffnesses and the rigidity of the connections in the structure to carry lateral loads. Vertical trusses use the stiffness of a cantilevered truss to resist the lateral loads. Combination systems place vertical trusses within a rigid frame.

To study the relative efficiencies of several lateral bracing systems, a plane frame of given size and loading was designed using nine different systems. These bracing systems include one rigid frame, two vertical trusses and six combination systems.

2.1 Rigid Frames

In the rigid frame, all joints are assumed to be fully moment resistant (AISC Type I) (2). This causes the beams and columns to act together to resist lateral loads through their flexural stiffnesses. Fig. 2.1 shows a rigid frame subjected to lateral loads. The frame deforms in a sidesway mode, with the flexural deformations in the beams and columns causing the typical deflected shape shown. The horizontal forces are resisted by the column shears and the corresponding overturning moment is resisted by the column axial load couple and the column bending moments.

Several advantages can result from the use of a rigid frame. The spans are simple and unobstructed, allowing maximum architectural flexibility. The member end fixity resulting from the rigid connections can also result in reduced beam sizes if the lateral loads do not cause significant moments. However, the sidesway motion of the frame can result in the buckling behavior of the columns being considerably more critical (i.e. $K \geq 1.0$) than that corresponding to the basic pinned end condition. This increase in effective length is demonstrated in Fig. 2.1. The result of the larger effective length factors is usually heavier column sections in comparison to a braced frame design, for which $K \leq 1.0$. In addition, lateral loads cause significant moments in the lower beams and columns of a multistory rigid frame which necessitates the use of larger, heavier sections in this area. This can occur in rigid frames as small as five stories high.

Lateral deflections arising from the S shapes taken by the columns are also seen in Fig. 2.4a. The drift is influenced by both the beam and column stiffnesses. Relative story to story drift deflections tend to be somewhat greater in the lower part of a rigid frame where the story shear is a maximum and tend to decrease in the upper stories where the story shear decreases. The larger member stiffnesses at the lower floors act to minimize these differences in deflection.

2.2 Vertical Trusses

All connections in the vertical truss are assumed to carry no moment (AISC Type II). Diagonal bracing members are required at each floor to insure the stability of the structure (Fig. 2.2). Lateral loads are carried by the vertical truss to the foundations. The vertical truss is formed by the beams, columns and bracing which are in

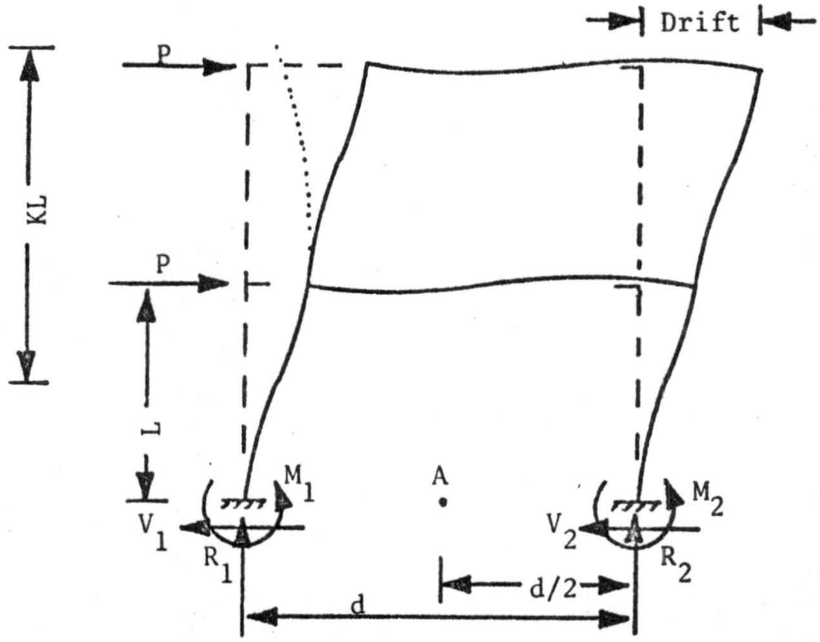


Fig. 2.1 Small rigid frame resisting lateral loads.

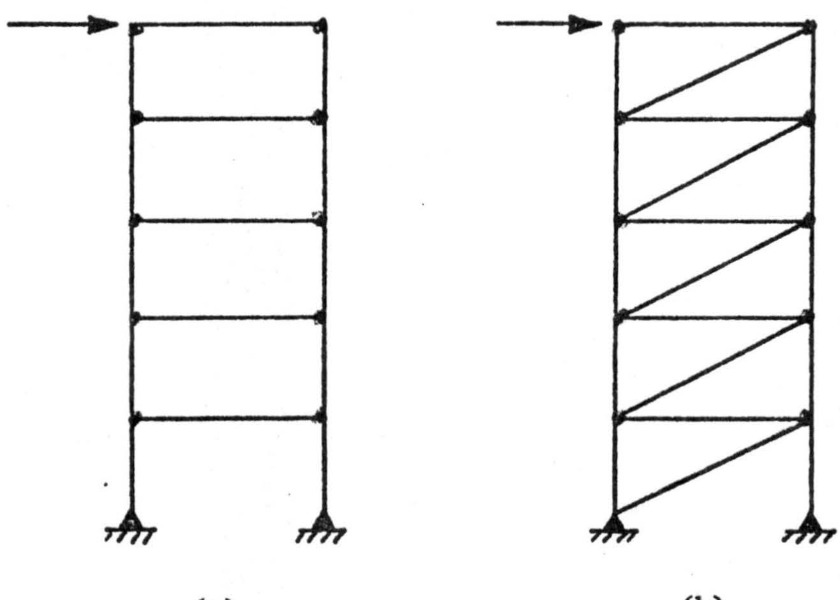


Fig. 2.2 An unstable (a) and a stable (b) vertical truss.

the braced bays of the structure. Lateral loads on unbraced bays must be carried to braced bays by rigid floor diaphragms.

Pinned connections create an effective length of one in every member of the vertical truss. This smaller effective length factor helps reduce the amount of column steel from that needed in the rigid frame. However, the beams do not enjoy the benefit of end restraint which exists with rigid end connections and must support the entire simple beam moment. Pinned connections have some economic advantages because of their simplicity and the resulting ease of joining members in the field.

The bracing members necessary in the vertical truss can cause significant architectural problems. The presence of bracing diagonals limits the size and shape of openings in the bay and may be considered aesthetically objectionable. This effect can be minimized by placing the bracing in utility shafts or in service areas, or by choosing a less restrictive bracing configuration. Two types of internal bracing systems were investigated, X and K. Each type has one or more variations.

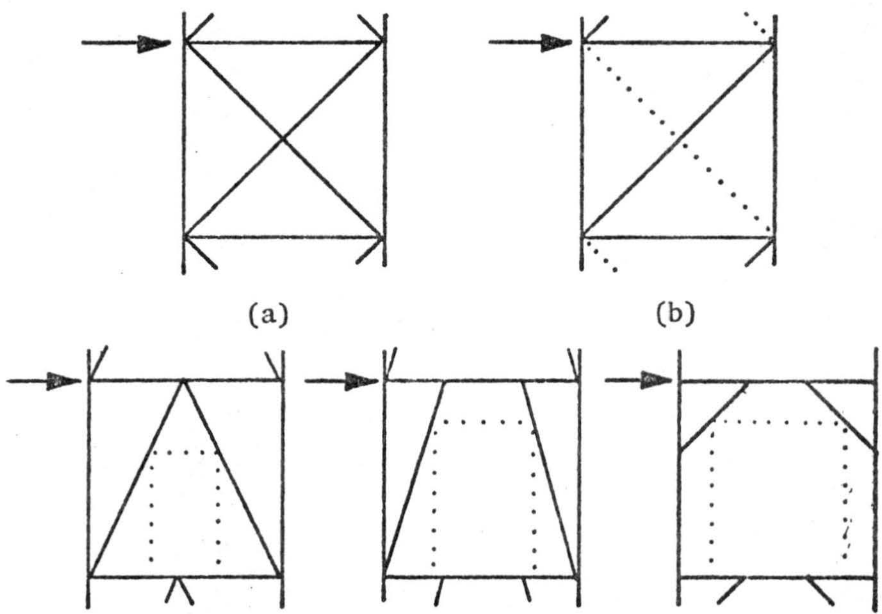
X bracing (Fig. 2.3a) carries the lateral load through both diagonals, one acting in tension and the other in compression. Often there is no effective connection where the two members of the X bracing cross, which can result in very high bracing member slenderness ratios and thus severely limit the allowable stress in the compression diagonals. Since equal sizes in both diagonals are usually required for symmetry, the tension member is often grossly under-stressed. A variation of X bracing, called counters (Fig. 2.3b), allows the tension member to control the bracing size. It is assumed that the compression diagonal buckles and that the entire lateral load is carried through the beam to the tension diagonal. Because tension governs the design, it is not necessary to use structural shapes for the diagonals. Rods, angles and tees are often used in a counter system.

The behavior of K bracing (Fig. 2.3c) is similar to X bracing, but the reduced lengths of the bracing members cause an increase in their stiffnesses and allowable stresses. Note that the K bracing permits a larger opening through the bay than is possible with X bracing. If still larger openings are required, the K bracing may be split (Fig. 2.3d). With this arrangement, additional moments are introduced into the horizontal members of the truss. Knee bracing (Fig. 2.3e) provides the largest opening, but at the cost of causing sizable moments in the columns because of the horizontal force component where the bracing connects to the column.

Drift deflections in the vertical trusses are primarily due to the axial deformations of the columns caused by the wind component of the combined wind and gravity loads. The behavior of the vertical truss is similar to a large cantilever beam, with the windward columns forming the tension flange and the leeward columns acting as the compression flange (Fig. 2.4b).

2.3 Combination Rigid Frame and Vertical Truss

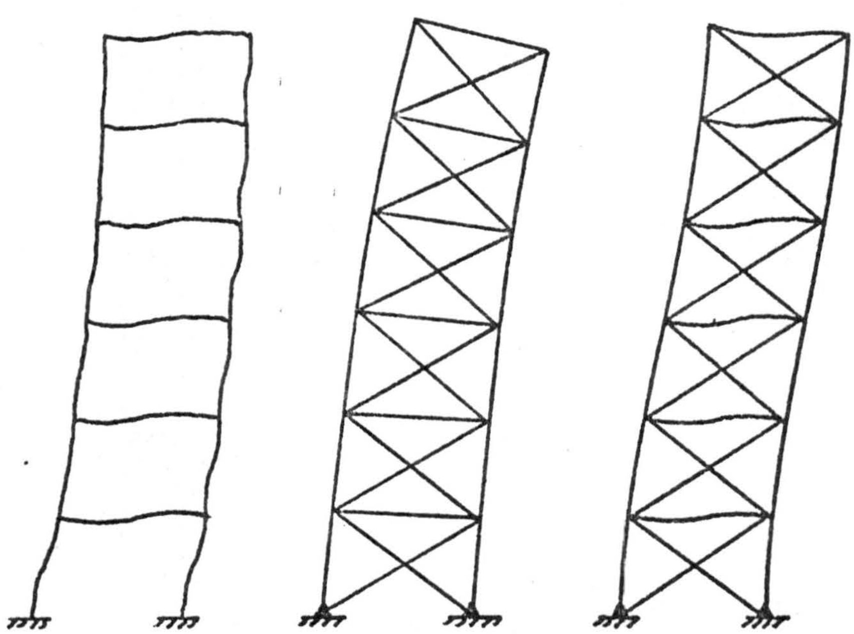
In the combination frames, pinned bracing is placed within a rigid frame. Lateral loads are carried by the combined action of the rigid frame and the vertical truss. Reduced beam sizes are possible, both because the bracing carries most of the lateral load and thus reduces the wind-caused moments in the frame, and because the rigid connections and resulting end moments allow the beams to be proportioned



(a) (b)

(c) (d) (e)

Fig. 2.3 Some various forms of lateral bracing.



(a) (b) (c)

Fig. 2.4 Deflected shapes for a rigid frame (a), a vertical truss (b) and a combination (c).

for less than the simple beam moments. However, the use of rigid connections will increase the connection costs. If the bracing contributes a sizable proportion of the lateral stiffness, the columns can be designed assuming sidesway is prevented in the plane of the frame. If the allowable axial load is controlled by the slenderness ratio in the plane of the frame, increased allowable axial stresses can result. The in-plane effective length factors were assumed to be less than one for all columns in the combination frames.

Drift deflections are also reduced in the combination frames. The deflected shapes of the rigid frame and the vertical truss combine to form the S shape shown in Fig. 2.4c. The vertical truss supports the rigid frame near the base of the building, while the rigid frame tends to pull back the deflected tip of the vertical truss.

2.4 Other Systems

Several other major systems are available for carrying lateral loads. They include semi-rigid frames (AISC Type III connections), tubular configurations, and the inclusion of reinforced concrete shear walls. These systems were not studied for various reasons, most of which were associated with the limitations of the design program used and the scope of the investigation.

The Type III connections of a semi-rigid frame have enough rigidity to develop some member end moment, but not as much as a rigid connection can develop (4). These semi-rigid connections can be combined with bracing to produce an economical and reasonable design for some frames. However, limitations of the analysis portion of the design program prevented the consideration of these connections. The analysis routine utilizes a stiffness approach, and connections which are not either pinned or fixed cannot be handled by the program without extensive modification. It was also felt that this type of framing is not suitable for the particular frame and lateral loading considered in the design.

Tubular structures have closely spaced columns in the external walls of the structure connected by deep and very stiff spandrel beams. This causes the entire building to behave as a cantilevered tube, thus utilizing the steel framing very efficiently and providing high structural stiffness (14). Connecting beams between the columns are proportioned with sufficient stiffness to transfer enough flexural shear between the columns of the tube (which is pierced by window openings) to keep shear lag effects to an acceptable level.

Shear lag effects result when a shear deformation takes place between the fibers of a beam (i.e. the columns of a tubular structure). This causes the normal assumptions of flexural and axial behavior, namely that a concentric axial load causes equal strains in all member fibers and that strains caused by moments vary linearly with the distance from the neutral axis, to become inexact. The behavior of the tube resembles that of a beam possessing two properties, a low shear stiffness in the web and an effective width in the flanges which is less than the actual width. Shear lag effects in the tube increase as the connecting spandrel beams are less able to prevent displacement and rotation of the member ends. Because the spandrel beam is usually short and deep to minimize its deformations (sometimes with span to depth ratios of less than one), shear deformations as well as flexural deformations must be considered in analysis.

Variations of the basic tubular system include bundled tube and tube in a tube structures. The bundled tube configuration, for which the 1454 foot Sears Tower is the most notable example, uses the internal framing system like beam webs to help to reduce shear lag effects. The tube in a tube system, used mainly in tall reinforced concrete structures, includes the design of both the utility core and the outer columns for tubular action. A tubular system would probably provide a workable structure for the structure considered in this study.

The consideration of all three dimensions is important in a tubular structure, and extensive modification of the design program would be required to accommodate the third dimension. A three-dimensional system can be modeled in a two-dimensional analysis program (13), but shear lag effects can be accurately included only if shear deformations of the individual members are considered, a capability not currently included in program DESIGN1. A tubular structure would also raise additional problems with the design program because of the increased number of joints and members and the orientation of corner columns.

The use of shear walls is another possible method of bracing moderately sized buildings. Basically, a concrete shear wall is assumed to resist all or a portion of the lateral loads, while the steel frame carries any gravity loads the shear wall does not. Shear walls are usually placed around the service areas of the structure. Methods exist for the computer modeling of shear wall and steel frame interaction (18). However, the design of shear walls is well beyond the scope of the design program and thus these types of structures are not considered in this paper.

2.5 The Basic Frame

To compare the relative efficiencies of the rigid frame, vertical truss and combination bracing systems, a basic frame was first chosen. The selected geometry is shown in Fig. 2.5. This frame represents an interior bent of a building with a plan length large relative to the width. Wind forces are assumed to be resisted by several bents extending across the narrow dimension of the building rather than by lateral load resisting elements placed at the ends of the structure and connected to the interior bents by adequate floor diaphragms. The dimensions selected reflect compromises between various limitations. A height tall enough to produce a significant wind load was desired, but design time and computer costs favored as small a structure as possible. The height of twenty stores at twelve feet each (except for the fourteen foot first floor) provided a reasonable compromise between these limitations. The effect of the lateral loading was increased as a result of two other assumptions: a relatively small bay width (25 feet) and a location in a high wind area; e.g., Miami, Florida. The three evenly spaced bays provided two desirable properties: symmetry about the centerline of the frame (bracing was always placed in the center bay) and a configuration which was easily adaptable to all the proposed bracing systems. The interior and exterior spans were selected to be of equal length, although for many buildings architectural requirements would result in shorter interior spans. The frame was assumed to have an effective width of twenty feet. Alternatively, the frame may be thought to represent the conditions in a building placed in a lower wind area and with the frame resisting the lateral forces from more than one bay width.

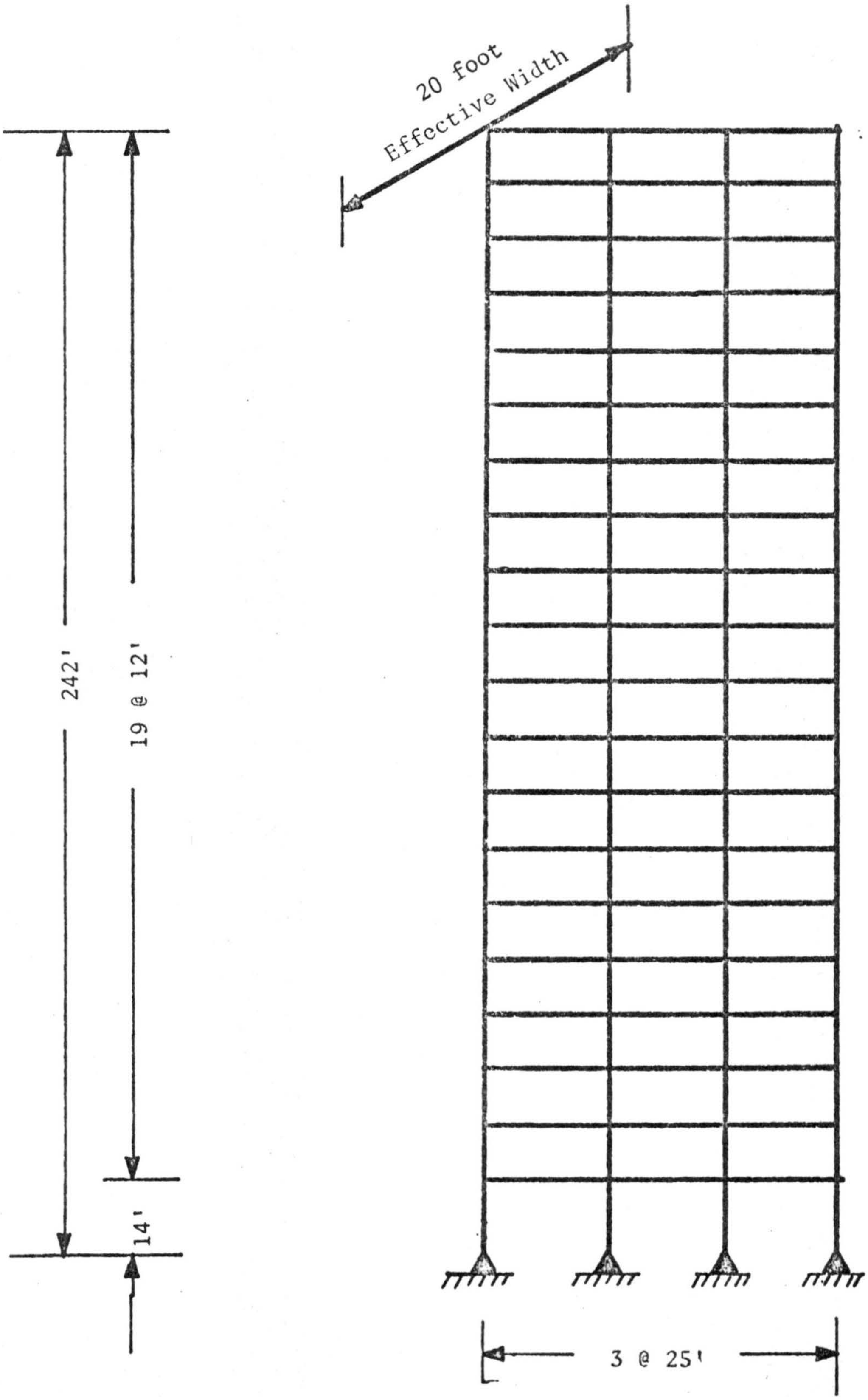


Fig. 2.5 The basic frame.

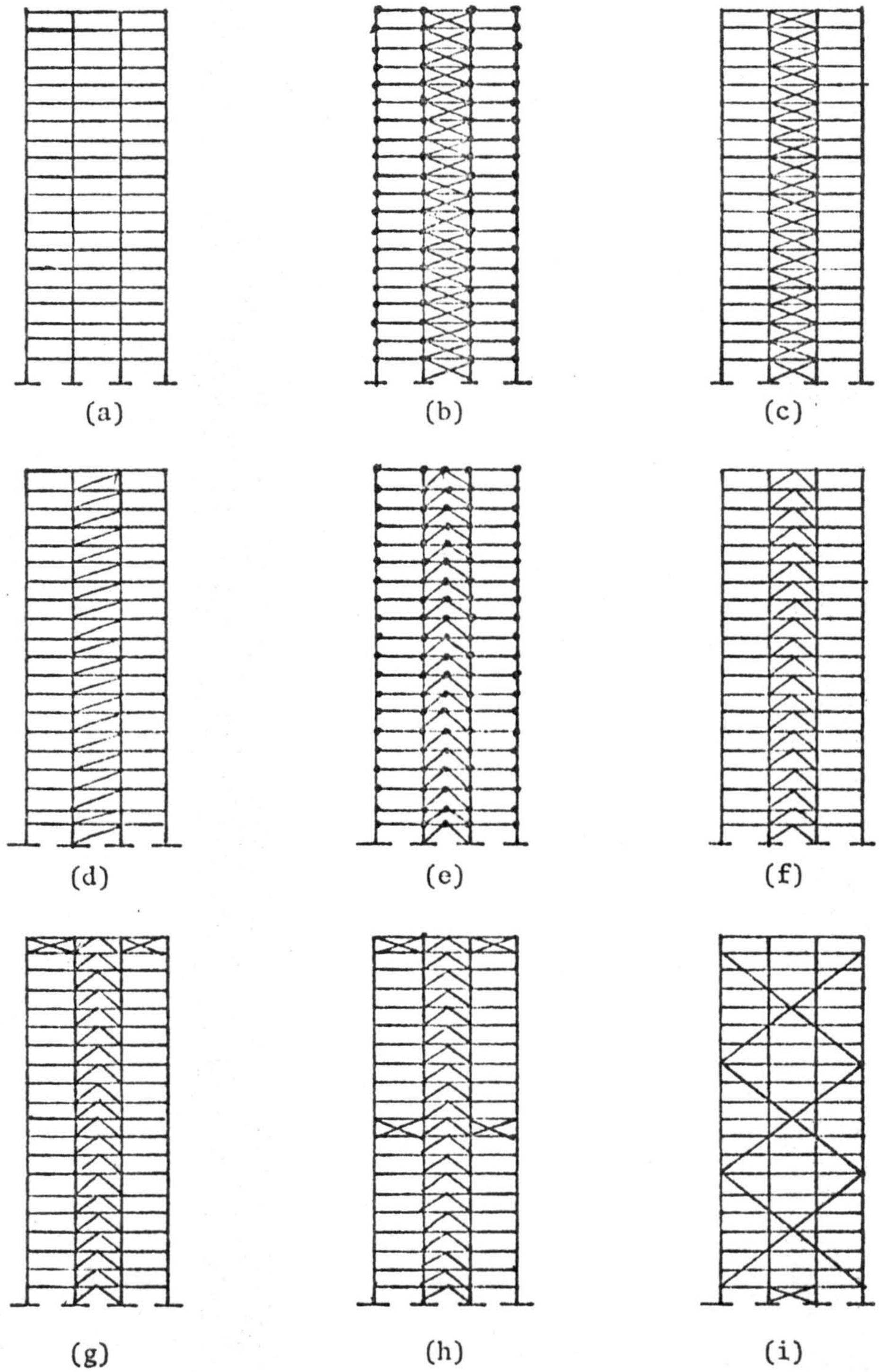
2.6 The Design Frames

The various frame configurations used are illustrated in Fig. 2.6. The rigid frame is shown in Fig. 2.6a. X bracing of the center bay was used in three frames, as a vertical truss in a pinned frame (Fig. 2.6b), in a rigid frame (Fig. 2.6c), and for a counter system in a rigid frame (Fig. 2.6d). K bracing was used in a vertical truss (Fig. 2.6e) and in a combination rigid frame-vertical truss system (Fig. 2.6f). These systems have been discussed in sections 2.1, 2.2, and 2.3.

After these six frames had been designed, three variations of the combined systems were tried, two designs with outrigger trusses and a design using multistory X bracing (also called inclined columns). All three systems were used in combination with a rigid frame.

The outrigger trusses consisted of extending the bracing into the external bays on the 20th floor (Fig. 2.6g) and on the 10th and 20th floors (Fig. 2.6h). The purpose of these trusses is to further engage the outer columns as a part of the vertical truss and thus to increase the stiffness of the frame. K bracing was chosen for the internal bay because it proved stiffer than the X bracing in the original six frames. The trusses in the outer bays were chosen to be of an X configuration to avoid introducing additional joints at the beam midspan in some outer bay locations.

The multistory X system (Fig. 2.6i) was chosen as another method of engaging the entire structure to resist the lateral loads. The first floor is braced by a simple X system in the center bay. The next eighteen floors are braced by three large X braces, each extending six stories high and covering the width of the building. The top floor is an unbraced rigid frame. Each bracing member in the multistory X



(a) (b) (c)

(d) (e) (f)

(g) (h) (i)

Fig. 2.6 The design frames.

system is assumed to be pin ended and to have no joint where it crosses a beam. However, they are assumed to be laterally braced at the level of each floor.

Chapter 3

DESIGN OF THE FRAMES

The design of the frames shown in Fig. 2.6 was accomplished by extensive use of program DESIGN1, which is described in Appendix A. Throughout the designs many assumptions had to be made. These assumptions were applied consistently to all nine frames.

3.1 Assumptions

Two types of assumptions were necessary. The first type occurred within the design program. These include such items as the idealized moment-rotation relationship for AISC Type I and Type II connections and assumptions about some of the design variables. These assumptions are listed and discussed in Appendix A.

The second type of assumptions concern loadings, deflections and other external limitations. Live load reductions, allowable in most building codes, were not made. Although it would have been possible to use the reductions, provisions for live load reductions vary between the major building codes and the same provisions are often interpreted in different ways. For simplicity and to be consistent for all frame configurations, live load reductions were not made for any of the frames. Limitations were placed upon both the vertical deflections of the beams and on the wind drift of the structure. Beam deflections were not permitted to exceed more than the span divided by 360. The frame was stiffened if the wind drift exceeded the height divided by 500 under the design wind loads. Individual story to story drift limitations were not considered directly. These deflection limitation values were chosen because they are typical for the type of frame considered in this study. A nominal depth limitation was placed on the beams and columns. Beam depth could not exceed 30 inches, except on the first floor. Columns were held to a nominal depth of 14 inches or less. The yield point of the structural steel was always assumed to be 36 kips per square inch.

3.2 Loading

Typical design loadings were selected. A dead plus live gravity load of 80 pounds per square foot of floor area was assumed, producing a 0.13333 kips per inch uniform loading on all beams. Member weights were included automatically by the design program. Wind loads were taken from the specifications for Miami, Florida (3). The wind loading is shown in Fig. 3.1. All loadings were reduced by one-fourth for design using combined wind and gravity loads.

3.3 Description of Design

The design of a frame by program DESIGN1 is done in three steps; wind, gravity and drift. A flow chart of these steps is given in Fig. 3.2. This flow chart is general, and was not always followed exactly. The wind loading controlled most of the member sizes in every design. Because of the unsymmetrical nature of the wind loading, many equivalences (see Appendix A.2.2, set 5b) had to be declared. The availability of the equivalence statement allows the program user to recognize symmetry between two members by forcing the size of the less critically loaded member to be equal to the more critically loaded member. Gravity controlled a few member sizes in the upper stories of most frames, and about one-third of the member sizes in the vertical trusses. Generally, just a check (not a complete design) of the gravity design was more efficient for most of the frames. If a member was overstressed, it was adjusted and checked again. The need to increase

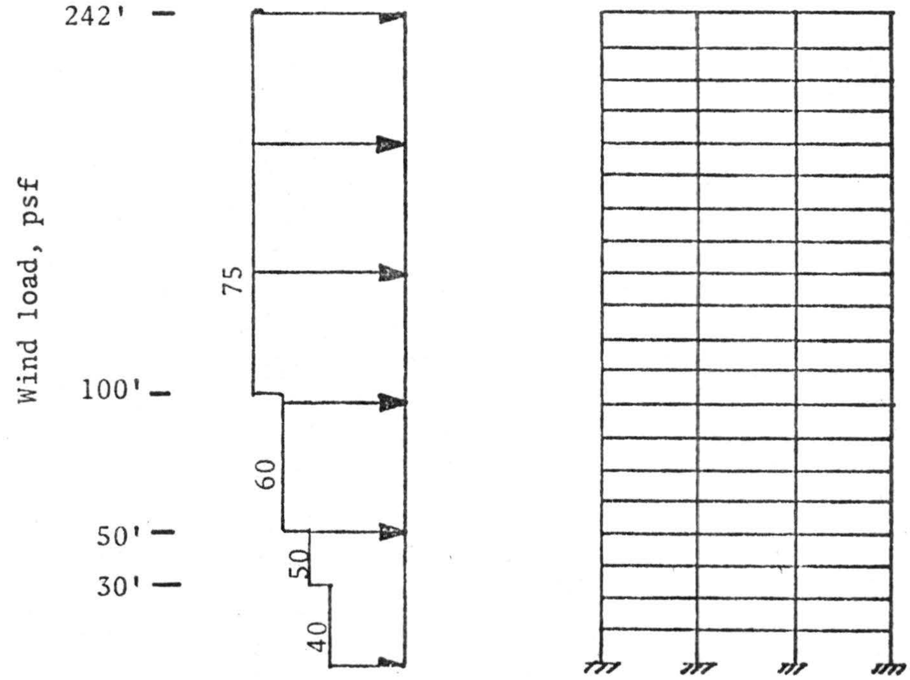


Fig. 3.1 Design frame wind load

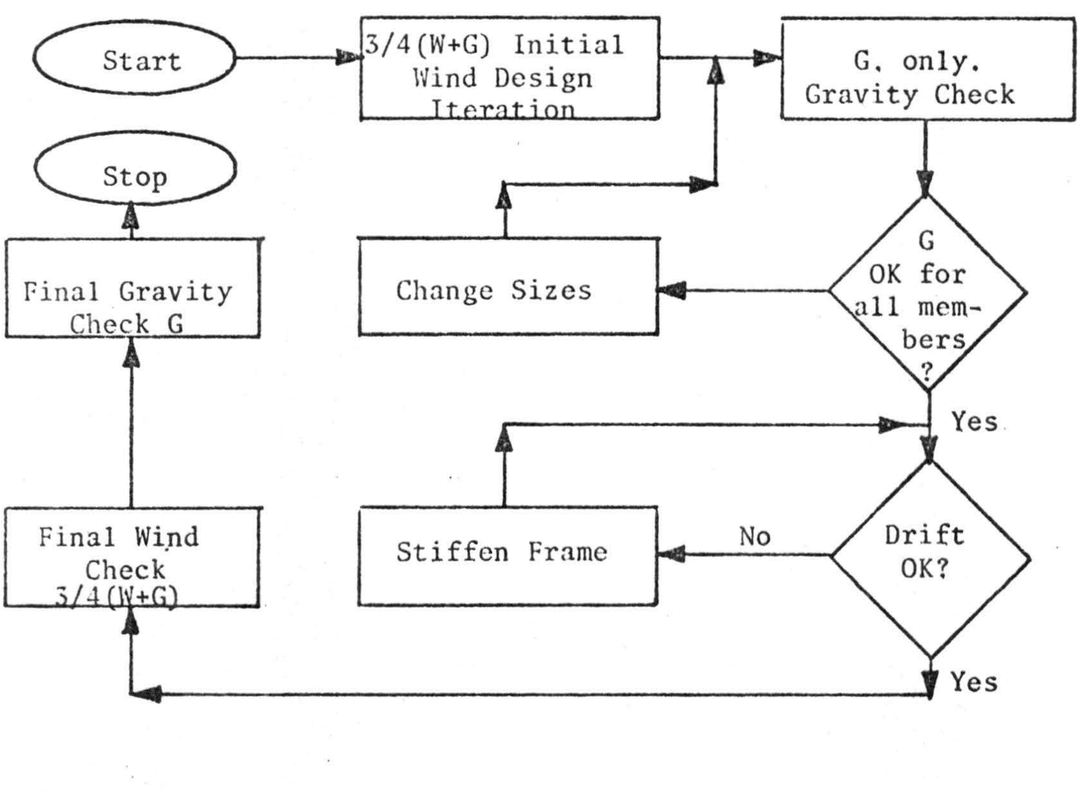


Fig. 3.2 General flow chart for frame design

the stiffness of members to reduce drift controlled a number of the member sizes in some frames. The techniques used to stiffen the frames are described later.

The computer used to run the design program was the CDC 6400 available at Colorado State University. The program required a field length of 110000_8 to compile and load and a field length of 71500_8 during execution. Execution run times for the initial free iteration on the wind loading case varied between 65 and 110 seconds, depending upon the number of members and the number of equivalences declared. Commercial rate costs at CSU are about $15.00 per 100 seconds for the field length specified above. Runs to check the adequacy of members for the gravity loading and to check drift deflections averaged about 15 seconds (execution) at the same field length. Compilation time for program DESIGN1 was about 28 seconds using FTN(OPT=2), the optimizing mode of the compiler.

3.4 Comparison of Designs

To compare the results for the nine frames, a measure of their relative efficiencies must be adopted. Cost is certainly the measure with the widest practical importance. Unfortunately, cost is dependent upon many factors which are variable in nature and can be very difficult to quantify, such as connection type and complexity and costs of labor, materials and construction time. Structural steel weight is used as the measure of merit because it is easily quantified. Weight accurately measures only the relative efficiency of the structural steel in the frame. Costs will have to be discussed intuitively.

The degree to which each design frame approaches its optimum design can affect the evaluation of the relative efficiencies of the various frames. Each frame has an optimum set of members, all of which are adequate under all loading conditions, which form the lightest structure. The design program does not have the sophisticated optimization routines which would be required to find the optimum design. Instead, it simply designs a structure in which each member is fully stressed under one of the loading conditions. These fully stressed designs do not guarantee the optimum design has been reached. However, the difference between the best fully stressed design and the optimum design is often quite small for most well behaved problems (9). It is possible for one frame to be closer to its optimum than is another. This certainly introduces some inaccuracies, but if the frame weights differ significantly, this inaccuracy will not affect the conclusions appreciably.

Chapter 4

RESULTS AND DISCUSSION

Nine frames were designed using the design program described in Appendix A, with the loading, geometry and limitations specified in Chapters 2 and 3. This section presents the results of these frame designs and discusses the behavior of each system, some comparisons between systems, the performance of the bracing during gravity loadings, and methods of stiffening the frames to reduce wind drift.

Table 4.1 shows the results of the frame designs. Two sets of results are shown for each frame, the initial wind plus gravity design (with a 1/4 loading reduction) and the final design. The wind drift deflection and the frame weight is first given for the initial wind design, in which gravity load (no wind) stresses and deflection limitations were ignored. The final designs satisfy all the requirements for stresses and deflections for both loading conditions.

A gravity-only design of the basic frame was done to provide some idea of the wind premium (i.e. the additional material required to satisfy lateral loading stresses) required by the various design frames. For the gravity frame the following assumptions were made: no wind loading, a no sidesway mode of failure ($K \leq 1.0$), rigid connections and zero weight bracing members. The weight of the gravity frame is used as the basis for the Relative Weight column of Table 4.1. It should be noted that this column does not reflect the relative weights of the structural steel for the entire building with and without the effect on lateral loadings. The final design weight contains only the weight of the structural steel in the frame itself, and does not include

Table 4.1 Summary of Design Frame Results

Description	Frame Type	Bracing	Initial Design to satisfy 3/4 (W+G) Stresses			Addition-al steel tons	Final Design			Relative Weight
			Drift inches	Δ/h	Weight tons		Drift inches	Δ/h	Weight tons	
Rigid Frame	Rigid Frame	-	12.7	229	108.7	118.8	5.78	503	227.5	378
Vertical truss x	Vert. Truss	X	11.5	252	103.4	67.3	5.72	509	170.8	284
Vertical truss K	Vert. Truss	K	11.9	244	85.5	72.0	5.91	493	157.5	262
Counters	Braced R.F.	X Count.	9.37	310	95.2	60.7	5.84	498	155.8	260
X Bracing	Braced R.F.	X	7.33	396	103.0	40.0	5.74	507	143.0	238
K Bracing	Braced R.F.	K	7.31	398	87.4	20.5	5.89	494	107.8	179
Outrigger Floor 20	Braced R.F.	X & K	6.75	431	82.8	14.8	5.80	502	99.5	166
Outrigger Floor 10 & 20	Braced R.F.	X & K	6.43	452	84.3	10.2	5.76	505	94.5	157
Multistory X	Braced R.F.	X	3.12	932	70.6	8.7	3.12	932	79.3	132
Gravity Frame	Braced R.F.	-	Gravity only design, no wind loading, no bracing weight.						60.0	100

the weight of out-of-plane girders, spandrels and joists. The addition of these out-of-plane weights to both the final designs and the gravity only design would increase all frame weights, but would probably decrease the frame weights relative to the gravity frame weight.

4.1 Rigid Frame

The final design for the rigid frame was controlled by the drift limitation, which was chosen as height/500. Most beam and column sections had to be chosen from the highest architecturally allowable nominal depths to provide the flexural stiffness necessary to supply adequate resistance to the wind loading. Fig. C.1 (in Appendix C) shows the final member sizes for the rigid frame. Typically, members were only stressed to about 50 percent of their allowable stresses in either the wind (3/4(D + L + W)) or the gravity (D + L) loading cases, thus resulting in a very heavy frame with a total weight of 227 tons.

The wind loading caused two types of actions which had to be carried by the frame, a shear and an overturning moment. The wind shear was carried as column shears in the rigid frame. The overturning moment was resisted by a combination of column moments and column axial loads (Fig. 4.1). The contribution of the column moments to this resistance varies with the height of the structure. At the 16th story, column moments resisted 18 percent of the overturning moment, but this figure had dropped to only 5 percent at the 2nd story. Below the 16th story these moments became quite significant in column design, typically causing the flexural term in the interaction equation to be about equal to the axial term.

The column axial loads provided most of the resistance to the overturning wind moment. Representative column axial loads are shown in Fig. 4.2. The transfer of axial loads from the windward columns to the leeward columns is accomplished by the flexural action of the beams (Fig. 4.3). This allows the axial loads of the leeward columns to increase, while the axial loads in the windward columns decrease. This creates a couple which resists the overturning wind moment. Three separate analyses were performed for a wind loading on the rigid frame to test the accuracy of the portal and the cantilever approximate analysis methods (17). A stiffness analysis similar to the analysis routine used in the design program was used to determine the "exact" solution. Member properties for the stiffness analysis were taken from the final design members of the rigid frame. The assumption of inflection points at the midpoints of beams and columns was made for the approximate methods. For the portal method, the four column shears were assumed to be distributed as 1/6, 1/3, 1/3 and 1/6 of the total lateral load for the floor. Equal column areas were assumed for the cantilever method.

The results of the analyses are shown in Table 4.2. Although the assumptions made for the approximate methods were crude, the cantilever and portal methods both produced results which were reasonably close to the stiffness solution. The application of these two approximate methods for a quick preliminary analysis appears to be reasonable for a frame with similar geometry.

4.2 Vertical Trusses

The two vertical trusses were designed next. The critical loading case for a member was either wind stress, gravity stress, or drift deflection, depending upon the member's position in the truss (Fig. 4.4). The X and K bracing performed in a very similar manner. However, the

Table 4.2 A comparison of the portal, cantilever and stiffness analysis methods for a wind (W only) loading on the rigid frame.

Columns

Story, Bay	Moment			Shear			Axial		
	P	C	S	P	C	S	P	C	S
12 I	2754	2863	2620	38.3	39.7	36.6	0.0	23.6	24.1
12 E	1377	1280	1490	19.1	17.8	20.8	78.5	70.7	70.3
8 I	4051	4138	3870	56.0	57.4	53.8	0	50.7	33.8
8 E	2025	1835	2140	28.1	26.4	29.9	169.0	152.0	158.0
4 I	5112	5202	4810	70.5	72.0	66.9	0	87.0	30.6
4 E	2556	2285	2730	35.0	31.7	38.0	290.0	261.0	280.0

Beams

Story, Bay	Moment			Shear		
	P	C	S	P	C	S
12 I	2592	3030	2900	17.3	20.2	19.3
12 E	2592	2340	2760*	17.3	15.6	16.2
8 I	3872	4635	4040	25.8	30.9	26.9
8 E	3872	3480	4080*	25.8	23.2	25.3
4 I	4903	5894	4750	32.7	39.3	31.7
4 E	4903	4425	5300*	32.7	29.5	33.3

P = Portal Method

C = Cantilever Method

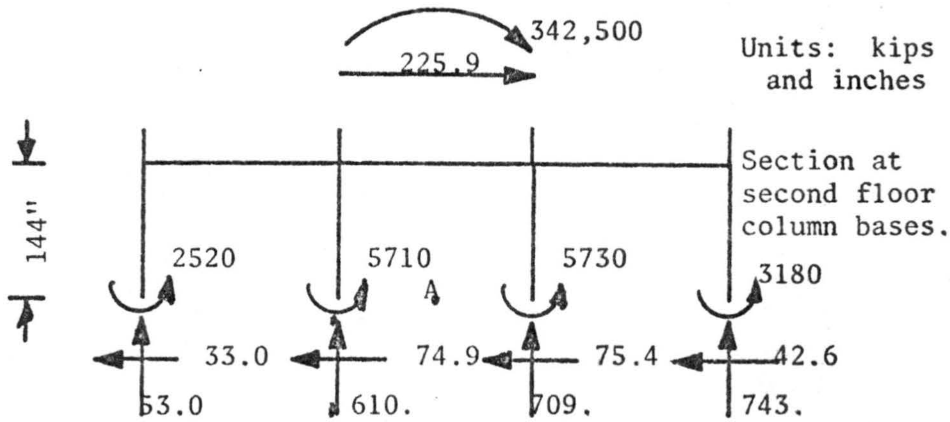
S = Stiffness Method

E = Exterior Bay

I = Interior Bay

All units are kips and inches.

* Maximum end moment for beam.



Column Moments = 2520 + 5710 + 5730 + 3180 = 17150} 5%

Interior Column couple about A = (709 - 610) 150 = 14,850

Exterior Column couple about A = (743 - 53) 450 = 310,500

} 95%

TOTAL = 342,500

Fig. 4.1 Column axial and moment actions at the second floor level of the rigid frame, 3/4(W+G) loading.

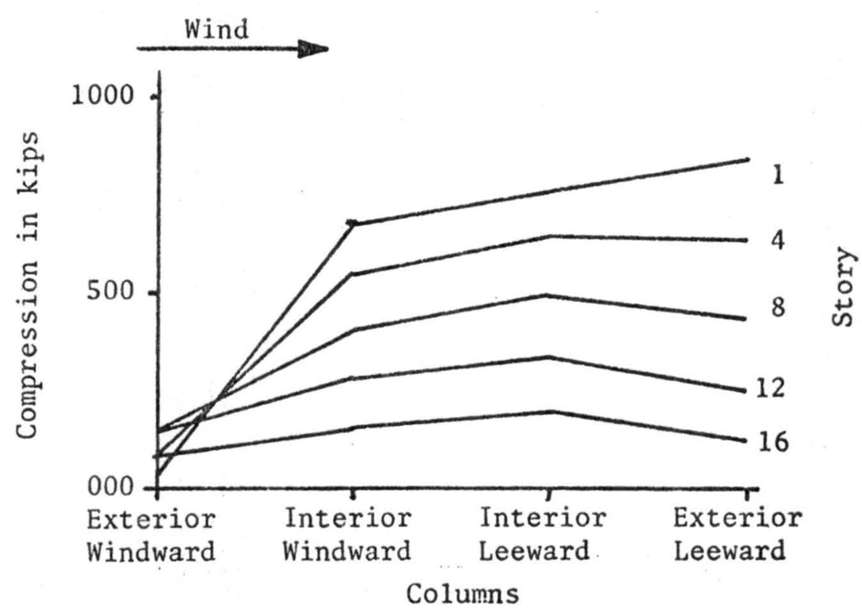


Fig. 4.2 Column axial load distributions in the rigid frame, 3/4(W+G) loading

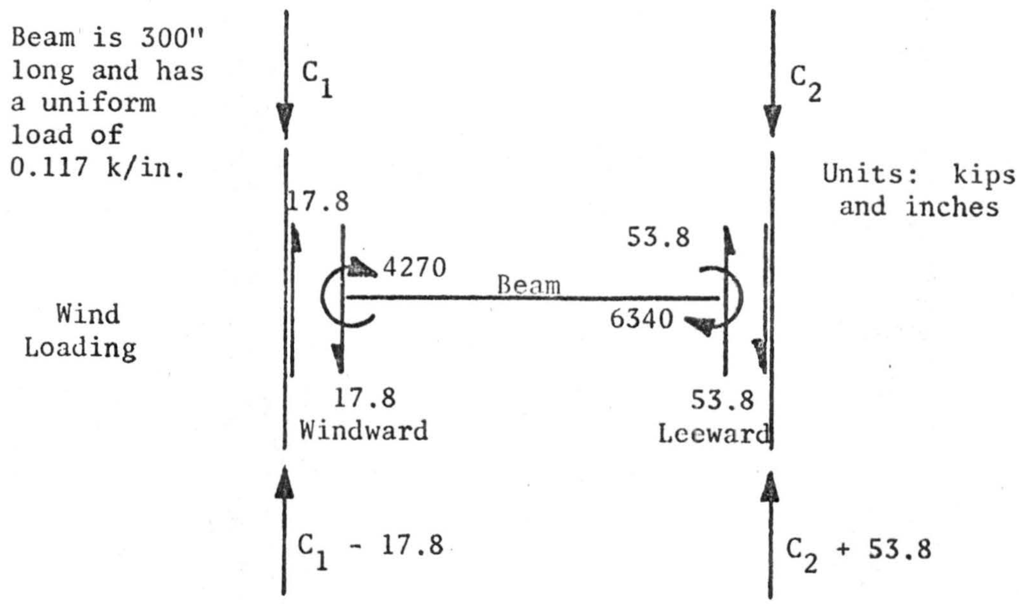
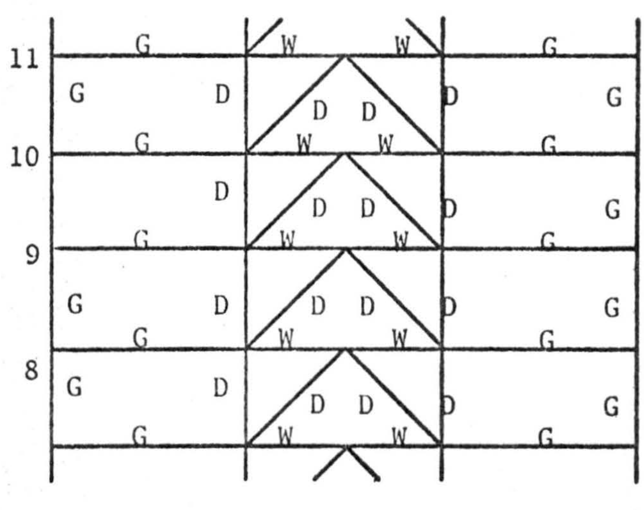


Fig. 4.3 Transfer of column axial load by beam action.



Governing Case: G = Gravity Stress
W = 3/4(W+G) Stress
D = Drift Deflection

Fig. 4.4 Governing loading case for a typical section of the K braced vertical truss.

longer lengths of the X bracing had two adverse effects. First, a lower allowable axial stress resulted and the axial stiffness (AE/L) was decreased. Second, the bracing in the upper five stories of the X braced vertical truss (Fig. C.2 in Appendix C) was controlled by the slenderness ratio limitation of 200 for compression members. These two factors combined to produce the difference in final weights between the X braced and the K braced vertical trusses.

The vertical truss carried the lateral loads entirely by the action of the vertical truss formed by the bracing, beam and column members in the center bay. The wind shear was carried by the diagonal bracing members and the floor members contained within the truss. The computed axial stress in these interior beams was typically about 60 percent of the allowable axial stress at the bottom of the truss and decreased to about 35 percent of the allowable axial stress at the top of the truss. The overturning wind moment was resisted primarily by the couple formed by the axial loads in the two interior columns (see Fig. 4.5 and 4.6). This couple accounts for 97.6 percent of the resisting moment at the second floor in the K braced truss. The remaining 2.4 percent is supplied by the couple formed by the vertical force components of the bracing axial loadings. The numbers are similar for the X-braced vertical truss.

4.3 The X and K Combinations

The next three design frames were combinations of the rigid frame and the X or K bracing. Two of the frames used X bracing, one with the bracing acting in tension only (counters) and one with the bracing acting in both compression and tension. One K braced rigid frame was designed. All three frames were braced only in the center bay. Member

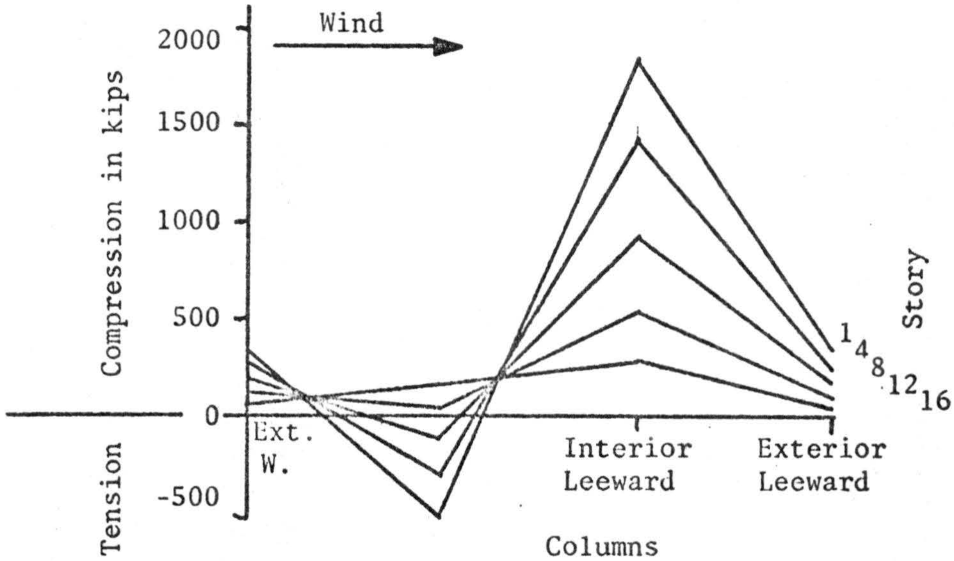
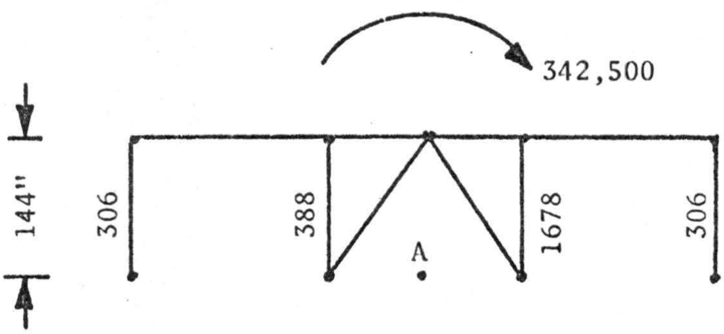


Fig. 4.5 Column axial load distributions in the K braced vertical truss, 3/4 (W+G) loading.



Units: kips and inches

Section at second floor column bases.

Positive = Compression

Internal column couple about A = (1678 + 388) 150 = 309,500

Bracing couple about A = (147 + 171) (.6925) 150 = 33,000

Total = 342,500

Fig. 4.6 Column and bracing axial actions of the K braced vertical truss, 3/4 (W+G) loading.

sizes were determined by stresses and by needed stiffness to meet drift limitations. However, drift deflections were not as critical as those in the rigid frame and vertical trusses.

Wind loading stresses tended to govern the external column sizes in all three frames. These larger stresses resulted primarily from the transfer of the interior column axial loads to the exterior columns by the rigid frame beam mechanism already mentioned (Fig. 4.3). Column moments did increase column sizes somewhat, but this effect was greatly reduced over that of the rigid frame. The contribution of the flexural term of the interaction equation increased in moving up the frame. The flexural term values varied from 0.03 to 0.25 (going from bottom to top) in the interior columns and from 0.10 to 0.50 in the exterior columns.

The beam sizes in the counter X and X design frames were largely determined by the need to stiffen the frames to reduce drift deflections. Stiffening of frames to reduce wind drift is discussed in Section 4.7. This stiffening resulted in beams which were usually understressed. Typical values of the interaction equations at the 10th floor of the structure were about 0.85 for the K bracing, 0.70 for the X bracing and 0.60 for the counters. An exception occurred in the exterior span beams in the mid and upper stories of the K braced frame. These members were usually fully stressed under both the gravity and the wind loading cases. (Members are considered fully stressed when the interaction equation value exceeds 0.90). This occurred because the frame did not require increases in the sizes of these beams to meet the drift limitations. The interior beams of the counter bracing system were significantly larger than the external beams in the lower stories. This resulted from the increase in axial load in the beam which was required to transfer the lateral load between the tension only bracing on successive floors.

The K braced frame had the least weight of the three combination frames, weighing 25 percent less than the X braced frame and 32 percent less than the counter-X braced frame. This can be attributed to the same advantages the K bracing had in the vertical truss, first, higher allowable axial loads and increased axial stiffness of the bracing members because of their reduced length, and second, the reduction in interior beam sizes allowed because of the K bracings' support at their midspan. Although slenderness limitations could be ignored in the counter bracing design, the loss of the compression diagonals' stiffness required a significantly larger amount of steel to stiffen the frame.

The wind loads were carried by the combined action of the rigid frame and the bracing. Nearly all of the horizontal shear was carried by the bracing in all three frames. This was true at any height in the structure. The remaining horizontal shear was carried by the columns. The overturning moment was resisted primarily by the column axial loads (Fig. 4.7). The rigid frame action transferred some of the axial load from the windward exterior columns towards the leeward exterior columns, allowing the external columns to form a couple which has a greater lever arm (450 inches) than the two interior columns (150 inches). Because of their greater lever arm, the exterior columns resist the overturning moment much more efficiently. In the second story of the K braced rigid frame the exterior column couple resisted 36 percent of the overturning moment while the interior column couple, the bracing and the column moments resist 55 percent, 8.5 percent and 0.5 percent of the overturning moment respectively.

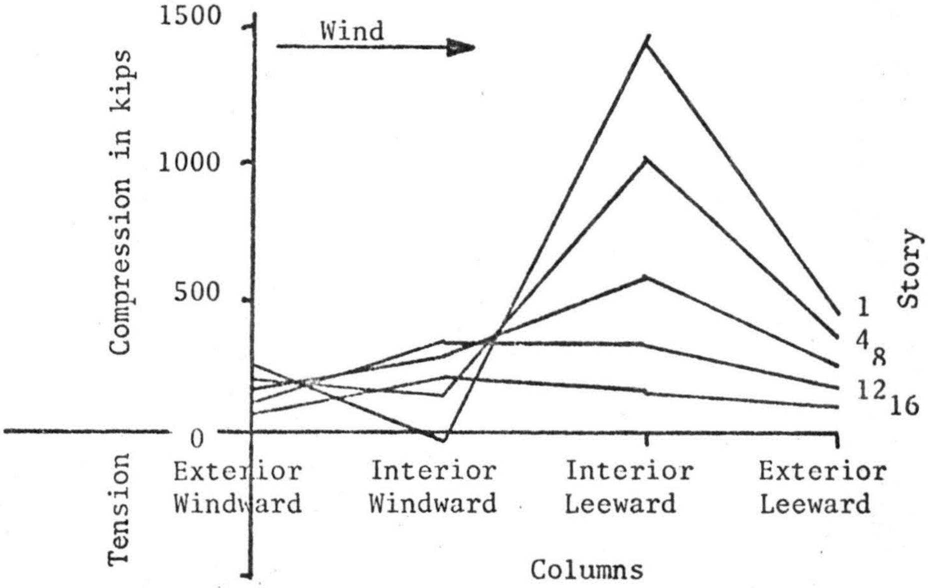


Fig. 4.7 Column axial load distributions in the K braced combination frame, 3/4 (W+G) loading.

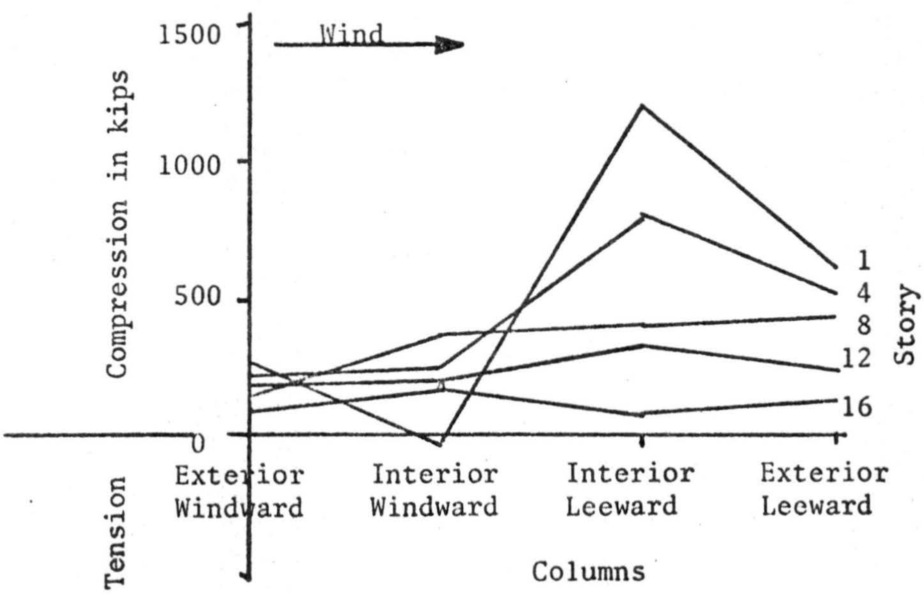


Fig. 4.8 Column axial load distributions in the outrigger truss bracing system, floors 10 and 20 fully braced, 3/4(W+G) loading.

Thus, the bracing and the rigid frame complement each other to yield a lighter frame than with either the vertical truss or the rigid frame alone. The rigid frame action engages the outer columns to resist the wind-induced overturning moment while the bracing acts to reduce the column moments.

4.4 Three Other Combinations

From the initial designs it became obvious that drift deflection limitations were causing a significant increase in the weights of the frames. Two types of bracing systems were proposed to stiffen the frame by increasing the proportion of the overturning wind moment resisted by the external columns. The first type included the use of an outrigger truss to brace the external bays of the K braced rigid frame at selected floors. The second type included bracing the entire frame with multi-story X bracing extending the full width of the frame.

Both of these types of systems attempt to make the frame more analogous to a cantilever beam. The bracing in the outer bays provides greater continuity between the internal and external columns. This results in a transfer of the wind overturning moment from the interior column couple to the exterior column couple. The frame is then similar to a cantilever beam: the bracing and internal framing system forms the web (and carries its share of the vertical load) and the external columns form the flanges and resist most of the overturning moment from the lateral loads. This type of behavior is especially obvious in the multistory X braced frame.

Both of the outrigger frames were stiff enough that drift deflection limitations did not greatly increase the total steel weight. Wind loading stresses generally governed the design of the bracing and of the exterior columns. Gravity loading stresses determined member sizing of the interior columns and at a few locations where beam sizes were increased slightly to meet L/360 vertical deflection limitations. Most beam sizes were either controlled by wind stresses or by wind drift stiffening requirements. However, only slight increases in beam sizes were required to stiffen the frame.

The effectiveness of the outrigger trusses varied greatly with the distance from these fully braced floors. Table 4.3 and Fig. 4.8 clearly illustrate that the fully braced floors resulted in a significant change in the axial stress distribution. However, the behavior of the frame gradually reverted back to the simple braced frame (Fig. 4.7) at levels further below the floor with the full width bracing. This accounts for the decrease in steel weight of the 10th and 20th story bracing system over just having the full width bracing at the 20th story only.

Drift deflection limitations did not affect any member sizes in the multistory X braced frame. Wind loading stresses generally controlled member sizing in the external columns, the bracing and a few beams in the exterior bays. Gravity determined the internal column sizes and most beam sizes. However, most members in the multistory X braced frame were highly stressed (interaction values of 0.75 or more) for both the gravity and wind loading conditions.

The ability of the multistory X bracing to move the axial wind loadings to the exterior leeward columns is clearly shown in Fig. 4.9 and Table 4.4. In every braced story, the external column couple resisted at least 60 percent of the wind's overturning moment, while the internal column couple provided only minimal resistance. The position of the bracing members within the individual floor has a very important effect on the response of the members in this floor. When the bracing is in the external bays, its lever arm builds up and the bracing is able to carry a significant portion of the wind moment (stories 2 and 9 in Table 4.4). When the bracing acts through the building's centerline, as in the fifth floor, the lever arm of the bracing is reduced to zero and the columns must resist the entire wind moment. The bracing couple can work with the wind moment (12th story) and cause an increase in external column axial loads. The wind shear is carried by the bracing.

4.5 Comparison of the Various Bracing Systems

Several different comparisons of the various frames can be noted. Quantitative comparisons of the relative steel weights follow from the final design weights shown in Table 4.1. It is also possible to make some approximate qualitative comparisons of relative costs. The performance of the different types of bracing and framing systems can also be discussed.

The rigid frame resists lateral loads entirely by using the flexural stiffness of the members, which was very inefficient for the 20 story frame considered. The rigid framing without bracing resulted in a greater increase in wind moments in the beams and columns of the lower stories than in any other frame. These large moments caused significant increases in the size and weight of these members. In addition, stiffening the rigid frame to reduce wind drift to acceptable limits required doubling the weight of the frame from the initial wind stress design. It is observed that the efficiency of the rigid frame varies directly with the magnitude of the moments produced by the lateral loading and the drift limitations. Thus, a rigid frame could

Table 4.3 Percentage of wind moment resisted in the outrigger truss bracing system with floors 10 and 20 fully braced.

Story	Exterior Column Couple	Interior Column Couple	Bracing Couple	Column Moments
12	58.7	22.5	17.1	1.7
11	53.8	30.0	11.7	4.5
10 fully br.	74.1	22.6	4.9	-1.6
9	88.0	0.0	10.0	2.0
8	81.8	4.3	12.8	1.1
5	61.8	27.2	10.2	0.8
2	48.7	42.0	8.6	0.7

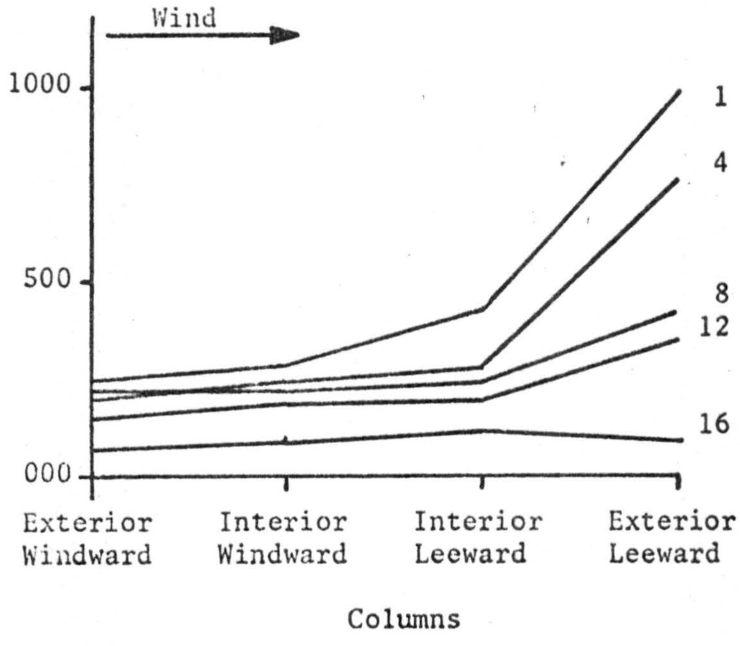


Fig. 4.9 Column axial load distribution in the multistory X bracing system, 3/4(W+G) loading.

provide a reasonable design in low frames or frames with light lateral loadings. However, the design frame selected was too high and the lateral loads too great for efficient use of the rigid frame system.

The results for the vertical truss and the combination frames provide some interesting comparisons. In each type of system the K bracing proved to be far superior to the X (and counter) bracing system. This was primarily due to the shorter lengths of the K bracing members which resulted in higher allowable axial loads and greater axial stiffnesses. The K bracing reduced the size of the internal bay beams by providing a support at the midspan of the beams (see Fig. C.4). Also, K bracing offers some architectural advantages, such as allowing larger openings in the braced bay.

The final design weights for the combination frames were about 2/3 to 3/4 of the design weights for comparable vertical trusses. Whether the combination system or the vertical truss system is better depends upon the relative economies of Type I (rigid connections) and Type II (pinned connections) framing. Type I framing is certainly more costly to fabricate and assemble than is Type II framing. A quantitative cost analysis would have to be performed to see if the reduction in steel cost for a combination frame would justify the expensive Type I connections.

The three designs with the least weight were the ones which engaged the outer columns in the vertical cantilever beam action most effectively. The two outrigger bracing systems and the multistory X bracing system were stiff enough to reduce or eliminate the inclusion of steel to satisfy drift deflection limitations. All three systems have architectural handicaps. The outrigger trusses block out a whole story or two in the frame, allowing only a few openings between the bracing. For this reason, outrigger trusses are often placed on mechanical floors, in end walls, or are placed along lines where interior walls are planned. This handicap is not as serious in the multistory X bracing, but it is still present.

The multistory X braced frame had the lightest steel weight by far. It was also extremely stiff. These attributes probably make it the most efficient frame designed. However, several improvements can be suggested. The stiffness of the multistory X braced frame could permit either of two choices: less expensive semi-rigid or pinned connections, or an increase in the lateral loads on the frame. Since the external column couple is formed more by the action of the bracing than by the action of the rigid frame (Table 4.4), a change of connection type which would reduce or eliminate the rigid frame action is feasible. It is economical to alternate the braced wind bent with one or more gravity bents in the structure, as shown in Fig. 4.10. This would increase the lateral loads on the wind frame and use the stiffness of the multistory X bracing more effectively.

4.6 Bracing and Gravity Loads

Bracing is often ignored when the beams and columns are sized for gravity loads. The bracing obviously does carry some of the gravity load, but this contribution is usually considered small enough to neglect. The design frames indicate that this is usually a reasonable assumption. Generally, bracing was stressed to only about 10 percent to 15 percent of its allowable stress by gravity loads in the vertical trusses, the X and K braced combinations and in the outrigger truss bracing system (see Fig. 4.11). The only exception in these frames

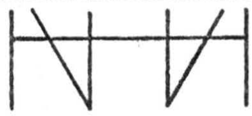
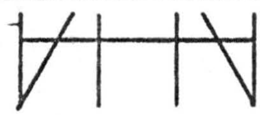
Story	Position of X bracing	Exterior column couple	Interior column couple	Bracing couple	Column moments
12		114.1	2.7	-18.5	1.7
9		63.9	3.4	32.7	0.0
5		96.0	4.0	0.0	0.0
2		69.5	2.3	28.0	0.2

Table 4.4 Percentage of wind moment carried in the multistory X braced frame.

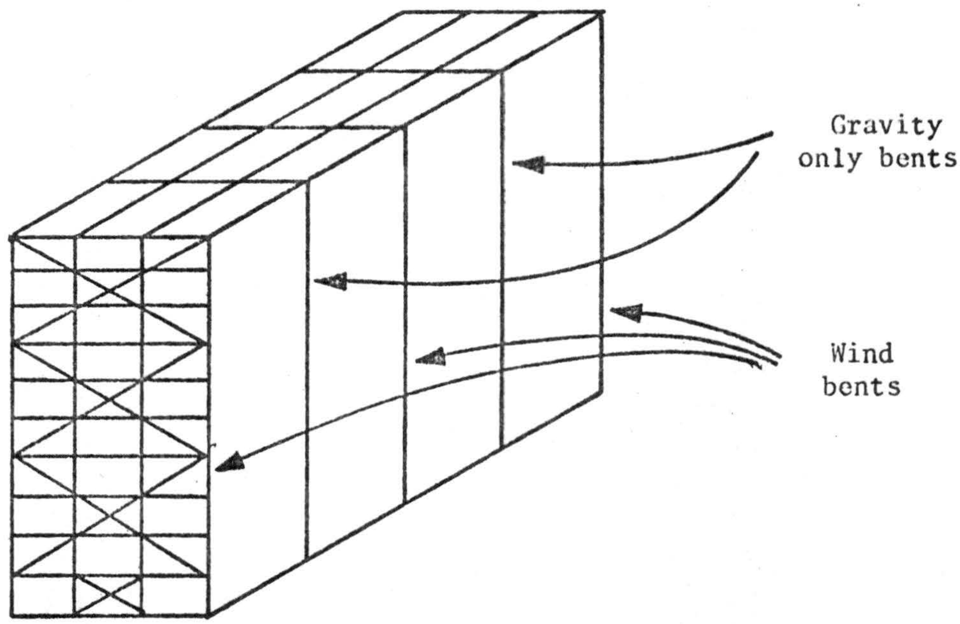


Fig. 4.10 Alternate wind and gravity frames in a structure.

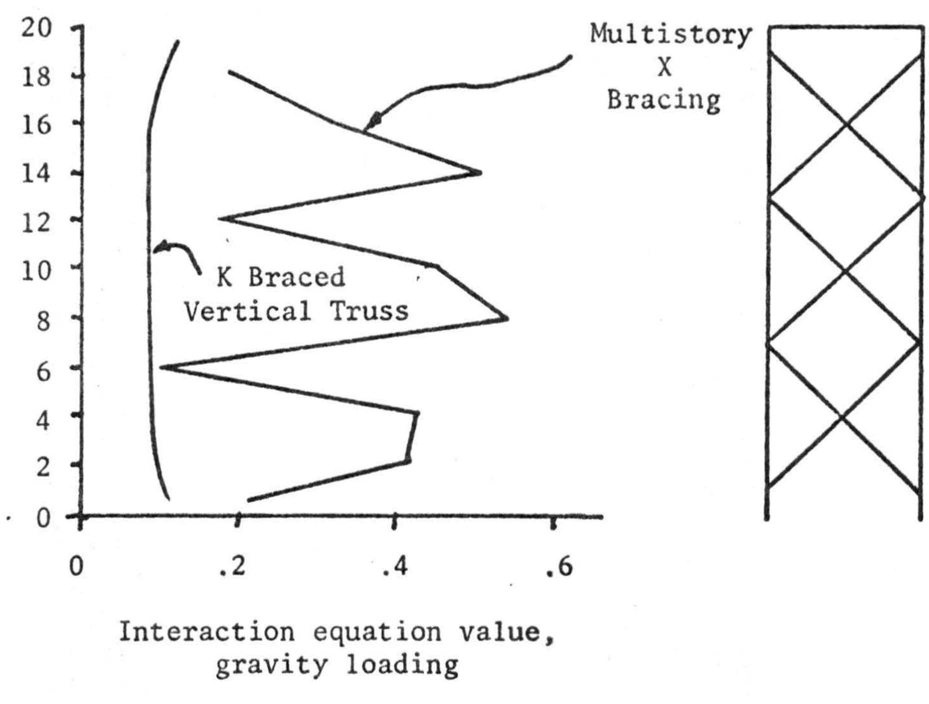


Fig. 4.11 Degree of stress in bracing during gravity only loading.

occurred in the 20th story of the outrigger truss systems. Bracing members in that position were stressed to about 50 percent of the allowable stress in the gravity loading. This occurred because the outrigger truss acted to equalize the axial shortening of the interior and exterior columns. The multistory X bracing did receive stresses up to about 50 percent of the allowable stress, but these stresses dropped off when the bracing was entering the exterior bay (stories 6, 12 and 18). With this bracing system the bracing picks up enough of the gravity load to be considered in analysis. Partly to emphasize this behavior, the bracing elements of the multistory X bracing system are sometimes designated as inclined columns. Thus, it would appear to be reasonable to neglect the contribution of the bracing during a gravity loading in all the types of systems considered except in the multistory X bracing system. However, the increasing availability of computer-aided analysis methods makes feasible the inclusion of gravity load actions in the bracing member design and analysis.

4.7 Stiffening Frames Against Drift

Every frame designed, with the exception of the multistory X braced frame, required some stiffening to satisfy drift limitations. In most systems, additional steel was placed in the frames by intuition to stiffen the structure. However, two frames were investigated to determine.the optimum locations for the additional steel.

The rigid frame required 110 percent of the initial wind stress design weight to reduce the drift deflections to the h/500 limitation. The majority of the beams were increased to the nominal depth limitations. Column sizes were also increased greatly.

For two of the bracing systems designed, an investigation was made of the best placement of steel to stiffen a vertical truss. An X braced cantilever truss with geometry similar to the interior bay of the X braced vertical truss was used to determine the effects of increasing the cross sectional areas of the bracing, columns and beams. An initial computer run was made with the area for each member in the truss equal. Three additional runs were made, one each for increases in the beam, column or bracing member sizes. In each case the cross sectional area of beam, column or bracing was doubled while the other areas were held to their original values. The curves showing the results of these studies are given in Fig. 4.12. Clearly, in a vertical truss of this geometry, increasing the interior column sizes has the greatest effect on drift deflections. Increasing the bracing sizes has only a minor effect, while increases in beam sizes do not significantly decrease drift.

These results can be explained by examining the pinned truss deflection equation:

$$\Delta = \sum_{i=1}^{NM} \frac{n_i N_i L_i}{A_i E_i} \tag{4.1}$$

where Δ = deflection, A = area, L = length, E = modulus of elasticity and NM = number of members, N_i = member force due to real loading, n_i = member force due to the application of a unit load at the position and in the direction of the desired deflection. Table 4.5 shows the truss deflection contributions of the beam, bracing and columns of the truss used in Fig. 4.12. Because the columns, beams and bracing do not all have the same area, the $\Delta/\sum A_i L_i$ term is used to weigh the results.

Table 4.5 Contributions of beams, columns and bracing to vertical truss deflections.

Type	$\sum A_i L_i$	$\sum \frac{L_i}{A_i E_i}$	$\Delta = \sum \frac{p_i P_i L_i}{A_i E_1}$	$\Delta / \sum A_i L_i$
Columns	1.50×10^5	8.28×10^{-3}	14.952	99.6×10^{-6}
Beams	1.05×10^5	5.79×10^{-3}	0.001	-------
Bracing	2.52×10^5	13.8×10^{-3}	0.395	1.5×10^{-6}

Total Drift = 15.347

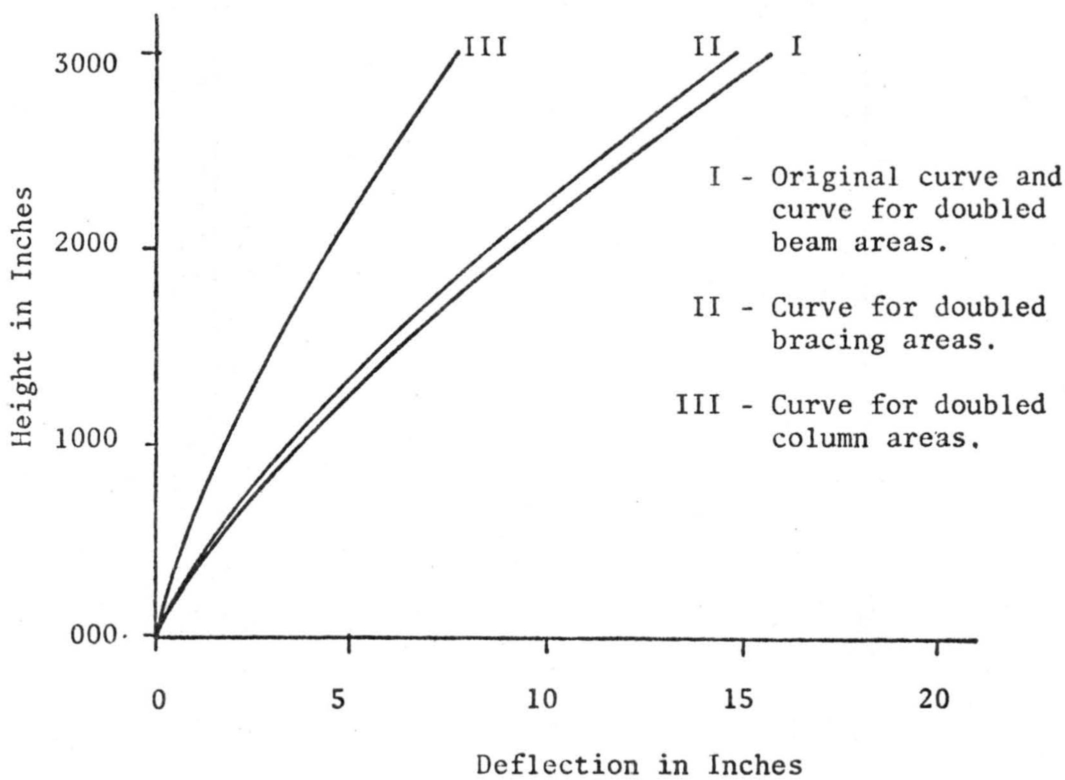


Fig. 4.12 Truss deflections for lateral loads, effects of doubling member areas of beams, bracing or columns while holding other areas constant.

These figures clearly illustrate that the axial shortening of the columns contribute most of the deflection. This makes them the most efficient place for additional stiffening steel.

Fig. 4.13 shows the stiffening history of the K braced vertical truss. Only the bracing and column sizes were increased. The additional stiffening steel was placed uniformly in the bracing or columns throughout the height of the frame. From the starting point to point 1, the column stiffening was about 5 times as efficient as was placing the additional steel in the bracing members. From points 1 to 3, only the column sizes were increased, but for each step the columns required more steel to cause a unit decrease in deflection. At point 3 an increase in bracing sizes was tried again. The bracing efficiency at point 3 was the same as it was at the starting point. However, an increase in column sizes showed that their efficiency was now equal to that for the bracing. An increase in bracing size was used to go from point 4 to point 5. Column increases brought the frame to the acceptable drift limit at point 6. This efficiency drop for column stiffening indicates that the best placement for stiffening steel may shift from the area which was predominant at the start of stiffening to a different area of the frame.

A more complete, although briefer, stiffening history was recorded for the K braced rigid frame with the 20th floor outer bays fully braced (Fig. 4.14). Increases in the inside column area and beam sizes had the greatest and almost equal efficiencies. Weight increases of the bracing and outside columns had significantly lower stiffening efficiencies. Increases in the inside column sizes were used to stiffen the frame and reduce the drift to point 1 in Fig. 4.14. Note that

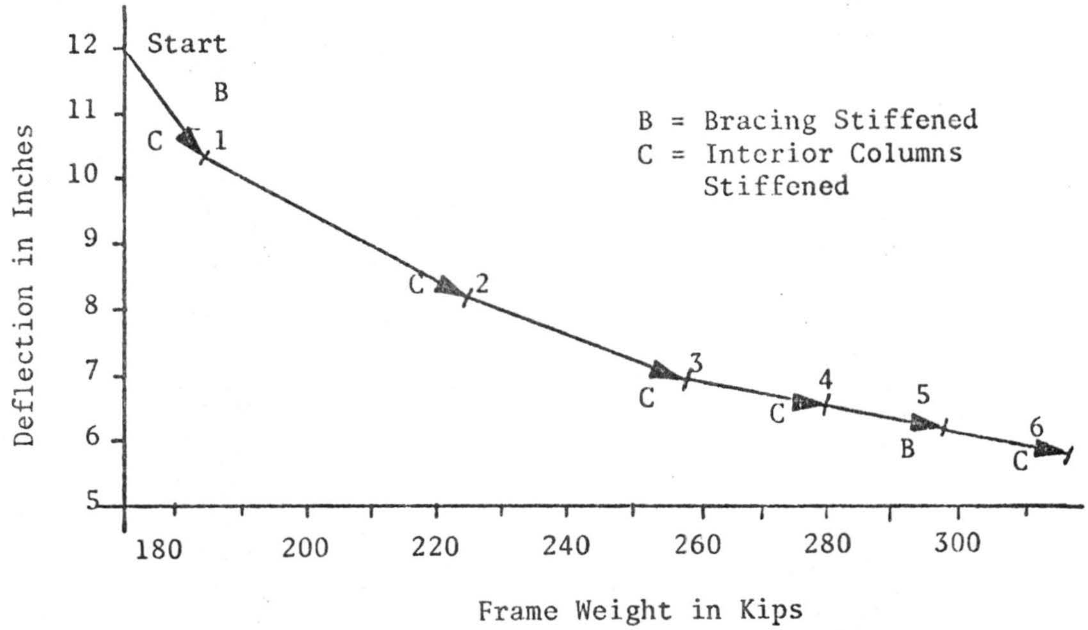


Fig. 4.13 Stiffening history of the K braced vertical truss.

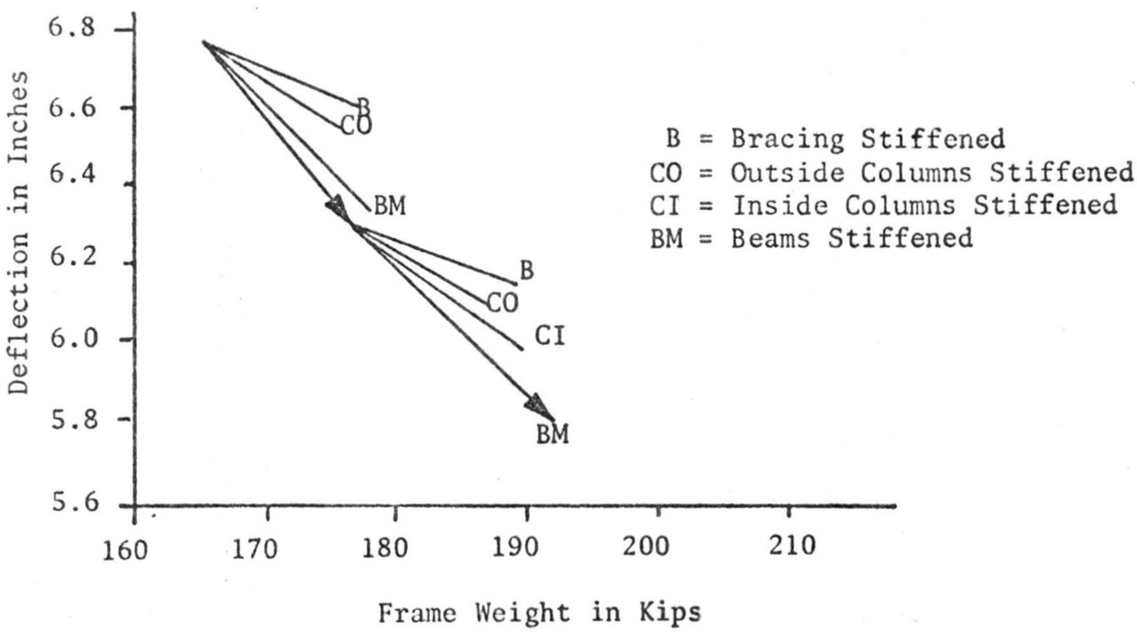


Fig. 4.14 Stiffening history of the K braced rigid frame, 20th floor fully braced.

from point 1 the slopes (steeper slopes show greater stiffening efficiency) of every area except the inside columns are almost the same as they were at the starting point. The inside columns have had an efficiency drop and the beams have become the best place for stiffening steel.

The drift of a structure can be considered to be the sum of the deflections arising from the flexibility of each individual member. As equal increments of steel are added to one type of component, such as the inside columns, the relative contribution of successive increments of steel will decrease, as shown in Fig. 4.13. After a number of increments have passed, the contribution to stiffening of one component may decrease to a value below the contribution of a different component, such as bracing. When this occurs, the most efficient placement of stiffening steel will shift to the other component.

Chapter 5

CONCLUSIONS

In this study, an investigation into the relative economies of various lateral bracing systems in steel plane frames was made. Nine different systems were placed in a standard 20 story, 3 bay framework with the same loading applied to each frame. The design was done with an automated design program written by the author. The results of the designs illustrate the relative efficiencies and behavior of several different types of lateral bracing systems.

Results obtained show that the lateral bracing systems which tend to engage the entire frame in action similar to a cantilever beam produce the stiffest and least weight designs for the geometry and loading considered. These types of systems result in most of the overturning moment being resisted by the couple formed by the exterior columns of the frame. The exterior column couple is more efficient than either the interior column couple or the bracing couple because of its greater lever arm. The multistory full width X bracing and the two systems including outrigger trusses were the most efficient in producing this type of action. The rigid frame action of a braced rigid frame also allowed the exterior column couple to carry more of the overturning moment than unbraced rigid frames or vertical trusses, but at the price of introducing larger moments into all beams and columns.

Vertical trusses (AISC Type II connections) were found to require 20 percent to 46 percent more steel than a comparable braced rigid frame. The economies of vertical truss systems depend upon the additional costs of the rigid and semi-rigid connections in the braced rigid frame as compared to the additional steel the vertical truss would require. A rigid frame proved to be an inefficient design for the 20 story frame considered because excessive amounts of stiffening steel were required to reduce drift.

The results indicate that the K bracing was structurally superior to the X bracing when placed in either a rigid frame or vertical truss. The shorter member lengths of the K bracing resulted in higher allowable axial loads and greater axial stiffness. Consequently, the K bracing resulted in a frame or truss which required less steel to stiffen and had a lighter final design weight than a comparable X braced frame or truss.

Steel used to stiffen a frame should be placed in areas (beams, columns or bracing) that contribute the most to the overall truss deflection. In general, increased stiffness can most efficiently be achieved by first increasing the columns forming the chords of a vertical truss. As one area is stiffened, the optimum location for the stiffening steel may shift to another area.

Results of this study can be used to help determine the efficiency of various lateral bracing systems in mid-height range structures, thus resulting in a savings of material and more economical structural frameworks.

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APPENDIX A

PROGRAM DESIGN[1]

Appendix A

PROGRAM DESIGN1

A.1 Introduction

Program DESIGN1 is an analysis-design computer program for plane frames composed of structural steel sections. Given the structural loading and geometry information, DESIGN1 selects a set of standard steel shapes which satisfy the applicable AISC interaction equations and other limitations imposed by the designer. The program begins by performing a stiffness analysis using the input structural geometry, loading, and either input or internally selected initial trial sizes. Each member is then checked to determine if all applicable stress conditions are satisfied. If a given member is over- or under-designed by more than the allowable limits, the program selects a new shape to support the member actions given by the previous analysis. After all members have been individually checked, the program reanalyzes the structure using the new member sizes and then performs another design check. This continued until the design check shows that all the members are adequate. Output includes the selected section shapes, member end actions, interaction equation values, joint deflections and maximum beam deflections. The general flow chart for the program is shown in Fig. A.1. DESIGN1 is written in ANSI FORTRAN IV computer language and consists of five basic parts: input, analysis, design, control, and output.

A.2 Input Information

Program DESIGN1 requires that all input data be consistent with the FORTRAN format field specified for the data. The program requires

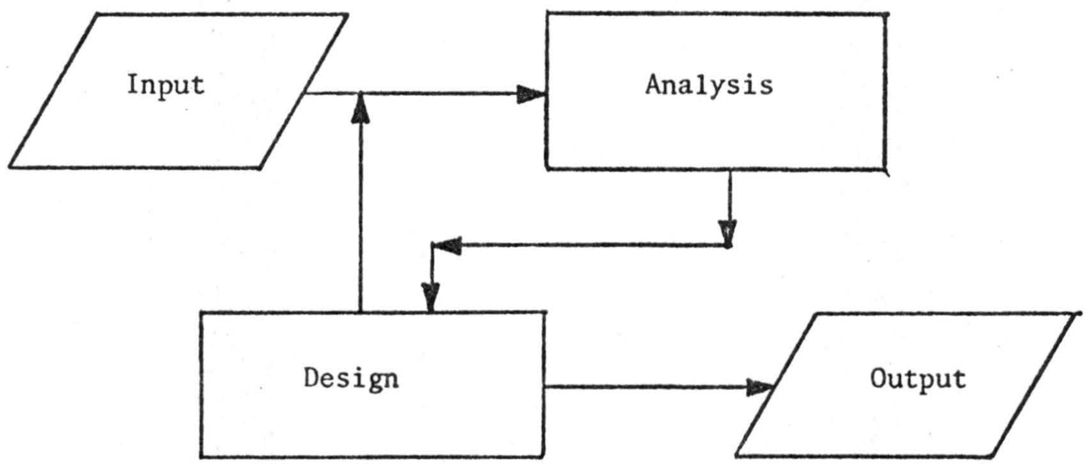


Fig. A.1 Simplified flow chart for Program DESIGN1.

a description of the geometry, loading, member information, and values used to specify how certain parts and options of the program are to be controlled. Initial member sizes may be input or chosen internally. Each AISC shape is assigned a member code number which is used as a reference for that shape. These code numbers are tabulated in Table A.1.

A.2.1 Definition of Structural System

All input data must have consistent force and length units of kips and inches. An orthogonal structure coordinate system must be established as shown in Fig. A.2. The program requires that columns be nearly parallel to the Y axis. All joints and members must be numbered sequentially beginning with one. Support conditions and loadings must be identified.

Joint Numbering

Each joint must be assigned a unique joint number, with the maximum number assigned equaling the number of joints in the structure. The amount of computer storage and run time is dependent upon the maximum difference between the two joint numbers at the ends of each member. In Fig. A.2, the maximum difference occurs with member 3, which has a difference of 6 - 3 = 3. Joints should be numbered to keep this difference as small as possible. For further explanations of joint numbering, see Sections A.2.2 set 3, and A.3.5.

Member Numbering

Each member must be assigned a unique number, with the maximum number assigned equal to the number of members in the structure.

Table A.1 Member reference table.

1 =	M 4	13.00	2 =	W 4	13.00	3 =	M 4	13.80	4 =	M 4	16.30	5 =	W 5	16.00
6 =	W 5	18.50	7 =	M 5	18.90	8 =	M 6	4.40	9 =	W 6	8.50	10 =	W 6	12.00
11 =	W 6	15.50	12 =	W 6	16.00	13 =	M 6	20.00	14 =	W 6	20.00	15 =	M 6	22.50
16 =	W 6	25.00	17 =	M 6	33.75	18 =	M 7	5.50	19 =	M 8	6.50	20 =	W 8	10.00
21 =	W 8	13.00	22 =	W 8	15.00	23 =	W 8	17.00	24 =	M 8	18.50	25 =	W 8	20.00
26 =	M 8	22.50	27 =	W 8	24.00	28 =	W 8	28.00	29 =	W 8	31.00	30 =	M 8	32.60
31 =	M 8	34.30	32 =	W 8	35.00	33 =	M 8	37.70	34 =	W 8	40.00	35 =	W 8	48.00
36 =	W 8	58.00	37 =	W 8	67.00	38 =	M10	9.00	39 =	W10	11.50	40 =	W10	15.00
41 =	W10	17.00	42 =	W10	19.00	43 =	W10	21.00	44 =	W10	22.40	45 =	W10	25.00
46 =	W10	29.00	47 =	M10	29.10	48 =	W10	33.00	49 =	W10	39.00	50 =	W10	45.00
51 =	W10	49.00	52 =	W10	54.00	53 =	W10	60.00	54 =	W10	66.00	55 =	W10	72.00
56 =	W10	77.00	57 =	W10	89.00	58 =	W10	100.00	59 =	W10	112.00	60 =	M12	11.80
61 =	W12	14.00	62 =	W12	16.50	63 =	W12	19.00	64 =	W12	22.00	65 =	W12	27.00
66 =	W12	31.00	67 =	W12	35.00	68 =	W12	36.00	69 =	W12	40.00	70 =	W12	45.00
71 =	W12	50.00	72 =	W12	53.00	73 =	W12	58.00	74 =	W12	65.00	75 =	W12	72.00
76 =	W12	79.00	77 =	W12	85.00	78 =	W12	92.00	79 =	W12	99.00	80 =	W12	106.00
81 =	W12	120.00	82 =	W12	133.00	83 =	W12	161.00	84 =	W12	190.00	85 =	M14	17.20
86 =	W14	22.00	87 =	W14	26.00	88 =	W14	30.00	89 =	W14	34.00	90 =	W14	38.00
91 =	W14	43.00	92 =	W14	48.00	93 =	W14	53.00	94 =	W14	61.00	95 =	W14	68.00
96 =	W14	74.00	97 =	W14	78.00	98 =	W14	84.00	99 =	W14	87.00	100 =	W14	95.00
101 =	W14	103.00	102 =	W14	111.00	103 =	W14	119.00	104 =	W14	127.00	105 =	W14	136.00
106 =	W14	142.00	107 =	W14	150.00	108 =	W14	158.00	109 =	W14	167.00	110 =	W14	176.00
111 =	W14	184.00	112 =	W14	193.00	113 =	W14	202.00	114 =	W14	211.00	115 =	W14	219.00
116 =	W14	228.00	117 =	W14	237.00	118 =	W14	246.00	119 =	W14	264.00	120 =	W14	287.00
121 =	W14	314.00	122 =	W14	320.00	123 =	W14	342.00	124 =	W14	370.00	125 =	W14	398.00
126 =	W14	426.00	127 =	W14	455.00	128 =	W14	500.00	129 =	W14	550.00	130 =	W14	605.00
131 =	W14	665.00	132 =	W14	730.00	133 =	W16	26.00	134 =	W16	31.00	135 =	W16	36.00
136 =	W16	40.00	137 =	W16	45.00	138 =	W16	50.00	139 =	W16	58.00	140 =	W16	64.00
141 =	W16	71.00	142 =	W16	78.00	143 =	W16	88.00	144 =	W16	96.00	145 =	W18	35.00
146 =	W18	40.00	147 =	W18	45.00	148 =	W18	50.00	149 =	W18	55.00	150 =	W18	60.00
151 =	W18	64.00	152 =	W18	70.00	153 =	W18	77.00	154 =	W18	85.00	155 =	W18	96.00
156 =	W18	105.00	157 =	W18	114.00	158 =	W21	44.00	159 =	W21	49.00	160 =	W21	55.00
161 =	W21	62.00	162 =	W21	68.00	163 =	W21	73.00	164 =	W21	82.00	165 =	W21	96.00
166 =	W21	112.00	167 =	W21	127.00	168 =	W21	142.00	169 =	W24	55.00	170 =	W24	61.00
171 =	W24	68.00	172 =	W24	76.00	173 =	W24	84.00	174 =	W24	94.00	175 =	W24	100.00
176 =	W24	110.00	177 =	W24	120.00	178 =	W24	130.00	179 =	W24	145.00	180 =	W24	160.00
181 =	W27	84.00	182 =	W27	94.00	183 =	W27	102.00	184 =	W27	114.00	185 =	W27	145.00
186 =	W27	160.00	187 =	W27	177.00	188 =	W30	99.00	189 =	W30	108.00	190 =	W30	116.00
191 =	W30	124.00	192 =	W30	132.00	193 =	W30	172.00	194 =	W30	190.00	195 =	W30	210.00
196 =	W33	118.00	197 =	W33	130.00	198 =	W33	141.00	199 =	W33	152.00	200 =	W33	200.00
201 =	W33	220.00	202 =	W33	240.00	203 =	W36	135.00	204 =	W36	150.00	205 =	W36	160.00
206 =	W36	170.00	207 =	W36	182.00	208 =	W36	194.00	209 =	W36	230.00	210 =	W36	245.00
211 =	W36	260.00	212 =	W36	280.00	213 =	W36	300.00						

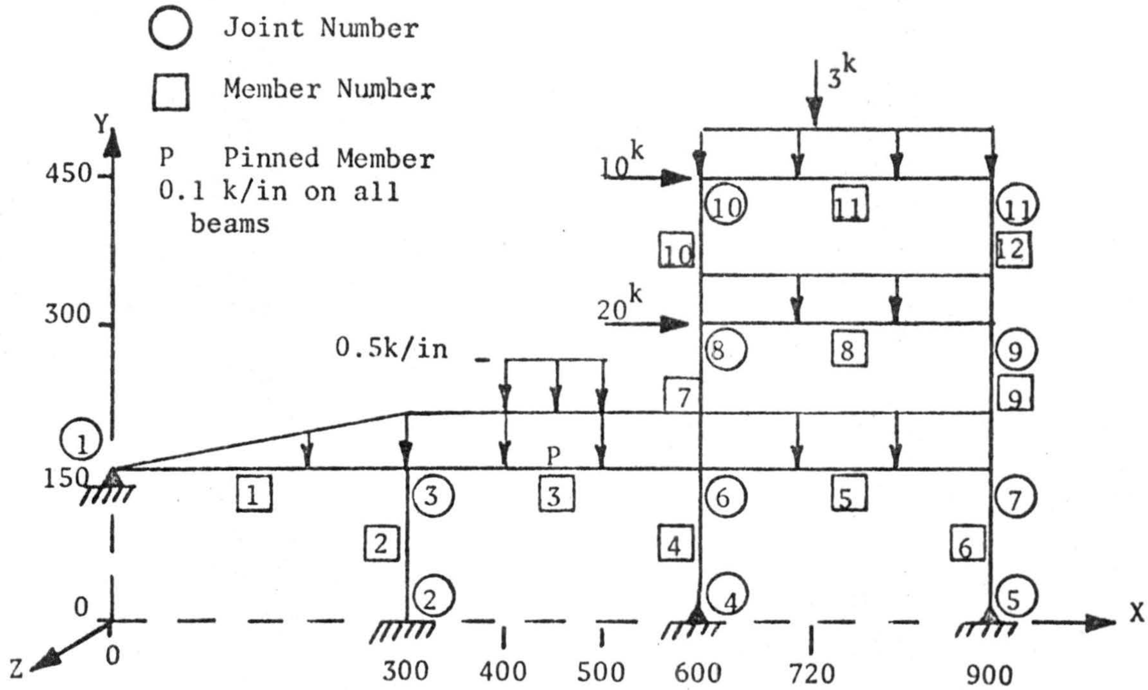


Fig. A.2 Plane frame with structural coordinate system, joint numbering and loads. All units are kips and inches.

A.2.2 Required Input Data

The input required for program DESIGN1 can be broken down into several units or data sets, as outlined below:

Set 1 Descriptive information and program control flags.

Set 1 consists of one card containing values of NJ, NM, E, SWAY, INTS, IOFLAG, IGROV, IPFLAG, INOUT, DEEFL, and MAXBAND using format (2I10, E10.2, F10.2, 5I5, F5.0, I5) where

NJ	Total number of joints.
NM	Total number of members.
E	Modulus of elasticity for all members.
SWAY	The height above which sidesway is present. This important variable is used in determining the effective length factors.
INTS	The number of iterations allowed before a subroutine is called to help convergence. INTS should not be less than three. The program assumes INTS = 5 if a zero or blank is read. For further explanations see Section A.5.1.
IOFLAG	A variable controlling the program operating mode and having two possible input values, zero or one. Zero allows the analysis-design sequence to iterate normally. A one instructs the program to only analyze the structure and print the interaction equation values for each member using the trial member sizes. No iteration is permitted.
IGRAV	A control flag which specifies the loading condition with one of three possible values; zero, one, or two.

IGRAV=0 specifies that analysis and design be performed using input load values with no modification for wind or moment redistribution. IGRAV=1 signifies a gravity loading and allows the program to use a maximum one-tenth redistribution of the negative beam momemts when permitted by AISC 1969, Section 1.5.1.4.1. A value of two signifies a wind loading and all input loadings are reduced one-fourth and no moment redistribution is allowed.

IPFLAG This control flag specifies the output desired and may have one of four values: zero, one, two, or three. IPFLAG=0 results in an echo of the input data, plus final member sizes, actions, interaction equation values, and member deflections (for deflections, also see Section A.6.4). IPFLAG=1 results in the output specified by the value 0 and also a listing after each iteration of the new member sizes and the member K_x, P_{max}, M_{max}, C_{mx}, C_{my}, F_a, and F_{bx} values. IPFLAG=2 causes a card to be punched for each member giving the member number and its size code (information useful to use as trial member input for later runs). In addition, a member size code table is printed and all the output for IPFLAG=0 is generated. IPFLAG=3 is used when all the above output is desired.

INOUT This flag also controls the output and has three possible values: zero, one, and two. INOUT=0 is the normal mode. INOUT=1 suppresses the input echo. INOUT=2 allows only the printing of joint deflections unless the maximum drift deflection is within the limits defined by DEFFL. If the drift deflections are less than DEFFL restrictions, output is the same as INOUT=1.

DEFFL	Drift deflection limit. This is used only when INOUT=2. If drift deflections are limited to height/500, enter 500; if the limit is height/600, enter 600. If DEFFL=0 is entered, 500 is assumed.
MAXBAND	Maximum bandwidth of the stiffness matrix. See Sections A.3.5 and A.8.

Set 2 Geometry

The set consists of one card for each joint and contains values for JN, X, Y, FIXYZ(1), FIXYZ(2), and FIXYZ(3) using format (I10, 2F10.3, 3F5.2) where:

JN	Joint number.
X	Structural X coordinate of the joint.
Y	Structural Y coordinate of the joint.
FIXYZ(1)	Fixity of the joint against X translation.
FIXYZ(2)	Fixity of the joint against Y translation.
FIXYZ(3)	Fixity of the joint against Z rotation. To fix a joint against translation or rotation, enter any non-zero positive number (usually one) for the appropriate FIXYZ. If the movement is free, enter a zero or leave blank. For further explanations see Section A.3.4.

Set 3 Individual member data

The set consists of one card for each member and contains values for M, JC, KC, FYIELD, BRCEX, BRECY, DL, YMNE, YMFE, IBC, IPIN, and SETSIZ using format (3I5, F10.2, I5, 2F10.2, 2I2, I6) where:

M Member number.

JC Joint number of the "near end" of the member. This must be the lower end for columns which frame into supports, and this convention is suggested for all columns. Beams may have either end designated the near end, but the use of the left end as the near end is most convenient and is suggested. The locations of the near and far ends must be recognized when specifing member loadings.

KC Joint number of the far end.

FYIELD Yield point of the steel in the member. Any grade may be used. 36 ksi is assumed if none is specified.

BRCEX Maximum distance between lateral bracing points along the strong axis.

BRCEY Maximum distance between lateral bracing points along the weak axis. The effective length factor for the weak axis, K_y, is always assumed to be one. If this factor varies, vary BRCEY accordingly. For beams, this is the distance between lateral bracing of the compression flange.

DL Limit on the maximum nominal depth permissible for the member. If not specified, 36 and 14 inches are assumed for beams and columns respectively.

YMNE	Weak axis (out of plane) moment on the near end of the member. YMNE must be a positive number. Shear reactions coming in from out of plane members can be entered as joint loads parallel to the X or Y axes.
YMFE	Weak axis (out of plane) moment on the far end of the member. YMFE must be a positive number.
IBC	Beam or column designation. IBC=0 designates a beam, IBC=1 designates a column. Beams are usually picked from the economy tables. Because of this, bracing members should be declared columns. The effective length factor is always set equal to one except for vertical or near vertical columns.
IPIN	IPIN=1 will cause the program to treat the member as pinned at both ends. See Sections A.3.2 and A.3.3.2 for details.
SETSIZ	This option allows the programmer to declare the member to be a given size which will not change throughout the iterations. Enter the appropriate member size code number for the member.

Set 4 Member properties and references

These data sets reference the entire selection of AISC M and W shapes. Set 4a consists of three cards which reference the nominal depths to the member size codes. Set 4b contains 213 cards, each describing the properties of an AISC M or W shape. Set 4c contains four cards which reference 54 economy table shapes. This set should have been included with the program package.

Set 5 Trial member sizes, which consists of sets 5a and 5b given below.

Set 5a Trial sizes flag

The set consists of one card containing the value for MEMTRI using format (I5) where:

MEMTRI - MEMTRI=1 indicates that the trial sizes will be read in. If MEMTRI=1 is specified the entire set 5b must follow. DESIGN1 does have a subroutine which will find preliminary sizes if necessary, but it is not very efficient. MEMTRI=0 will cause the subroutine to size all the members initially and set 5b is not needed.

Set 5b Trial member sizes and equivalences

The set consists of one card for each member containing values for M, MSIZE, and IEQUIV using format (3I10) where:

M Member number.

MSIZE Number of the initial trial member size selected from Table A.1. MSIZE=0 causes the sizing subroutine to choose the initial size.

IEQUIV This option allows the user to declare the equivalence of this member to another member. Typically this would be used in a symmetrical structure with an unsymmetrical loading, as in the top half of Fig. A.2. For the loading shown, member 7 is probably less critical than member 9, but it is assumed that symmetry dictates that they should be the same size. Thus enter a 9 as the IEQUIV value for member 7. Do not specify 7 as the IEQUIV value for member 9.

Set 6 Member loadings, which consists of sets 6a and 6b given below.

Set 6a Number of member loading

The set consists of one card containing the value for MLOADS using Format (I10).

MLOADS Total number of member load cards to follow. A member may have more than one loading. MLOADS must not exceed the dimension of the variables in the common block /LOADS/. Member weights are automatically considered within the program.

Set 6b Member loads

The set consists of one card for each member loading defining MEM, JCODE(s), RLOAD(s), RN, and RF using format (2I5, 3F10.2) where:

MEM Member number.

JCODE Loading code:

1 = concentrated load.

2 = uniform load on all or part of the span.

3 = concentrated moment. Counterclockwise is positive.

4 = joint displacement in member Y direction.

5 = temperature change. Positive for increasing temperatures. Units in degrees Fahrenheit.

6 = joint displacement in member X direction.

7 = linearly varying load over the full span. The larger load intensity must be at the near end of the member.

RLOAD Magnitude of the loading. Care must be taken to enter the correct sign. RLOAD is specified by the member coordinate system shown in Fig. A.3. If the convention of placing the near end on the left is followed, most gravity loadings will have a negative sign. Member coordinate systems are explained in Section A.3.

RN Distance from the near end of the member to the load, expressed as a fraction of the span length. RN must be specified for uniform loads, concentrated loads, and concentrated moments. For example, a uniform load covering the middle third of the span would have RN=.333. Note that RN=0 would denote a load starting at the near end.

RF Distance from the far end to the start of the load, expressed as a fraction of the span length. RF is only needed for uniform loads. If RF is not given, the uniform load is assumed to extend to the far end.

Set 7 Joint loadings, which consists of sets 7a and 7b given below.

Set 7a Number of joint loadings

The set consists of one card containing the value for JLOADS using format (I5) where:

JLOADS Total number of joint loading cards to follow. A joint may have any number of cards. There is no limit on the number of joint loadings.

Set 7b Joint loads

The set consists of one card for each joint loading defining JN, X, Y, and Z using format (I10, 3F10.2) where:

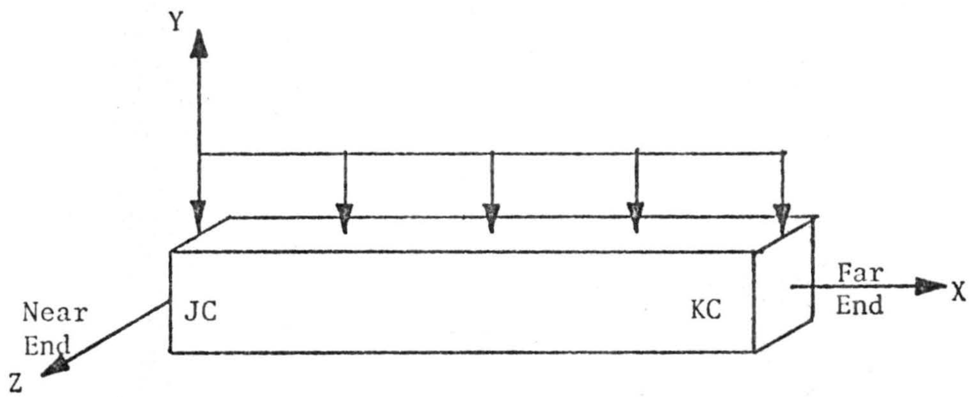


Fig. A.3 Loaded member with member coordinate system.

JN	Joint number.
X	Force in the X direction.
Y	Force in the Y direction.
Z	Moment about the joint, moment vector directed in Z direction, i.e., counterclockwise positive. Do not place a concentrated moment on a joint if all members entering the joint are pinned at the joint. Note the X and Y directions for the joint refer to the structural coordinate system.

A.3 Analysis

The analysis routine utilizes a fully automated stiffness approach. Notation and procedure are more fully discussed in Ref. 23. All loadings are stored in the AJ (Actions at Joints) matrix. A series of routines transforms individual member stiffnesses from member to structural coordinates and then adds these transformed member stiffnesses into the large S (Stiffness matrix in structural coordinates) matrix. Then the DJ (Displacements at Joints) matrix is found by solving

$$DJ = S^{-1} * AJ \tag{A.1}$$

using a simultaneous equation solving routine. These joint displacements are then utilized to compute the member end actions. Fig. A.4 shows the analysis flow chart. This method of analysis does not consider shear deformations nor changes in member stiffness resulting from axial loadings.

A.3.1 Program Trial Member Sizing

If trial sizes are not INPUT or if an individual member does not have a trial size, subroutine SIZE is called. The subroutine bases

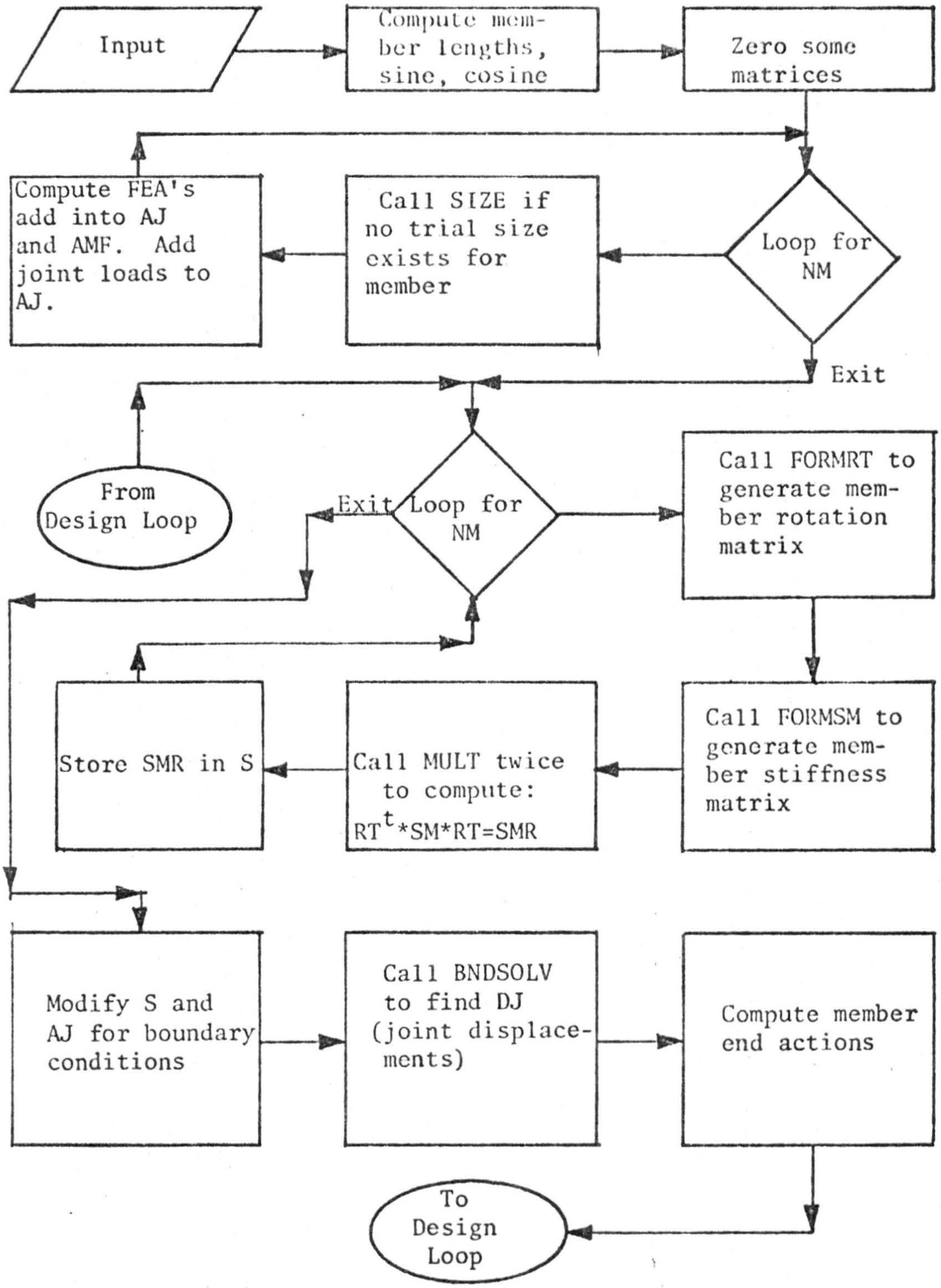


Fig. A.4 The analysis loop.

the trial member size upon span to depth ratios. This usually provides a fair guess for beams, but it tends to pick very light sections for columns.

A.3.2 Loadings

Three matrices and several subroutines are used to transform member loadings into an acceptable form for analysis. Subroutine FIXEND computes the six fixed end actions (FEA) caused by a member loading and places them in the FEA matrix. Each member FEA matrix is taken by subroutine STORE and added to the AJ and AMF (Final Member Actions) matrices. The fixed end action values from member self weights are computed by subroutine WTFEA and stored by subroutine STORE. However, if a member is pinned, then only the axial and shear actions are computed by FIXEND and WTFEA and placed in FEA. Joint loads are added directly into AJ.

A.3.3 Formation of the Stiffness Matrix

Unlike the AJ and AMF matrices, the S matrix is dependent upon the individual member stiffnesses. A change in member size during the design loop changes the stiffness matrix, and thus the analysis. Therefore the formation of the stiffness matrix must be repeated during each iteration through the analysis loop.

The S matrix represents the summation of all the individual member stiffnesses. The summation is accomplished by computing each SM (Stiffness Member) matrix, rotating it to structural coordinates, indexing it, and adding it into the structural stiffness matrix.

A.3.3.1 FORMRT

Subroutine FORMRT builds the members' RT (Rotation, Translation) matrix. This 6 x 6 matrix inclues the direction cosines which define the relation between member coordinates (Fig. A.3) and structural coordinates (Fig. A.2).

A.3.3.2 FORMSM

Subroutine FORMSM generates the member stiffness matrix in member coordinates. This matrix contains AE/L, EI/L, and EI/L^2 terms. Pinning of a member is achieved by dropping the EI terms to a near zero value, effectively leaving only the axial terms in SM.

A.3.3.3 SMR

Subroutine MULT is used to multiply the rotation and member stiffness matrices together to form the SMR (Rotated Member Stiffness) matrix. Using the equation

$$SMR_m = RT_m^T * SM_m * RT_m \tag{A.2}$$

the member stiffness matrix in structural coordinates is formed.

The SMR matrix is then indexed to its appropriate joints by subroutine INDEX and added into the S matrix. The formation of SMR is repeated for each member.

A.3.4 Boundary Conditions

The stiffness approach to structural analysis relates directly to the joint motions. The three possible motions of a planar joint, structural X translation, structural Y translation and rotation each generate an equation which becomes a row in the S, DJ, and AJ matrices. The positive X and Y translations and the positive rotation of a joint correspond to the structure X, Y, and Z coordinates.

Modification for joint fixities is accomplished by mathematically forcing the corresponding joint displacement to zero. Because the system of equations

$$S * DJ = AJ \tag{A.3}$$

is solved, fixity is achieved by placing a very large number, 10^{60}, on the appropriate diagonal term of the S matrix. This increases the stiffness of the joint against that translation or rotation enough that effectively a zero displacement results. The AJ corresponding to that joint motion must be zeroed.

A.3.5 Number of Equations-Bandwidth

Three equations are generated at each joint. The structure shown in Fig. A.2 would result in 3 * 11 = 33 equations.

Two properties of the S matrix, symmetry and banding, allow a considerable savings in computer storage. Symmetry allows the storage of just the diagonal and one-half of the off-diagonal terms, since the other half of the off-diagonal terms are related to the stored half by

$$S_{ij} = S_{ji} \tag{A.4}$$

Furthermore, beyond a certain distance from the diagonal, only zero terms are present and these zero terms need not be stored. The bandwidth of the S matrix depends upon the maximum difference between the two joint numbers of a member and is given by:

$$\text{Bandwidth} = 3 * (\text{Maxdiff} + 1). \tag{A.5}$$

For the structure of Fig. A.2,

$$\text{Bandwidth} = 3 * (6 - 3 + 1) = 12. \tag{A.6}$$

Thus, instead of storing the entire 33 x 33 S matrix, only the 33 x 12 half band is stored.

A.3.6 BNDSOLV

Subroutine BNDSOLV is essentially a Gauss elimination routine that operates on the symmetrical, banded stiffness matrix. Gauss elimination is a well-known procedure for solving systems of simultaneous equations.(8) The elimination procedure takes the square coefficient matrix and manipulates it until the lower off-diagonal terms are all zero. Once this is done the unknowns can be found by using a back substitution. Subroutine BNDSOLV does this, but uses the symmetrical, banded S matrix.

A.3.7 Member End Actions

Once the joint displacements are found, the axial, shear and moment terms at the ends of a member can be found by

$$AMF = SM * RT * D + FEA \tag{A.7}$$

where D is the matrix of displacements at the ends of a member. The actions in member coordinates are shown in Fig. A.5

A.4 Design

The objective of the design loop is to find the lightest M or W shape which satisfies the 1969 AISC code requirements and other constraints imposed by the user. The loop utilizes the provisions of AISC Sections 1.5 (allowable stresses), 1.6 (combined stresses), 1.8 (stability and slenderness ratios) and 1.9 (width-thickness ratios) to check the applicable interaction equation(s). If the member size is not satisfactory, it is changed and the adequacy of the new size is determined. After a member size is chosen, a search is made to find another adequate section which is lighter, but still meets any nominal

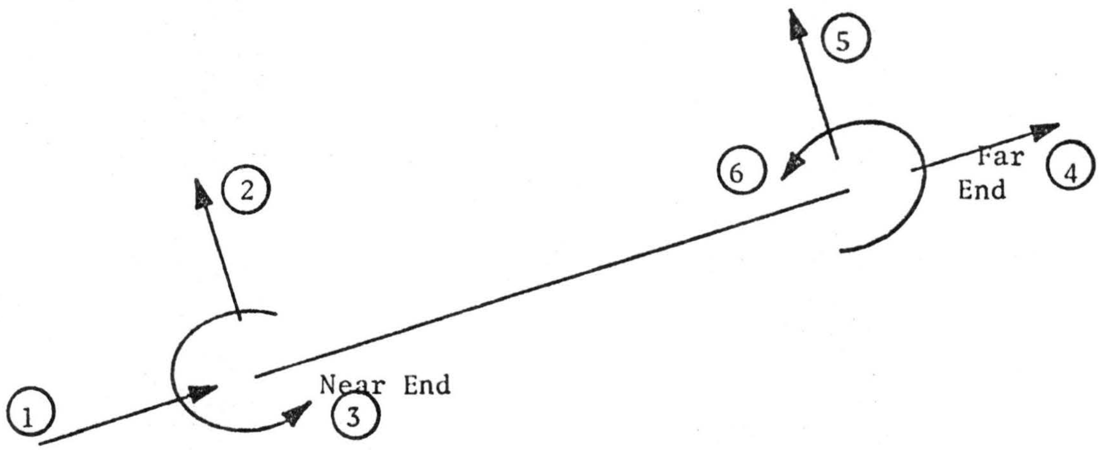


Fig. A.5 Positive member end actions.

depth requirements. An abbreviated flow chart of the design loop is shown in Fig. A.6.

A.4.1 DO 142

The statement DO 142 M=1, NM sets up the control in the design loop. M is the member number. J, the coded AISC shape number for the current member, is stored in MSIZE. Entrance to the loop comes directly from the analysis section, and the loop exits to the control section.

A.4.2 IEQUIV and SETSIZ

If a member is declared equivalent or has its size set, control is immediately passed to the end of the loop and a change in the member size is not allowed. Member sizing for equivalent members is checked outside the design loop.

A.4.3 Effective Lengths

The effective length is assumed to be one for all members except vertical or near vertical, unpinned members which are declared columns in the array IBC. A column is assumed to be "vertical or near vertical" if the sine of the angle between the member and the structural X axis is greater than 0.95. The effective lengths are calculated in subroutine EFLEN which can consider both sidesway prevented and sidesway permitted cases. The subroutine calculates the stiffness to length ratios of all members framing into the column ends and uses these values to solve the appropriate transcendental equation[5] for K_x by trial and error. If a column has pinned or fixed ends, stiffness values of 10 and 1, respectively, are assumed.[2] All out of plane effective lengths are assumed to be unity. The program will select only member

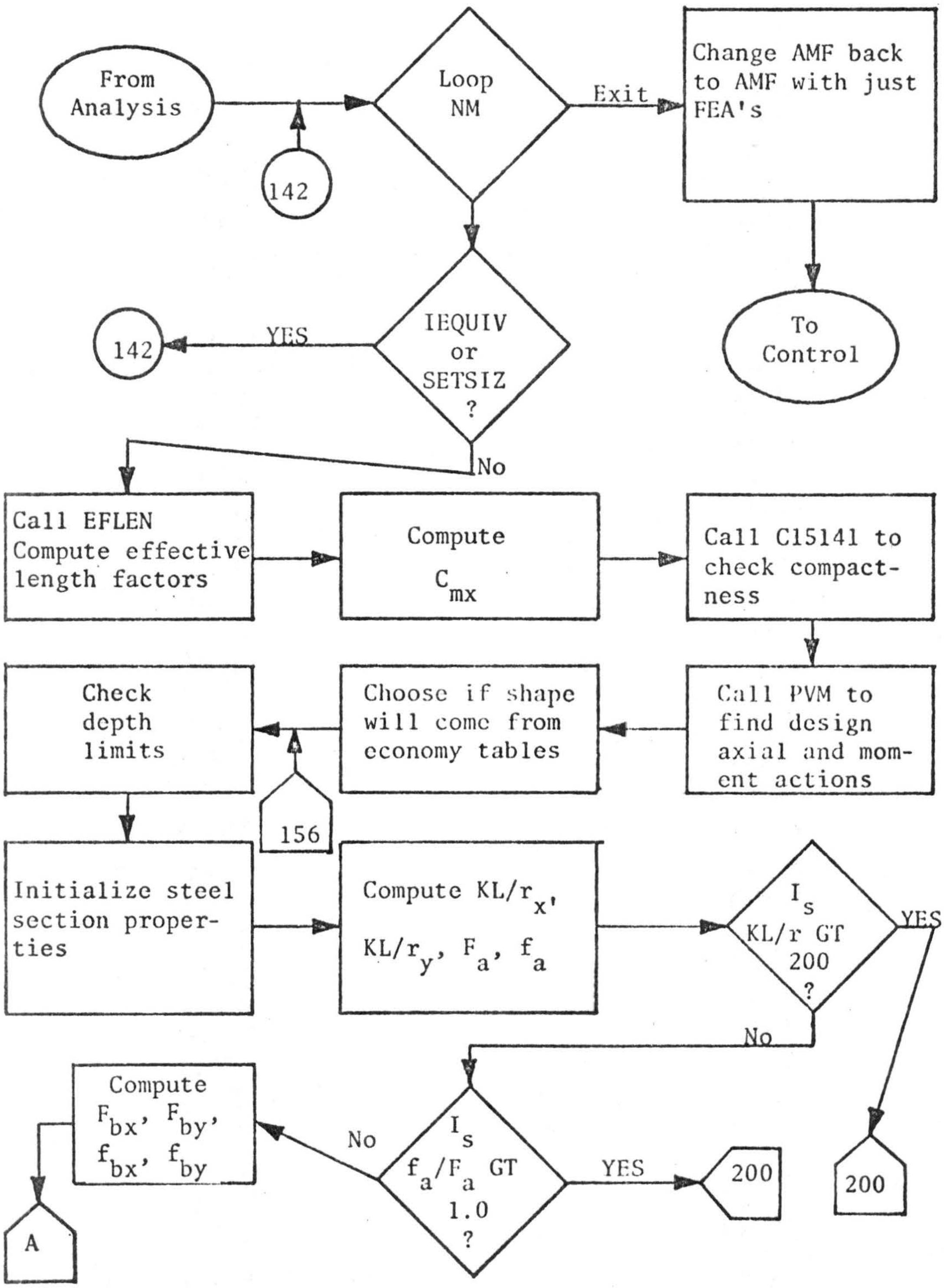


Fig. A.6 The design loop.

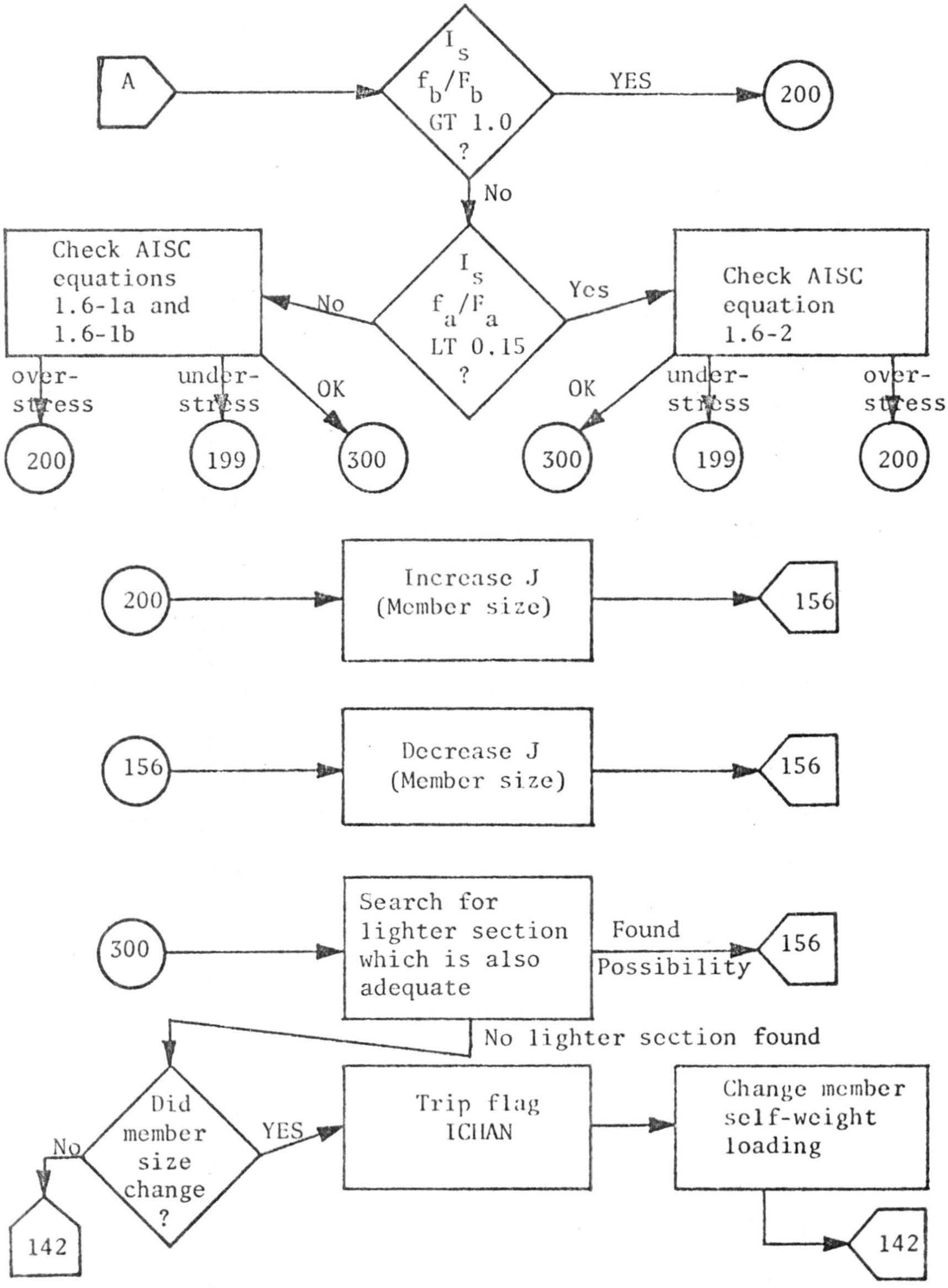


Fig. A.6 (continued) The design loop.

sizes with Kl/r less than 200 about both axes. Effective lengths are ignored for tension members although the AISC code does recommend some limitations in Section 1.8.4.

A.4.4 Compactness

Subroutine C15141 checks the AISC code requirements for section compactness (Section 1.5.1.4.1). This compactness criterion is used to decide whether a one-tenth reduction in negative moment (see Section A.4.5) can be used and whether an allowable strong axis bending stress of up to $0.66F_y$ is permissible.

A.4.5 Design Actions

Subroutine PVM computes the maximum axial and moment actions acting on a member. It assumes that the maximum axial force occurs at one of the ends of a member. PVM computes the moment at eleven points along the members span including each end, and finds its maximum values. If allowed by the compactness criteria and the loading, it will reduce the negative moments up to ten percent. This reduction is made in one percent increments and is stopped before the full ten percent redistribution is reached if the positive moment is increased to a value equal to or larger than the negative moment. These moment reductions are not carried to the supporting columns. The subroutine returns the maximum axial, end moment and mid-span actions to the main program. Shear is not considered by the design loop, but a shear check is made during output.

A.4.6 Economy Tables

If a member has an IBC designation of zero and if f_a/F_a is less than 0.15, the section is chosen from the economy tables. If a nominal depth limitation intervenes, the design loop exits from the economy table mode.

A.4.7 Statement 156 CONTINUE

This statement marks the beginning of the free iteration for the member size. Although the values of K_x, C_m, M_{max}, and P_{max} can change with variations in member size, these changes are usually small and their values are assumed to be constant during the iteration. Before the interaction equations are computed several checks and initializations must be performed. This includes a check to determine if the current member size violates the nominal depth restrictions for that member. If it is satisfactory, the member section properties are initialized.

A.4.8 Interaction Equations

The central part of the design loop determines if the member size satisfies the interaction equations (AISC equations 1.6-1a and 1.6-1b or 1.6-2). The program finds the allowable and actual axial and bending stresses and makes a preliminary check of the interaction equations. If the member passes these initial checks, the additional variables necessary for the interaction equations are calculated. Finally, the interaction equation values are computed.

Depending upon the interaction equation values, the program will choose to increase, decrease, or accept the member size. For all three equations, if the value exceeds 1.03 the member size is increased. The member size is decreased if the value is below 0.95 for equation 1.6-2 and 0.93 for equations 1.6-1a and 1.6-1b. The size is accepted if the value falls between the two limits. The 0.95 and 0.93 values were chosen on the assumption that the next lightest section would have a value greater than 1.03. The slightly higher value for equation 1.6-2 was adopted to make the maximum use of the economy tables. Of course, there may not be a size which falls in the acceptable region. In this case the program will begin to oscillate, picking a size which is above the upper limit, then choosing a size which is below the lower limit, then cycling back to the first size, and so on. After this oscillatiion occurs two times the program chooses one of the sizes and does not retest the member until after the next analysis.

Once a size has been selected, a search is made to find a lighter size which is still adequate. Because the basic factor in the search is weight, it is assumed that the better section, if it exists, has a nominal depth which is equal to or greater than the current nominal depth. This assumption is made to save the additional computer time required to search through all the sections which are lighter.

If a member has changed size during the design loop, its member self weight loading has changed. Therefore, it is necessary to remove the old member weight loading from the AJ and AMF matrices and replace it with the new member self weight loading. This is done by calling subroutines WTFEA and STORE twice, once to remove the old member weight and again to replace it. A flag (ICHAN) is also set if the member size has changed.

A.5 Control

Two requirements must be met by the control around the analysis-design sequence. First, all members should be as fully stressed as possible without exceeding the allowable stresses. Second, convergence to the final member selections should be as rapid as possible. The basic problem is that in a highly indeterminate structure such as a plane frame, the actions in a given member are dependent upon the stiffnesses of the other members framing into it. Thus, a change in member size not only causes a change in the member's actions, but also in the actions of the members around it. This means that another analysis is required each time a member size is changed.

The most important control variable is IOFLAG. The program has three possible modes, corresponding to IOFLAG values of zero, one, and two. IOFLAG=0 allows the analysis-design loop to cycle freely until a set of close-to-fully stressed members are found. Because of the way convergence is achieved, some members may end up slightly overstressed. IOFLAG=2 allows only these overstressed members to increase in size. IOFLAG=1 is the output mode.

A.5.1 IOFLAG=0

The program begins here. Using individual trial sizes, the program performs a complete structural analysis and resizes the members if necessary. The analysis is performed again and the new set of members is checked and resized if necessary. Convergence is speeded in two ways. First, if a member retains the same size for three consecutive iterations, it is set to that size for the remainder of the IOFLAG=0 mode. Second, after the number of iterations exceeds the variable INTS (see A.2.2), subroutine SETUM is called to speed convergence. SETUM examines the pattern of member sizes during the last three iterations. If an oscillation is present, subroutine SETUM selects the larger member code size and sets the member to that size. Convergence is said to have occurred if none of the member sizes are changed by the design loop. This usually takes about nine iterations, assuming INTS=5.

A.5.2 IOFLAG=2

Because a member size may be set early in the IOFLAG=0 mode, changes in member sizes around that member may cause it to be over-stressed. The IOFLAG=2 mode allows the members to increase (but not decrease) in size if they do not satisfy the interaction equations. This results in a final set of members which are all adequate. This mode usually lasts about three iterations..

A.6 Output, IOFLAG=1

The IOFLAG=1 mode may occur in two manners. Either the program has gone through the two modes described in A.5.1 and A.5.2 or a check of the interaction equations is being run on the input trial sizes. Both cases are handled alike. An analysis is performed and the values of the interaction equations are computed. No changes in member sizing are allowed to occur. The output consists of the member number, size, actions, interaction equation values, and deflection information.

A.6.1 Actions

The member end actions are printed. These actions are specified in member coordinates, as shown in Fig. A.5. The maximum design moment, which can occur at one of the internal tenth-span points, is also printed.

A.6.2 Interaction Equation Values

The controlling interaction equation number and its values are printed. The three terms output are axial, strong axis bending and weak axis bending terms respectively.

A.6.3 Shear

If the shear stress on the ends of a member exceeds the allowable, an error message is printed. The maximum shear stress is always assumed to occur at the member ends. Shear stresses are not considered during the design loop.

A.6.4 Deflections

Member deflections are calculated for the beams. Subroutine DEFLET uses the Newmark method of concentrated angle changes[19] to compute the deflections at nine points along the span. Deflections are relative to the ends of the span and include the member self weights. The span to deflection ratios are also printed.

Joint deflections are outputted. These are referenced to the structural coordinate system and have units of inches or radians. Deflection limitations are not considered by the program.

A.6.5 Punched Output

In addition to the above output, punched output may also be generated giving the member number and its coded shape size number. The punched cards are in the format specified for inputting the trial member sizes. These cards are useful for making adjustments in member sizes, for example to stiffen the frame, and should be used with an IOFLAG=1 mode. Table A.1 is also printed for reference.

A.7 Design Example

The frame shown in Fig. A.2 is designed in this section. The geometry, loadings and the joint and member numbering are also shown in Fig. A.2. Additional assumptions are listed below:

1. 50 ksi steel was assumed for members 1 and 3 and 36 ksi steel for the other members.
2. Bracing lengths were assumed and are shown in Fig. A.8.
3. Nominal depth limits were assumed to be 14 inches on columns and 36 inches on beams, except for member number 3, a beam, which was limited to 14 inches.
4. There are no out of plane moments.
5. Rigid framing (AISC Type I) is present everywhere except for member number 4, which is pinned at both ends.
6. Sidesway is present above the first story (150 inch level).
7. Neither an increase in allowable stress for the wind loading nor a one-tenth reduction in negative beam moments was permitted.

The input data are shown in Fig. A.7. IOFLAG=0 was specified to produce the desired free iteration of the analysis-design sequence. MAXBND=24 was used because the stiffness matrix was dimensioned for 24. Equivalences were declared between members 7 and 9 (9 critical) and 10 and 12 (12 critical). The input echo is shown in Fig. A.8.

The output of the free iteration run is listed in Fig. A.9. An initial inspection of the output reveals that many members seem under-stressed (and one overstressed). In fact, only two members have adequate interaction equation values of 0.95 or greater, and only five have values greater than 0.90. However, it can be shown that these apparent deficiencies are primarily due to some of the assumptions made at the start of the problem.

The overstress in member 7 is due to its equivalence to member 9. Because member 9 was assumed to be more critical than member 7, it was

```
          11         12  29000.      150.          5    0    0    0    0       24
           1       150.         0.    1.    1.    0.
           2         0.       300.    1.    1.    1.
           3       150.       300.
           4         0.       600.    1.    1.    0.
           5         0.       900.    1.    1.    0.
           6       150.       600.
           7       150.       900.
           8       300.       600.
           9       300.       900.
          10       450.       600.
          11       450.       900.
   1       1    3        50.       300.       300.
   2       2    3                  150.        75.                           1
   3       3    6        50.       300.       300.   14
   4       4    6                  150.        75.                           1 1
   5       6    7        50.       300.        50.
   6       5    7                  150.        75.                           1
   7       6    8                  150.       150.                           1
   8       8    9                  300.        50.
   9       7    9                  150.       150.                           1
  10       8   10                  150.       150.                           1
  11      10   11                  300.        50.
  12       9   11                  150.       150.                           1
   1
           1         88
           2         21
           3         88
           4         21
           5         88
           6         21
           7          0         9
           8          0
           9          0
          10          0        12
          11         88
          12          0
           8
   1       7         .1
   1       2        -.1
   3       2        -.1
   3       2       -0.5     0.333      0.333
   5       2        -.1
   8       2        -.1
  11       2        -.1
  11       1       -3.0      0.40
   2
           8       20.0
          10       10.0
```

Fig. A.7 Input data for sample problem.

THIS PROBLEM HAS 11 JOINTS AND 12 MEMBERS,

THE MODULUS OF ELASTICITY EQUALS 29000.00 KSI.

THESTRUCTURE IS UNBRACED AGAINST SIDESWAY ABOVE 150.0 INCHES.

THE PROBLEM WILL ITERATE 5 TIMES BEFORE CONVERGENCE IS FORCED.

JOINT	COORDINATES		FIXITY		
NUMBER	X	Y	X	Y	ROT
1	0.0	150.0	1.0	1.0	0.0
2	300.0	0.0	1.0	1.0	1.0
3	300.0	150.0	-0.0	-0.0	-0.0
4	600.0	0.0	1.0	1.0	0.0
5	900.0	0.0	1.0	1.0	0.0
6	600.0	150.0	-0.0	-0.0	-0.0
7	900.0	150.0	-0.0	-0.0	-0.0
8	600.0	300.0	-0.0	-0.0	-0.0
9	900.0	300.0	-0.0	-0.0	-0.0
10	600.0	450.0	-0.0	-0.0	-0.0
11	900.0	450.0	-0.0	-0.0	-0.0

MEMBER INDICES AND OTHER MEMBER DATA

MEMBER	NEAR	FAR	YIELD	X BRACE	Y BRACE	DEPTH	OUT OF PLANE MOMENTS	
NUMBER	END	END	POINT	LENGTH	LENGTH	LIMIT	NEAR END	FAR END
1	1	3	50.00	300.00	300.00	36	-0.00	-0.00
2	2	3	36.00	150.00	75.00	14	-0.00	-0.00
3	3	6	50.00	300.00	300.00	14	-0.00	-0.00
4	4	6	36.00	150.00	75.00	14	-0.00	-0.00
5	6	7	50.00	300.00	50.00	36	-0.00	-0.00
6	5	7	36.00	150.00	75.00	14	-0.00	-0.00
7	6	8	36.00	150.00	150.00	14	-0.00	-0.00
8	8	9	36.00	300.00	50.00	36	-0.00	-0.00
9	7	9	36.00	150.00	150.00	14	-0.00	-0.00
10	8	10	36.00	150.00	150.00	14	-0.00	-0.00
11	10	11	36.00	300.00	50.00	36	-0.00	-0.00
12	9	11	36.00	150.00	150.00	14	-0.00	-0.00

BMCOL	PINNED	SET SIZE
-0	-0	-0
1	-0	-0
-0	-0	-0
1	1	-0
-0	-0	-0
1	-0	-0
1	-0	-0
-0	-0	-0
1	-0	-0
1	-0	-0
-0	-0	-0
1	-0	-0

Fig. A.8 Input data echo.

MEMBER LOADINGS

THERE ARE 8 LOADED MEMBERS.

MEMBER	JCODE	MAGNITUDE	R-DIST NEAR	R-DIST FAR
1	7	.100	-0.000	-0.000
1	2	-.100	-0.000	-0.000
3	2	-.100	-0.000	-0.000
3	2	-.500	.333	.333
5	2	-.100	-0.000	-0.000
8	2	-.100	-0.000	-0.000
11	2	-.100	-0.000	-0.000
11	1	-3.000	.400	-0.000

JOINT LOADINGS

THERE ARE 2 LOADED JOINTS.

JOINT	X FORCE	Y FORCE	Z MOMENT
8	20.00	-0.00	-0.00
10	10.00	-0.00	-0.00

TRIAL MEMBER SIZES

MEMBER	AISC SHAPE	EQUIVILANCE
1	W14 30.00	-0
2	W 8 13.00	-0
3	W14 30.00	-0
4	W 8 13.00	-0
5	W14 30.00	-0
6	W 8 13.00	-0
7	W10 54.00	9
8	W21 44.00	-0
9	W10 54.00	-0
10	W10 54.00	12
11	W14 30.00	-0
12	W10 54.00	-0

Fig. A.8 (continued)

```
MEMBER SIZES AND ACTIONS

MBER NUMBER    1
W21    68.00                                ACTIONS
           AXIALN        SHEARN      MOMENTN      AXIALF        SHEARF       MOMENTF
           -31.0          3.55       .146E-10      31.0          -7.85       -.129E+04

        DESIGN EQUATION VALUES
        FORMULA 1.6-2 =    .0516 +    .8602 +  0.0000 =    .9118
--- THIS MEMBER HAS AXIAL TENSION.   FORMULA 1.6-1B WAS ACTUALLY CHECKED.
        THE DESIGN MOMENT WAS   .129E+04              THE SLENDERNESS RATIO WAS166.7

        BEAM DEFLECTION
        THIS BEAM DEFLECTED   .0463 INCHES MAXIMUM AT 210.0 INCHES FROM THE LEFT
        THE SPAN TO DEFLECTION RATIO WAS 6477.

MBER NUMBER    2
W 8    10.00                                ACTIONS
           AXIALN        SHEARN      MOMENTN      AXIALF        SHEARF       MOMENTF
            29.8        -.563E-02     .990        -29.6        .563E-02     -1.84

        DESIGN EQUATION VALUES
        FORMULA 1.6-1A =    .7043 +    .0217 +  0.0000 =    .7260
        FORMULA 1.6-1B =    .4655 +    .0122 +  0.0000 =    .4778
        THE DESIGN MOMENT WAS   1.84                  THE SLENDERNESS RATIO WAS 89.4

MBER NUMBER    3
W14    78.00                                ACTIONS
           AXIALN        SHEARN      MOMENTN      AXIALF        SHEARF       MOMENTF
           -31.0          37.5       .129E+04      31.0          41.8        -.236E+04

        DESIGN EQUATION VALUES
        FORMULA 1.6-2 =    .0451 +    .8432 +  0.0000 =    .8883
--- THIS MEMBER HAS AXIAL TENSION.   FORMULA 1.6-1B WAS ACTUALLY CHECKED.
        THE DESIGN MOMENT WAS   .250E+04              THE SLENDERNESS RATIO WAS100.0

        BEAM DEFLECTION
        THIS BEAM DEFLECTED   .7068 INCHES MAXIMUM AT 150.0 INCHES FROM THE LEFT
        THE SPAN TO DEFLECTION RATIO WAS  424.

MBER NUMBER    4
W14    22.00                                ACTIONS
           AXIALN        SHEARN      MOMENTN      AXIALF        SHEARF       MOMENTF
            79.1        .206E-08     .254E-20     -78.8        -.206E-08     .309E-06

        DESIGN EQUATION VALUES
        FORMULA 1.6-1A =    .7515 +    .0000 +  0.0000 =    .7515
        FORMULA 1.6-1B =    .5639 +    .0000 +  0.0000 =    .5639
        THE DESIGN MOMENT WAS   .309E-06              THE SLENDERNESS RATIO WAS 72.1
```

Fig. A.9 Member sizes, actions, design values and beam deflections from the initial run.

```
1BER NUMBER    5
 W14   26.00                                             ACTIONS
             AXIALN          SHEARN        MOMENTN      AXIALF          SHEARF        MOMENTF
             -14.5           14.9          763.         14.5            15.7          -884.

          DESIGN EQUATION VALUES
          FORMULA 1.6-2 =    .0632 +    .8391 +  0.0000 =    .9024
 --- THIS MEMBER HAS AXIAL TENSION.  FORMULA 1.6-1B WAS ACTUALLY CHECKED.
          THE DESIGN MOMENT WAS  884.                  THE SLENDERNESS RATIO WAS 53.2

          BEAM DEFLECTION
          THIS BEAM DEFLECTED  .2139 INCHES MAXIMUM AT 150.0 INCHES FROM THE LEFT
          THE SPAN TO DEFLECTION RATIO WAS 1403.


1BER NUMBER    6
 W14   22.00                                             ACTIONS
             AXIALN          SHEARN        MOMENTN      AXIALF          SHEARF        MOMENTF
              61.2           -.962         0.           -60.9           .962          -144.

          DESIGN EQUATION VALUES
          FORMULA 1.6-1A =     .5818 +    .2128 +  0.0000 =    .7946
          FORMULA 1.6-1B =     .4366 +    .2379 +  0.0000 =    .6745
          THE DESIGN MOMENT WAS  144.                  THE SLENDERNESS RATIO WAS 72.1


1BER NUMBER    7
 W14   43.00                                             ACTIONS
             AXIALN          SHEARN        MOMENTN      AXIALF          SHEARF        MOMENTF
              22.1           16.4          .159E+04     -21.5           -16.4         870.

          DESIGN EQUATION VALUES
          FORMULA 1.6-2 =     .1136 +  1.0582 +  0.0000 =  1.1718
          THE DESIGN MOMENT WAS  .159E+04               THE SLENDERNESS RATIO WAS 79.4


1BER NUMBER    8
 W18   50.00                                             ACTIONS
             AXIALN          SHEARN        MOMENTN      AXIALF          SHEARF        MOMENTF
             .729E-01        7.48          -582.        -.729E-01       23.8          -.186E+04

          DESIGN EQUATION VALUES
          FORMULA 1.6-2 =     .0003 +    .9497 +  0.0000 =    .9500
          THE DESIGN MOMENT WAS  .186E+04               THE SLENDERNESS RATIO WAS 40.7

          BEAM DEFLECTION
          THIS BEAM DEFLECTED  .1907 INCHES MAXIMUM AT 120.0 INCHES FROM THE LEFT
          THE SPAN TO DEFLECTION RATIO WAS 1573.
```

Fig. A.9 (continued)

```
MBER NUMBER    9
 W14   43.00                                      ACTIONS
          AXIALN        SHEARN       MOMENTN      AXIALF        SHEARF       MOMENTF
           45.2          13.6        .103E+04     -44.7         -13.6        .101E+04

        DESIGN EQUATION VALUES
        FORMULA 1.6-1A =    .2326 +    .6845 +   0.0000 =    .9170
        FORMULA 1.6-1B =    .1661 +    .6831 +   0.0000 =    .8492
        THE DESIGN MOMENT WAS  .103E+04          THE SLENDERNESS RATIO WAS 79.4

MBER NUMBER   10
 W14   43.00                                      ACTIONS
          AXIALN        SHEARN       MOMENTN      AXIALF        SHEARF       MOMENTF
           14.1          -3.51        -287.        -13.5         3.51         -239.

        DESIGN EQUATION VALUES
        FORMULA 1.6-2 =    .0723 +    .1910 +   0.0000 =    .2633
        THE DESIGN MOMENT WAS  287.              THE SLENDERNESS RATIO WAS 79.4

MBER NUMBER   11
 W18   35.00                                      ACTIONS
          AXIALN        SHEARN       MOMENTN      AXIALF        SHEARF       MOMENTF
           13.5          13.5         239.         -13.5         20.4        -.117E+04

        DESIGN EQUATION VALUES
        FORMULA 1.6-2 =    .0691 +    .9223 +   0.0000 =    .9914
        THE DESIGN MOMENT WAS  .117E+04          THE SLENDERNESS RATIO WAS 42.6

        BEAM DEFLECTION
        THIS BEAM DEFLECTED  .2534 INCHES MAXIMUM AT 150.0 INCHES FROM THE LEFT
        THE SPAN TO DEFLECTION RATIO WAS 1182.

MBER NUMBER   12
 W14   43.00                                      ACTIONS
          AXIALN        SHEARN       MOMENTN      AXIALF        SHEARF       MOMENTF
           20.9          13.5         852.         -20.4         -13.5        .117E+04

        DESIGN EQUATION VALUES
        FORMULA 1.6-2 =    .1075 +    .7807 +   0.0000 =    .8882
        THE DESIGN MOMENT WAS  .117E+04          THE SLENDERNESS RATIO WAS 79.4
```

Fig. A.9 (continued)

allowed to control the size of the pair. The reverse was the case and member 7 had an overstress.

The understressed members were picked because the program could not find a lighter member which had interaction values closer to, but not over 1.03. Table A.2 shows the member selections and results for a second run in which lighter sections were tested for adequacy (IOFLAG=1). These new sections were chosen to be as close in weight to, but lighter than the sections selected in the first run. Improved sections were found for only members 2 and 4. All the other sections tried were overstressed. The program did not pick the two lighter sections because of the lighter section search routine used in the program. This routine is not as effective for lightly loaded columns whose optimum section lies below the nominal depth of the adequate section the program currently has.

The final check of the final member sizes is also shown in Table A.2.

A.8 Storage Adjustments

If more (or less) storage is required for the program, only the following statements need to be changed:

Common Blocks and Dimension Statements

ARRAYS	VALUES NEEDED
MSIZE, AL, IPIN, CT, ST, NM, JC, KC, IBC, X, Y, DL, SETSIZ, MOSIZE, FYIELD, BRCEX, BRCEY, YMFE, YMNE, IEQUIV	(Number of Members)
AJ, DJ, AJO, FIXYZ	3*NJ (Number of Joints)
AMF	6*NM
S	(3*NJ, Bandwidth)
/LOADS/ Common Block =	Number of Member Loadings.

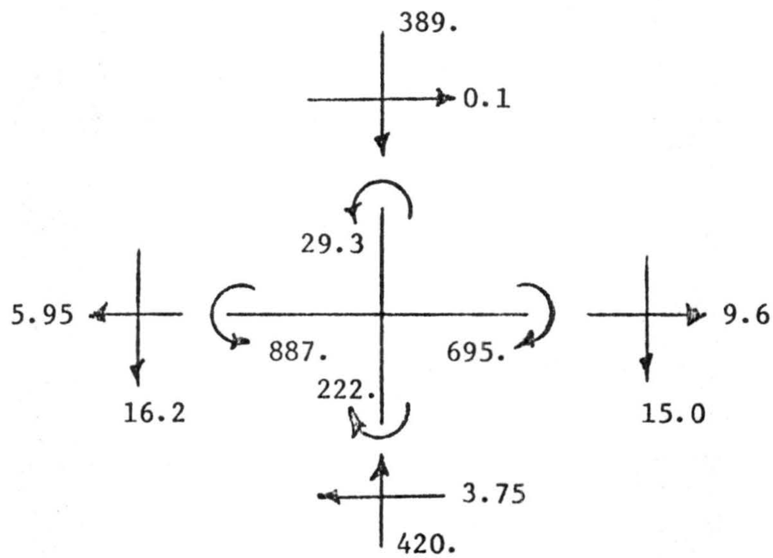
Table A.2 Design example summary.

Member Number	Run 1		Run 2		Run 3	
	Section	Inter-action value	Section	Inter-action value	Section	Inter-action value
1	W21 x 68	.91	W24 x 61	1.44	W21 x 68	.90
2	W 8 x 10	.72	W 6 x 8.5	.80	W 6 x 8.5	.80
3	W14 x 78	.89	W14 x 74	1.04	W14 x 78	.88
4	W14 x 22	.75	W12 x 19	1.01	W12 x 19	1.02
5	W14 x 26	.90	W14 x 22	1.09	W14 x 26	.91
6	W14 x 22	.79	W12 x 19	1.17	W14 x 22	.78
7	W14 x 43	1.17	W14 x 53	.99	W14 x 53	.98
8	W18 x 50	.95	W18 x 50	.94	W18 x 50	.94
9	W14 x 43	.92	W14 x 53	.71	W14 x 53	.72
10	W14 x 43	.26	W14 x 38	.33	W14 x 43	.25
11	W18 x 35	.99	W18 x 35	.97	W18 x 35	.99
12	W14 x 43	.89	W14 x 38	1.20	W14 x 43	.88

A.9 Numerical Error Analysis

Checks were made of the accuracy of the analysis. Two methods were used: statics and the method of residual corrections.

Static checks were made by constructing a free body of a joint or member and checking the requirements of statics. The example below is a check of an interior joint of the multi-story X braced frame:



$$F_H = 9.6 + 0.1 - 3.75 - 5.95 = 0.0$$

$$F_V = 420. - 16.2 - 15.0 - 389. = -0.2$$

$$M = 887. + 29.3 - 695. - 222. = -0.7$$

The small discrepancies in F_V and M are insignificant and probably have occurred because only three place accuracy was used in the computer printout.

The method of residual corrections is a more sophisticated technique which can give an accurate idea of the round off errors which occur during the analysis. The set of stiffness equations:

$$\text{(single precision)} \qquad S * DJ = AJ \tag{A.8}$$

is set up and solved using single precision operations. The DJ (unknown) matrix is then printed out. Then, using double precision operations, the DJ matrix is read back in and and S and AJ matrices are formed again. The S matrix is multiplied by DJ:

$$\text{(double precision)} \qquad S * DJ = AJ_e \qquad \text{(A.9)}$$

to form AJ_e. If the solution obtained from Equation (A.8) contains no round off error, then AJ_e will equal AJ. However, computing the simple difference between AJ_e and AJ would not show how much error the round off has caused in the DJ matrix. This can be found by solving:

$$\text{(double precision)} \qquad S * E = AJ - AJ_e \qquad \text{(A.10)}.$$

Of course, the E matrix should be very small.

This test was run on the rigid frame with a wind loading. The maximum round off error found was less than one percent of the corresponding term of the DJ matrix, proving that round off error is not significant.

A.10 Possible Improvements in DESIGN1

Throughout the use of DESIGN1, several inadequacies have been noticed and improvements have been suggested. They are discussed below.

A.10.1 Minor Axis Orientation

A useful addition to the program would be a provision which would allow a member to be declared MINOR, which would cause the program to treat the minor axis as the in-plane exis of the member. The only modification in the analysis loop would occur in FORMSM. When MINOR was declared, the subroutine would have to be instructed to find and use I_y rather than I_x. Subroutine EFLEN would also need this modification.

A.10.2 Pinned-Fixed Members

In its present form, DESIGN1 is only capable of handling pinned-pinned or fixed-fixed end conditions of a member. The analysis routine was originally for only fixed-fixed members, pinned-pinned members were introduced by reducing the moment of inertia to near zero (see Section A.3.3.2). Creating a pinned-fixed capability would be somewhat more difficult. Subroutine FORMSM would have to be modified to form the member's pinned-fixed 6 x 6 stiffness matrix, instead of the fixed-fixed or pinned-pinned matrix. IPIN=2 could be the flag for this. Modifications would also have to be made in subroutines FIXEND and WTFEA to allow them to compute pinned-fixed end actions, and in EFLEN to modify girder stiffnesses for known end conditions.

A.10.3 Variable and Non-I Shape Sections

Presently, DESIGN1 can only work with standard AISC rolled shapes. Plate girders, box columns, haunches, and other useful structural modifications are not allowed. Additional provisions for the design of these types of members would be extensive. However, it could be made possible to input the entire member stiffness matrix directly into the analysis routine, bypassing FORMSM for the member. The design loop would also have to be bypassed for this particular member. The user could also be given the option of adding to Table A.1. Required section input would be area, I_x, I_y, etc. SETSIZ would have to be used on these members.

A.10.4 Moment Reductions to Column Faces

DESIGN1 now assumes that the members are lines, and no reductions are made in the moments to allow for the finite size of connections. Subroutine PVM could easily be modified to reduce beam moments to the column faces. It would only be necessary to find the appropriate column depth in the member properties, and interpolate to the column face.

A.10.5 Allowable Stresses in Bracing

One obvious deficiency in DESIGN1 is the inability of the program to allow higher axial stresses in bracing elements of the structure (AISC equation 1.5-3). Only a few modifications would be required in the computation of allowable axial stresses to accomodate this.

A.10. 6 Drift Limitations

With extensive modifications of the design loop DESIGN1 could be used to automatically stiffen frames to reduce wind drift. The frame could be split into sections, such as upper beams, middle interior columns or lower bracing, and the best section could be found and stiffened. This process would be repeated in steps until the frame meets the drift requirements.

A.10.7 Improved Trial Member Selection Procedure.

Improvements could be made in subroutine SIZE to help it pick better trial member sizes. The routine for member selection should be rapid, because the analysis-design load is the best routine for member size selection. Any computational technique which required a significant amount of computer time (as compared to the analysis-design loop) would be inefficient.

APPENDIX B

Appendix B

This appendix provides a listing and a brief description of Program DESIGN1.

B.1 The Main Program

Line	Content of Lines
001 - 032	Storage assignments, initialization and descriptive statements.
033 - 074	Read input data and write input echo.
075 - 083	Compute member lengths, sines, cosines.
084 - 098	Zero some matrices as necessary.
099 - 145	Read in or internally select trial member sizes; read in, echo, and store member and joint loadings.
146 - 157	Compute and store member self-weight fixed end actions.
158 - 166	Begin analysis loop, zero stiffness matrix.
167 - 198	Form the stiffness matrix.
199 - 207	Modify the stiffness matrix for boundary conditions.
208 - 209	Solve for the joint displacements.
210 - 224	Compute member end actions.
225 - 236	Check maximum drift deflection.
237 - 249	Begin design loop.
250 - 258	Compute effective lengths.
259 - 266	Compute f_a, end moment ratio, and check to see if member is in compression or tension.
267 - 274	Compute the design loadings.
275 - 283	Initialize nominal depth indicator, II.
284 - 295	Decide whether or not to pick section from the economy tables; if so, choose initial economy table shape.
296 - 322	Check depth limits.

Line	Content of Lines
323 - 344	Initialize section properties.
345 - 364	Compute F_a, f_a/F_a.
365 - 378	Compute C_m.
379 - 434	Compute F_{bx}, F_{by}, f_{bx}, f_{by}.
435 - 458	Compute interaction equations.
459 - 480	Output section.
481 - 484	Test the interaction equations.
485 - 503	Change member size.
504 - 524	Search for a better section.
525 - 538	If member size has changed, trip flags and adjust member self-weight loadings.
539 - 550	Adjust equivalencies for member self-weight loading.
551 - 554	Control.
555 - 568	Change AMF matrix back to pre-analysis values.
569 - 580	Control.
581 - 597	Output.
598 - 664	Format statements.
665 - 666	Terminal statements.

B.2 DESIGN1 Subroutines

Subroutine Name	Description
SETUM	Called after the number of iterations set by the variable INTS to examine the member size change pattern. If an oscillation is present, this subroutine will set the member size.
DEFLET	Called during the output section to compute deflections along a beam span.
WTFEA	Computes the member self-weight fixed end actions.

Subroutine Name	Description
SIZE	Selects an initial member trial size if called. Selection is based on span to depth ratios.
FORMRT	Forms the member rotation-translation matrix which relates structural and member coordinate systems.
FORMSM	Forms the member stiffness matrix in member coordinates.
MULT	Called to multiply two matrices together.
INDEX	Called to index members to their corresponding terms in the S, AJ and DJ matrices.
BNDSOLV	Called to perform a Gauss elimination on the symmetrical, banded stiffness matrix to find the joint displacements.
FBCB	Called if Section 1.5.1.4.6a of the AISC code is applicable. Returns an allowable bending stress between $0.6F_y$ and $0.66F_y$.
FIXEND	Computes the fixed end moments for member loadings.
EFLEN	Computes the effective length factor for columns.
PVM	Computes the design actions for the design loop.
STORE	Takes the FEA matrix and stores it in the AJ and AMF matrices.
C15141	Checks the compactness of beams.
TABLEC	Called during the computation of C_m to select the appropriate case from Table C.1.6.1.2 of the AISC code.

B.3 Listing of Program DESIGN1

```
      PROGRAM DESIGN1
     1(INPUT,OUTPUT,TAPE5=INPUT,TAPE6=OUTPUT,PUNCH)
C  THIS PROGRAM HAS 3 BASIC SECTIONS... INPUT, ANALYSIS, AND DESIGN
C
C  DIMENSION AND COMMON STATEMENTS
      COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
      COMMON/ESM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
      COMMON/RT/RT(6,6), CT(208),ST(208),RTT(6,6)
      COMMON/LOADS/MEM(80), JCODE(80), PLOAD(80), PN(80), RF(80)
      COMMON/SIMEQ/S(312,30), AJ(312), DJ(312), AJO(312)
      COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
      COMMON/PV/M, MLOADS, AMX, PZ, MOM(9), X(208),NINTEN
      COMMON/EFL/Y(208),    EK, NM, SWAY, PIE
      COMMON/LIMS/DL(208),SETSIZ(208),MOSIZE(208),INT, INTS
      COMMON/ECON/KK(16), KJ(16), NN(16), CWT(213)
      COMMON/COMP/FA, FYIELD(208),CDEP(213), CTW(213), CBF(213), CDAF(21
     13), BRCEX(208),CTF(213)
      DIMENSION SMR(6,6), D(6), TEMP(6,6)
      DIMENSION CSECT(213), CSXX(213), CPXX(213), CSYY(213), CRYY(213),
     1CPT(213), IECON(54)
      DIMENSION BRCEY(208), YMNE(208), YMFE(208), IEQUIV(208)
C
      INTEGER DL, SETSIZ
      REAL IX, MOM
      REAL MRX, MRY, KLX, KLY, L
      PIE = 3.1415926536
      INT = 0
      YMAX = 0.0
C
C  READ IN DATA
C
C  GEOMETRY
      READ (5,1) NJ, NM, E, SWAY, INTS, IOFLAG, IGRAV, IPFLAG, INOUT,
     1 DEFFL, MAXBAND
      IF (IPFLAG.GT.3) IPFLAG = 3
      IF (INTS.EQ.0) INTS = 5
      IF (DEFFL.LE.0.0) DEFFL = 500.0
      IF (INOUT.NE.0) GO TO 701
      WRITE (6,50) NJ, NM, E, SWAY, INTS
      IF (IGRAV.EQ.1) WRITE (6,77)
      IF (IGRAV.EQ.2) WRITE (6,78)
      WRITE (6,51)
  701 DO 100 I=1, NJ
      I3 = I * 3
      READ (5,2) JN, Y(JN), X(JN), FIXYZ(I3-2), FIXYZ(I3-1), FIXYZ(I3)
      IF (Y(JN).GT.YMAX) YMAX = Y(JN)
      IF (INOUT.NE.0) GO TO 100
      WRITE(6,52)JN, X(JN), Y(JN), FIXYZ(I3-2), FIXYZ(I3-1), FIXYZ(I3)
  100 CONTINUE
C
C  READ IN MEMBER INDICES, PROPERTIES --
C
      IF (INOUT.NE.0) GO TO 702
      WRITE (6,53)
  702 DO 101 I=1, NM
```

```
      READ (5,3)  M, JC(M), KC(M), FYIELD(M), BRCEX(M), BRCEY(M), DL(M),
     1YMNF(M), YMFE(M), IBC(M), IPIN(M), SETSIZ(M)
      IF (IBC(M).EQ.1.AND.DL(M).EQ.0) DL(M) = 14
      IF (DL(M).EQ.0) DL(M) = 36
      IF (FYIELD(M).EQ.0.0) FYIELD(M) = 36.0
      IF (INOUT.NE.0) GO TO 101
      WRITE (6,54)M, JC(M), KC(M), FYIELD(M), BRCEX(M), BRCEY(M), DL(M),
     1YMNF(M), YMFE(M), IBC(M), IPIN(M), SETSIZ(M)
  101 CONTINUE
C
C  READ IN AISC SHAPES, ECONOMY TABLE REFERENCES
C
      READ (5,5) (KK(I), KJ(I), NN(I),I=1,16)
      DO 102 I=1, 213
      READ (5,4) CSECT(I), CWT(I), AREA(I), CDEP(I), CBF(I), CTF(I),
     1CTW(I), CDAF(I), DUM3,  IX(I), CSXX(I), CRXX(I), DUM1,
     2CSYY(I), CRYY(I),  DUM4,   DUM2, CRT(I)
  102 CONTINUE
      READ (5,5)(IECOM(I),I=1,54)
C  COMPUTE SIN  COS  LENGTHS OF MEMBERS.
      DO 107 I=1, NM
      JJ = JC(I)
      KL = KC(I)
      XL = X(KL) - X(JJ)
      YL = Y(KL) - Y(JJ)
      AL(I) = SQRT(XL*XL + YL*YL)
      ST(I) = YL/AL(I)
  107 CT(I) = XL/AL(I)
C  ZERO SOME MATRICIES
      N6 = NM*6
      NJ3 = NJ*3
      DO 108 I=1,N6
  108 AMF(I) = 0.0
      DO 109 I=1, NJ3
  109 AJQ(I) = 0.0
      DO 923 I=1, NM
      IEQUIV(I) = 0
      IF (SETSIZ(I).LE.0) GO TO 111
      MSIZE(I) = SETSIZ(I)
      SETSIZ(I) = SETSIZ(I) + 426
      GO TO 923
  111 MSIZE(I) = 0
  923 CONTINUE
C  THE NEXT STATEMENT DETERMINES WHETHER THE PROGRAM SIZES THE MEMBERS
C  OR THE TRIAL SIZES ARE READ IN
      READ (5,6) MEMTRI
      IF (MEMTRI.EQ.0) GO TO 103
      DO 104 I=1, NM
      READ(5,75) M, MSIZE(M), IEQUIV(M)
  104 IF (SETSIZ(M).GT.0) MSIZE(M) = SETSIZ(M) - 426
C  READ IN MEMBER LOADINGS, COMPUTE FEAS, SIZE MEMBERS, BUILD AJ
C  FIXEND SUBROUTINE WRITTEN BT GORDON PENFOLD, THANKS GORDON
  103 READ (5,8) MLOADS
      IF (MLOADS.EQ.0) GO TO 117
      IF (INOUT.NE.0) GO TO 703
```

```
      WRITE (6,57) MLOADS
  703 DO 105 I=1, MLOADS
      READ (5,9) MEM(I), JCODE(I), RLOAD(I), RN(I), RF(I)
      IF (IGRAV.EQ.2) RLOAD(I) = RLOAD(I) * 0.75
      M = MEM(I)
      IF (INOUT.NE.0) GO TO 704
      WRITE (6,58) MEM(I), JCODE(I), RLOAD(I), RN(I), RF(I)
  704 CALL FIXEND (M,I)
  105 CALL STORE (M)
C
C  READ IN JOINT LOADS
C
  117 READ (5,6) JLOADS
      IF (JLOADS.EQ.0) GO TO 113
      IF (INOUT.NE.0) GO TO 705
      WRITE (6,59) JLOADS
  705 DO 115 I=1, JLOADS
      READ (5,10)  JN, (FEA(J),J=1,3)
      J3 = JN * 3
      IN(1) = J3 - 2
      IN(2) = J3 - 1
      IN(3) = J3
      DO 110 IA=1, 3
      IF (IGRAV.EQ.2) FEA(IA) = FEA(IA) * 0.75
      MA = IN(IA)
  110 AJO(MA) = AJO(MA) + FEA(IA)
      IF (INOUT.NE.0) GO TO 115
      WRITE (6,60) JN, (FEA(J),J=1,3)
  115 CONTINUE
C
C  SIZE THE MEMBERS IF IT HASNT BEEN DONE ALREADY
  113 DO 114 I=1, NM
      IF (MSIZE(I).NE.0) GO TO 114
      CALL SIZE(I)
  114 CONTINUE
      IF (INOUT.NE.0) GO TO 706
  112 WRITE (6,55)
C  SET UP FOR MEMBER WT FEAS, COMPUTATIONS
  706 DO 118 I=1, NM
      IF (IEQUIV(I).GT.0) SETSIZ(I) = MSIZE(I) + 426
      MOSIZE(I) = 1
      M = MSIZE(I)
      IF (INOUT.NE.0) GO TO 707
      WRITE (6,56) I, CSECT(M), CWT(M), IEQUIV(I)
  707 CALL WTFEA (I,CWT(M),1.0)
  118 CALL STORE(I)
C  END INPUT SECTION
C  ANALYSIS SECTION
C  ZERO S MATRIX
  122 IBAND = 0
      BIG = 1.0E+60
  814 INT = INT + 1
      DO 816 I=1, NJ3
      AJ(I) = AJO(I)
      DO 816 J=1, MAXBAND
```

```
  816 S(I,J) = 0.0
C
C  FORM THE S MATRIX
C
      DO 124 I=1, NM
C  FORM RT, RTT MATRICES
      CALL FORMRT(I)
C  FORM THE SM MATRIX
      NSIZE = MSIZE(I)
      IF (SETSIZ(I).LT.214) SETSIZ(I) = MOSIZE(I)
      MOSIZE(I) = MSIZE(I)
      CALL FORMSM(I,NSIZE)
C
C  COMPUTE SMR = RTT*SM*RT
C
      CALL MULT (RTT,6,6,SM,6,6,TEMP)
      CALL MULT (TEMP,6,6,RT,6,6,SMR)
C
C  STORE SMR IN S
C
      CALL INDEX(I)
      DO 126 J=1, 6
      DO 125 K=1, 6
      JJ = IN(J)
      KL = IN(K)
      IF (JJ.GT.KL) GO TO 126
      KL = KL - JJ + 1
C  TEST FOR MAXIMUM KK TO GET IBAND
      IF (KL.GT.IBAND) IBAND = KL
      IF (IBAND.LE.MAXBAND) GO TO 128
      WRITE (6,61)
      GO TO 10000
  128 CONTINUE
C  MODIFY S FOR BOUNDRY CONDITIONS
      IF (KL.GT.1) GO TO 132
      IF (FIXYZ(JJ).EQ.0.0) GO TO 132
      S(JJ,1) = BIG
      AJ(JJ) = 0.0
  132 CONTINUE
      S(JJ,KL) = S(JJ,KL) + SMR(J,K)
  126 CONTINUE
  124 CONTINUE
C  SOLVE A = S*D
      CALL BNDSOLV   (NJ,IBAND)
C  FORM AME = SM*RT*DJ + FEA
      DO 134 I=1, NM
      CALL INDEX(I)
      DO 136 IA=1, 6
      M = IN(IA)
  136 D(IA) = DJ(M)
      NSIZE = MSIZE(I)
      CALL FORMSM(I, NSIZE)
      CALL FORMRT(I)
      CALL MULT (SM,6,6,RT,6,6,TEMP)
      CALL MULT (TEMP,6,6,D,6,1,AM1)
```

```
C  ADD AM1 TO AMF
      DO 134 IA=1, 6
      M = 6*(I-1) + IA
  134 AMF(M) = AMF(M) + AM1(IA)
C  CHECK DEFLECTION LIMITS
      IF (INOUT.LT.2) GO TO 712
      DEFFL = 1.0 / DEFFL
C  FIND MAXIMUM X DEFLECTION
      DAF = 0.0
      DO 710 I=1, NJ
      KL = 3 * (I-1) + 1
      IF (DJ(KL).GT.DAF) DAF = DJ(KL)
  710 CONTINUE
      YMAX = DAF/YMAX
      IF (YMAX.LE.DEFFL) INOUT = 1
      IF (INOUT.GE.2) GO TO 822
C
C  DESIGN SECTION HOOK UP
C  SOME OF DESIGN SECTION WRITTEN BT BRIAN PENNER. THANKS BRIAN
  712 ICHAN = 0
C  BEGIN DESIGN LOOP
  804 DO 142 M=1, NM
      ITEN = 0
      INTER = 0
      IF (SETSIZ(M).GT.213.AND.IOFLAG.EQ.0) GO TO 142
      IF (SETSIZ(M).GT.426.AND.IOFLAG.NE.1) GO TO 142
      J = MSIZE(M)
      JO = J
      MEMEX = 0
C  COMPUTE KLX, KLY
C  K=1.0 FOR ALL BEAMS AND PINNED MEMBERS
      EK = 1.0
      IF (IBC(M).NE.1.OR.IPIN(M).EQ.1) GO TO 144
      IF (ST(M).LT.0.05) GO TO 144
      CALL EFLEN (M)
  144 KLX = EK*BRCEX(M)
      KLY = BRCEY(M)
      KL = 6*(M-1)+3
      L2 = KL+ 3
      FA = ABS(AMF(KL+1))/AREA(J)
      IF (AMF(L2).EQ.0.0) AMF(L2) = 0.001
      MRX = AMF(KL)/AMF(L2)
      IF ( ABS(MRX) .GT. 1.0 ) MRX = 1.0/MRX
      KL = 6*(M-1)+3
      L2 = KL+ 3
      IF (AMF(KL-2).LT.0.0) ITEN = 1
C  COMPUTE MAXIMUM MOMENTS, AXIAL LOADS
C  P IS ASSUMED CONSTANT FOR ONE MEMBER
      NINTEN = 0
      IF(IBC(M).NE.0) GO TO 153
      IF (IGRAV.NE.1) GO TO 153
      CALL C1514)
  153 CALL PVM (CWT(J),AMXA)
      CMX = 0.85     $     CMY = 0.85
      II = 1
```

```
150 IF (J.GT.KJ(II)) II=II+1
    IF (J.GT.KJ(II)) GO TO 150
152 WTCHECK = 1000.0
    JM = 0
    JP = 0
    IFLAG = 0
    JFLAG = 0
    FA = PZ/AREA(J)
    IF (IBC(M).NE.0.OR.FA/(FYIELD(M)*0.6).GT.0.15) GO TO 156
C  WANT TO PICK FROM ECONOMY TABLES
    IF (IOFLAG.EQ.1) GO TO 156
    JFLAG = 1
    CKWT = CWT(J)
    CKDP = CDEP(J)
    DO 151 JJ=1,54
    IZ = IECON(JJ)
    IF (CWT(IZ).LT.CKWT.OR.CDEP(IZ).LT.CKDP) GO TO 151
    J = IZ
    GO TO 156
151 CONTINUE
    MFLAG = 0
156 CONTINUE
    INTER = INTER + 1
    IF (INT.GT.2.AND.INTER.GT.25) GO TO 650
    IF (INTER.GT.50) GO TO 650
    IF (J.GT.0) GO TO 828
    J = 8
    SETSIZ(M) = J + 213
    GO TO 650
828 CONTINUE
    IF (J.EQ.214) J = 122
C  THIS SECTION DEALS WITH DEPTH LIMITS
    IF (IOFLAG.EQ.1) GO TO 157
    IF (J.GT.NN(II)) II=II+1
    IF (J.LT.KJ(II)) II = II - 1
    IF (DL(M).GE.KK(II)) GO TO 157
    IF(MEMEX.EQ.0) GO TO 161
    J = MEMEX
    GO TO 650
161 IF (JFLAG.EQ.1) GO TO 163
    WRITE (6,68) M
    DL(M) = 36
    GO TO 157
163 JFLAG = 0
    II = II - 1
    J = KJ(II)
    IBC(M) = 1
157 KXFLAG = 0
    KYFLAG = 0
    SECT = CSECT(J)
    WT = CWT(J)
    AZ = AREA(J)
    DEP = CDEP(J)
    BF = CBF(J)
    TF = CTF(J)
```

```
      TW = CTW(J)
      DAF = CDAF(J)
      SXX = CSXX(J)
      RXX = CRXX(J)
      SYY = CSYY(J)
      RYY = CRYY(J)
      FY = FYIELD(M)
      L = BRCEY(M)
      IF ( WT .GT. WTCHECK ) GO TO 420
      GO TO 160
  158 IF(JJ .EQ. JM .OR. JJ .EQ.JP) GO TO 300
      J = IECON(JJ)
      GO TO 156
  160 FA = PZ/AZ
      SRX = KLX/RXX
      SRY = KLY/RYY
      IF (IOFLAG.EQ.1) GO TO 145
C  CHECK SLENDERNESS RATIOS
      IF (SRX.GT.200.0.OR.SRY.GT.200.0) GO TO 200
  145 CONTINUE
      IF (SRX.EQ.0.0) SRX = 0.001
      IF (SRY.EQ.0.0) SRY = 0.001
C     NOW COMPUTING ALLOWABLE COMPRESSION LOAD. EQ. 1.5-1 STEEL HANDBOOK.
      DUM2 = SRX
      IF ( SRX .LT. SRY ) SRX = SRY
      CC = SQRT (2. *PIE**2 * E/FY)
      CON = SRX/CC
      IF ( SRX .GT. CC ) GO TO 162
      FS = 5./3. + 0.375*CON - 0.125*(CON**3)
      FALL = FY*(1.-0.5*(CON**2))/FS
      GO TO 164
C     NOW COMPUTING THE ALLOWABLE COMPRESSION LOAD. EQ. 1.5-2 STEEL HANDBOOK.
  162 FALL = (12.0 * PIE**2 * E )/( 23.0 * SRX**2 )
  164 R1 = FA/FALL
C  COMPUTE CMX, CMY
      IF (INTER.NE.1) GO TO 718
      KL = JC(M)
      L2 = KC(M)
      IF (Y(KL).GT.SWAY.OR.Y(L2).GT.SWAY) GO TO 718
      CMX = 0.6 - 0.4*MRX
      IF (CMX.LT.0.4) CMX = 0.4
      DO 713 IA=1, MLOADS
      IF (MEM(IA).NE.M) GO TO 713
      GO TO (716,716,716,713,713,713,716) JCODE(IA)
  716 CALL TABLEC (DUM1)
      CMX = 1.0 + DUM1 * FA / ((12.0*PIE**2*E)/(23.0*(DUM2**2)))
      GO TO 718
  713 CONTINUE
C     THE PROGRAM IN NOW MAKING A PRELIMINARY CHECK OF EQUATIONS 1.6-1B
C     AND 1.6-2 IN THE AISC STEEL HANDBOOK.
  718 IF ( R1 .GT. 1.03 . AND . IOFLAG .NE . 1  ) GO TO 200
      CKWT = ABS(YMNE(M))
      CKDP = ABS(YMFE(M))
      AMY = AMAX1 (CKWT,CKDP)
      IFBXCHE = 0.66 * FY + 0.5
```

```
      FBXCHEC = IFBXCHE
      R2TEST = ( AMX/SXX )/FBXCHEC
      IFBYCHE = 0.75 * FY + 0.5
      FBYCHEC = IFBYCHE
      R3TEST = (AMY/SYY )/FBYCHEC
      TEST = R2TEST + R3TEST
      IF ( TEST .GT. 1.03 . AND . IOFLAG. NE . 1 ) GO TO 200
      FBX = 0.0
      IFBTRI = 0.60 * FY + 0.5
      FBTRI = IFBTRI
      BOT = 0.5*CBF(J)/CTF(J)
      CALL C15141
      IF (NINTEN.NE.1) GO TO 170
      IF ( FY .GE. 90.0 ) GO TO 172
      IFBX = 0.66 * FY + 0.5
      FBX = IFBX
      GO TO 173
  170 IF (NINTEN.NE.2) GO TO 173
      ABOT = 95.0/SQRT (FY)
      IF (ABOT .LT. BOT ) GO TO 173
      IF ( FY .GE. 90.0 ) GO TO 172
C     NOW COMPUTING THE ALLOWABLE BENDING STRESS, 1.5-5 STEEL HANDBOOK.
      IFBX = FY * ( 0.733 - 0.0014 * ( 0.5 * CBF(J)/CTF(J))*SQRT(FY))+.5
      FBX = IFBX
      GO TO 173
  172 FBX = FBTRI
  173 IF (BRCEX(M)) 174, 175, 174
C   WE ARE COMPUTING THE ALLOWABLE BENDING STRESS IN
C     THE MINOR AXIS, SECTION 1.5.1.4.3 OF THE STEEL HANDBOOK.
  174 ABOT = 52.2/SQRT (FY)
      IF ( BOT .GT. ABOT ) GO TO 175
      IF ( KLY/RYY .GT. KLX/RXX ) GO TO 175
      IFBY = 0.75 * FY + 0.5
      FBY = IFBY
      IF ( FBX .GT. 0.0 ) GO TO 181
  175 IF ( FBX .GT. 0.0 ) GO TO 178
      IF (AMX.GT.ABS(AMF(L2)).OR.AMX.GT.ABS(AMF(KL))) MRX = 1.0
      CBX = 1.75 - 1.05 * MRX + 0.3 * (MRX**2)
      IF ( CBX .GT. 2.3 ) CBX = 2.3
      CALL FBCB ( CBX, FY, L, CRT(J),DAF,FBTRI,FBX,KXFLAG)
  178 IF (YMNE(M).EQ.0.0.OR.YMFE(M).EQ.0.0) GO TO 179
      MRY = YMFE(M)/YMNE(M)
      IF ( ABS(MRY) .LT. 1.0 ) MRY = 1.0/MRY
      IF (AMY.GT.ABS(YMFE(M)).OR. AMY.GT.ABS(YMNE(M))) MYR = 1.0
      CBY = 1.75 - 1.05 * MRY + 0.3 * ( MRY**2 )
      GO TO 180
  179 CBY = 1.75
  180 IF ( CBY .GT. 2.3 ) CBY = 2.3
      CALL FBCB ( CBY, FY, L, CRT(J),DAF,FBTRI,FBY,KYFLAG)
  181 R2 = (AMX/SXX)/FBX
      R3 = (AMY/SYY)/FBY
      IF (R1.GT.0.15.AND.ITEN.NE.1) GO TO 182
      IF (ITEN.EQ.1) R1 = FA/(0.6*FY)
      IFLAG = 1
C   THE NEXT CARD COMPUTES EQUATION 1.6-2 OF THE STEEL HANDBOOK.
```

```
      SIX = R1 + R2 + R3
      IF (IOFLAG.EQ.1) GO TO 808
      IF(SIX .GT. 1.03) GO TO 200
      IF (IOFLAG.EQ.2) GO TO 650
      IF ( SIX .LT. 0.95 ) GO TO 199
      GO TO 300
  182 R4 = FA/(0.6*FY)
C  THE NEXT CARD COMPUTES EQUATION 1.6-1B OF THE STEEL HANDBOOK.
      B7 = R2 + R3 + R4
      FEX = 149000.0/(DUM2**2)
      CBX = 1.0
      IF ( KXFLAG .EQ. 1 ) CALL FBCB ( CBX, FY, L,CRT(J),DAF,FBTRI,FBX,
     1DUM1)
      R21 = (CMX* (AMXA/SXX)/FBX)/(1.0 - FA/FEX)
  183 R31= 0.0
C  THE NEXT CARD COMPUTES EQUATION 1.6-1A OF THE STEEL HANDBOOK.
  184 A7 = R1 + R21 + R31
      IF (IOFLAG.NE.1) GO TO 806
C  PART OF OUTPUT SECTION
  808 KL = 6 * (M-1) + 1
      L2 = KL + 5
      IF (IFLAG.NE.1) GO TO 810
      WRITE (6,63) M, SECT, WT, (AMF(I),I=KL,L2), R1, R2, R3, SIX
      IF (ITEN.EQ.1) WRITE (6,69)
      GO TO 812
  810 WRITE (6,64) M, SECT, WT, (AMF(I),I=KL,L2), R1, R21, R31, A7, R4,
     1R2, R3, B7
C  NOW COMPUTE DEFLECTIONS FOR THE SPAN
  812 IF (SRX.GT.SRY) SRY = SRX
      WRITE (6,70) AMX, SRY
      IF (ABS(ST(M)).GT.0.1) GO TO 813
      CALL DEFLET
C  SHEAR CHECK
  813 A7 = 0.4*FY
      B7 = ABS(AMF(KL+4))
      IF (ABS(AMF(KL+1)).GT.B7) B7 = ABS(AMF(KL+1))
      B7 = B7/(DEP*TW)
      IF (B7.LE.A7) GO TO 142
      WRITE (6,65) A7, B7
      GO TO 142
  806 IF ( B7 .GT. 1.03 .OR. A7 .GT. 1.03 ) GO TO 200
      IF (IOFLAG.EQ.2) GO TO 650
      IF ( B7 .LT. 0.93 .AND.A7 .LT. 0.93 ) GO TO 199
      GO TO 300
C  THE NEXT CARDS CHANGE THE SHAPE TO A BETTER ONE
  199 IF (INT.GT.INTS) CALL SETUM (-1, J, M, NM, II)
      IF (JFLAG.EQ.1) GO TO 1991
      JP = J
      J = J - 1
      GO TO 304
 1991 JM = JJ
      JJ = JJ - 1
      GO TO 158
  200 IF (IOFLAG.EQ.2) GO TO 201
      IF (INT.GT.INTS) CALL SETUM (1,J,M,NM,II)
```

```
  201 IF (JFLAG.EQ.1) GO TO 2001
      J = J + 1
      GO TO 304
 2001 JP = JJ
      JJ = JJ+1
      GO TO 158
  304 IF (JP.EQ.J) GO TO 300
      GO TO 156
  300 WTCHECK = WT
C  THIS SECTION TESTS THE DEPTH LIMIT AND SEES IF ANY SHAPES EXIST WITH BETTER
C     PROPERTIES
      MEMEX = J
 420  IF ( JFLAG .EQ. 1 ) GO TO 650
      IF (J.GT.KJ(II)+1.OR.MFLAG.EQ.1) GO TO 430
      MFLAG = 1
      J = KJ(II-1)
      II = II - 1
      GO TO 156
  430 IF (KK(II).GE.DL(M)) GO TO 650
      II = II + 1
      K = KJ(II)
  404 IF (CWT(K).GT.WTCHECK) GO TO 405
      IF (K.EQ.NN(II)) GO TO 405
      K = K + 1
      GO TO 404
  405 IF (K.EQ.KJ(II)) GO TO 650
      J = K - 1
      GO TO 156
  650 IF (IPFLAG.NE.0.AND.IPFLAG.NE.2) WRITE (6,71) M, J, JO, CSECT(J),
     1CWT(J), EK, PZ, AMX, CMX, CMY, FALL, FBX
      IF (J.NE.JO) GO TO 141
      IF (SETSIZ(M).NE.JO) GO TO 142
      IF (IOFLAG.NE.0) GO TO 142
      SETSIZ(M) = J + 213
      GO TO 142
  141 ICHAN = ICHAN + 1
C  REMOVE AND REPLACE MEMBER WTS IN FEA AND AJ
      CALL WTFEA (M, CWT(JO), -1.0)
      CALL STORE(M)
      CALL WTFEA (M, CWT(J), 1.0)
      CALL STORE (M)
      MSIZE(M) = J
  142 CONTINUE
      DO 832 I=1, NM
      IF (IEQUIV(I).EQ.0) GO TO 832
      J = IEQUIV(I)
      IF (MSIZE(I).EQ.MSIZE(J)) GO TO 832
      JO = MSIZE(I)
      CALL WTFEA(I,CWT(JO),-1.0)
      CALL STORE(I)
      MSIZE(I) = MSIZE(J)
      J = MSIZE(I)
      CALL WTFEA (I,CWT(J),1.0)
      CALL STORE(I)
  832 CONTINUE
```

```
      IF (IOFLAG.EQ.1) GO TO 822
      IF (IPFLAG.NE.0.AND.IPFLAG.NE.2) WRITE (6,72) ICHAN
      IF (ICHAN.EQ.0) GO TO 1000
C  CHANGE AMF BACK TO AMF BEFORE ACTIONS WERE ADDED
  815 DO 800 I=1, NM
      CALL INDEX (I)
      DO 802 IA=1,6
      M = IN(IA)
  802 D(IA) = DJ(M)
      NSIZE = MOSIZE(I)
      CALL FORMSM (I,NSIZE)
      CALL FORMRT (I)
      CALL MULT (SM,6,6,RT,6,6,TEMP)
      CALL MULT (TEMP,6,6,D,6,1,AM1)
      DO 800 IA=1, 6
      M = 6 * (I-1) + IA
  800 AMF(M) = AMF(M) - AM1(IA)
      GO TO 814
C  WHEN PROGRAM COMES HERE NO CHANGES HAVE BEEN MADE IN THE MEMBER SIZES
C  INTERATION
 1000 DO 1005 I=1, NM
      IF (SETSIZ(I).LT.214) GO TO 815
 1005 CONTINUE
      IF (IOFLAG.EQ.0) GO TO 801
      IOFLAG = 1
      GO TO 821
  801 IOFLAG = 2
      GO TO 815
  821 WRITE (6,62)
      GO TO 804
  822 WRITE (6,66)
      DO 820 I=1, NJ
      KL = 3 * (I-1) + 1
      L2 = KL + 2
  820 WRITE (6,67) I, (DJ(J),J=KL,L2)
      CKWT = 0.0
      DO 830 I=1, NM
      J = MSIZE(I)
  830 CKWT = CKWT + CWT(J)/12.0*(AL(I))
      WRITE (6,76) CKWT
      IF (IPFLAG.LT.2) GO TO 10000
C  PUNCHED OUTPUT. TABLE
      WRITE (6,73)
      WRITE (6,74) (I,CSECT(I),CWT(I),I=1,213)
      DO 826 I=1, NM
      PUNCH 75, I, MSIZE(I)
  826 CONTINUE
    1 FORMAT (2I10,F10.2,F10.2,5I5,F5.0,I5)
    2 FORMAT (I10,2F10.2,3F5.2)
    3 FORMAT (3I5,3F10.2,I5,2F10.2,2I2,I6)
    4 FORMAT (A4, 2F5.2, F4.2, F5.3, 2F4.3, F5.3, I3, F6.1, F6.2, F4.2,
     1 F6.2, F6.3, F4.3, F1.0, F2.0, F4.3)
    5 FORMAT (16I5)
    6 FORMAT (I5)
    7 FORMAT (2I5)
```

```
  8 FORMAT (I10)
  9 FORMAT (2I5,3F10.2)
 10 FORMAT (I10,3F10.2)
 50 FORMAT (1H1,*   THIS PROBLEM HAS*I4* JOINTS AND*I4* MEMBERS. *//*
   1   THE MODULUS OF ELASTICITY EQUALS*F9.2* KSI.                 *//*
   2   THESTRUCTURE IS UNBRACED AGAINST SIDESWAY ABOVE*F9.1       * INC
   3HES.*//*    THE PROBLEM WILL ITERATE* I2* TIMES BEFORE CONVERGENCE
   4 IS FORCED.*/)
 51 FORMAT (1H ,*  JOINT          COORDINATES                    FIXITY *,/
   1,*   NUMBER        X              Y            X            Y           ROT  *,/)
 52 FORMAT (1H ,I5,F12.1,4F10.1)
 53 FORMAT (1H1,*  MEMBER INDICES AND OTHER MEMBER DATA*///,3X* MEMBER
   1   NEAR FAR    YIELD    X BRACE   Y BRACE     DEPTH    OUT OF PLANE
   2MOMENTS*/,3X* NUMBER    END  END    POINT    LENGTH    LENGTH       L
   3IMIT   NEAR END   FAR END    BMCOL  PINNED  SET SIZE  *)
 54 FORMAT (1H ,I6,I7,I5,F11.2,2F10.2,I8,F12.2,F10.2,I8,I7,I10)
 55 FORMAT (1H1,*  TRIAL MEMBER SIZES*,///,*  MEMBER   AISC SHAPE   EQ
   1UIVILANCE*,/)
 56 FORMAT (1H ,I5,6X,A4,F6.2,I8)
 57 FORMAT (1H1,*  MEMBER LOADINGS*,///,* THERE ARE *,I4,* LOADED MEMB
   1ERS.*,//,*  MEMBER     JCODE   MAGNITUDE  R-DIST NEAR  R-DIST FAR*
   2,/)
 58 FORMAT (1H ,I6,I11,F12.3,F9.3,F13.3)
 59 FORMAT(1H ,*  JOINT LOADINGS*,//,*     THERE ARE*,I3,* LOADED JOINT
   1S.*,///,*    JOINT     X FORCE    Y FORCE  Z MOMENT *,/)
 60 FORMAT (1H ,I6,3F10.2)
 61 FORMAT (1H0,*    HEY---- I NEED MORE BAND WIDTH ON THE S MATRIX*)
 62 FORMAT (1H1,*    MEMBER SIZES AND ACTIONS*)
 63 FORMAT (1H0,/ ,* MEMBER NUMBER*I4,/ ,3X,A5,F7.2,    33X*ACTIONS*/,1
   18X,*AXIALN          SHEARN        MOMENTN       AXIALF          SHEARF        MOME
   2NTF*/ ,15X,6G12.3,//,15X*DESIGN EQUATION VALUES*/ ,15X*FORMULA 1.6
   3-2 =*F8.4* +*F8.4* +*F8.4* =*F8.4)
 64 FORMAT (1H0,/ ,* MEMBER NUMBER*I4,/ ,3X,A5,F7.2,    33X*ACTIONS*/,1
   18X,*AXIALN          SHEARN        MOMENTN       AXIALF          SHEARF        MOME
   2NTF*/ ,15X,6G12.3,//,15X*DESIGN EQUATION VALUES*/ ,15X*FORMULA 1.6
   3-1A =*F8.4* +*F8.4* +*F8.4* =*F8.4,/,15X*FORMULA 1.6-1B =*F8.4* +*
   4F8.4* +*F8.4* =*F8.4)
 65 FORMAT(1H ,* --- THIS MEMBER FAILED THE SHEAR CHECK, ALLOWABLE =*,
   1F7.2,*    ACTUAL =*F7.2)
 66 FORMAT (1H1,*    JOINT DEFLECTIONS*,///,* JOINT NO.    XTRANS       YT
   1RANS        ROT*,/)
 67 FORMAT (1H ,I6,2X,3F11.3)
 68 FORMAT (1H0,*    THE DEPTH LIMIT WAS TOO RESTRICTIVE FOR MEMBER*I4*
   1.  IT WAS INCREASED TO 36 INCHES.*)
 69 FORMAT (1H ,*    --- THIS MEMBER HAS AXIAL TENSION.  FORMULA 1.6-1B
   1 WAS ACTUALLY CHECKED.*)
 70 FORMAT (1H ,14X*THE DESIGN MOMENT WAS*G10.3,10X*THE SLENDERNESS RA
   1TIO WAS*F5.1)
 71 FORMAT (1H ,* MEMBER NO*I3*  NEW*I4* OLD*I4*  NEW SHAPE = *A4,F6.2
   1,10X,7G10.3)
 72 FORMAT (1H0,* NEW SET, OLD SET HAD *I5* CHANGES*)
 73 FORMAT (1H1,*    MEMBER REFERENCE TABLE*//)
 74 FORMAT (1H0,5(5X,I4,* = *A5,F7.2))
 75 FORMAT (3I10)
 76 FORMAT (1H0,*     THE TOTAL WEIGHT OF THIS STRUCTURE IS*G15.6*POUND
```

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PROGRAM      DESIGN1                                        CDC 6400 FTN

          1S*)
       77 FORMAT (1H ,*   THIS IS A GRAVITY LOADING.*///)
       78 FORMAT (1H ,*    THIS IS A WIND LOADING.  REDUCTION = 3/4 ON ALL L
          1OADS.*//)
    10000 STOP
          END

SUBROUTINE   SETUM                                          CDC 6400 FTN

          SUBROUTINE SETUM (NEGPOS, J, M, NM, II)
          COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
          COMMON/ERT/RT(6,6), CT(208),ST(208),RTT(6,6)
          COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
          COMMON/LIMS/DL(208),SETSIZ(208),MOSIZE(208),INT, INTS
          COMMON/ECON/KK(16), KJ(16), NN(16), CWT(213)
          INTEGER DL, SETSIZ
          IF (SETSIZ(M).EQ.MSIZE(M).OR.MOSIZE(M).EQ.MSIZE(M)) GO TO 103
          IF (INT.LT.INTS+2) GO TO 100
          GO TO 104
      103 IF(NEGPOS.GT.0) GO TO 102
          IF (MOSIZE(M).LT.MSIZE(M)) MSIZE(M) = MOSIZE(M)
          SETSIZ(M) = MSIZE(M) + 213
          GO TO 100
      102 IF (MOSIZE(M).GT.MSIZE(M)) MSIZE(M) = MOSIZE(M)
          SETSIZ(M) = MSIZE(M) + 213
          GO TO 100
      104 MSIZE(M) = MAX0(J,MOSIZE(M))
          SETSIZ(M) = MSIZE(M) + 213
      100 RETURN
          END

SUBROUTINE   DEFLET                                         CDC 6400 FTN

          SUBROUTINE DEFLET
          COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
          COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
          COMMON/SIMEQ/S(312,30), AJ(312), DJ(312), AJO(312)
          COMMON/PV/M, MLOADS, AMX, PZ, MOM(9), Z(208),NINTEN
          COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
C   THIS SUBROUTINE USES THE NEWMARK METHOD TO COMPUTE DEFLECTIONS ALONG THE SPAN
          REAL MOM, IX
          I = 6*(M-1) + 3
          N = MSIZE(M)
          AJO(1) = -AMF(I)
          AJO(11) = AMF(I+3)
          DO 100 I=2,10
      100 AJO(I) = MOM(I-1)
C   COMPUTE ALPHA = M/EI
```

```
      DO 101 I=1,11
  101 AJO(I) = AJO(I)/(IX(N)*E)
      H = AL(M)/10.0
C  COMPUTE ALPHA BARS.----USE PARABOLIC FITS.
      AJ(1) = (H/24.0)*(7.0*AJO(1) + 6.0*AJO(2) - AJO(3))
      AJ(11) = (H/24.0)*(7.0*AJO(11) + 6.0*AJO(10) - AJO(9))
      DO 102 I=2, 10
  102 AJ(I) = (H/12.0)*(AJO(I-1) + 10.0*AJO(I) + AJO(I+1))
      I = JC(M)
      N = KC(M)
      IF (Z(I).GT.Z(N)) I = N
      I = I * 3
      AJO(1) = DJ(I) + AJ(1)
      DO 103 I=2, 10
  103 AJO(I) = (AJO(I-1) + AJ(I))
C  GET THE DELTA YS
      DO 104 I=1, 10
  104 AJO(I) = AJO(I)* H
      AJ(1) = 0
      DO 105 I=2, 11
  105 AJ(I) = AJ(I-1) + AJO(I-1)
      CORRECT = AJ(11)*(-1.0)
      DO 106 I=1, 11
      X = I
      X = X - 1.0
  106 AJ(I) = AJ(I) + (CORRECT*X/10.0)
C  FIND THE MAXIMUM DEFLECTION
      X = AJ(1)
      H = 1.0
      DO 107 I=2, 11
      IF (ABS(AJ(I)).LE.X) GO TO 107
      X = ABS(AJ(I))
      H = I
  107 CONTINUE
      H = ((H-1.0)/10.0)*AL(M)
      AJ(1) = AL(M)/X
      WRITE (6,50) X, H, AJ(1)
   50 FORMAT (1H0,14X,*BEAM DEFLECTION*/,15X,*THIS BEAM DEFLECTED*F7.4*
     1INCHES MAXIMUM AT*F6.1* INCHES FROM THE LEFT  HAND SIDE OF THE SPA
     2N.*/,15X*THE SPAN TO DEFLECTION RATIO WAS*F6.0)
      RETURN
      END
```

SUBROUTINE WTFEA   CDC 6400 FTN

```
      SUBROUTINE WTFEA (I,WT,A)
      COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
      COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
      COMMON/FRT/RT(6,6), CT(208),ST(208),RTT(6,6)
      WT = WT * 0.001 * A * 0.083333333
      RL = ABS(AL(I))
```

```
      CX = ABS(CT(I))
      CY = ABS(ST(I))
      FEA(1) = WT *RL*CY
      FEA(2) = WT*RL*CX*0.5
      FEA(3) = WT*RL**2*CX*0.0833333
      IF (IPIN(I).EQ.1) FEA(3) = 0.0
      FEA(4) = 0.0
      FEA(5) = FEA(2)
      FEA(6) = -FEA(3)
      WT = WT * 1000.0 * 12.0 * A
      RETURN
      END
```

SUBROUTINE SIZE   CDC 6400 FTN

```
      SUBROUTINE SIZE (M)
      COMMON/ECON/KK(16), KJ(16), NN(16), CWT(213)
      COMMON/LIMS/DL(208),SETSIZ(208),MOSIZE(208),INT, INTS
      COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
      COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
      COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
      INTEGER DL
      DEP = AL(M)/15.0
      IDEP = DEP
      IF (DL(M).LT.IDEP) IDEP = DL(M)
      DO 100 I=1, 16
      IF (KK(I).GE.IDEP) GO TO 101
  100 CONTINUE
  101 IF (IBC(M).NE.0) MSIZE(M) = KJ(I) + (NN(I)-KJ(I))*2/3
      IF (IBC(M).NE.1) MSIZE(M) = KJ(I)
      IF (MSIZE(M).LT.1.OR.MSIZE(M).GT.213) MSIZE(M) = 73
      RETURN
      END
```

SUBROUTINE FORMRT   CDC 6400 FTN

```
      SUBROUTINE FORMRT(M)
      COMMON/FRT/RT(6,6), CT(208),ST(208),RTT(6,6)
      REAL IX
      DO 108 I=1, 6
      DO 108 J=1, 6
  108 RT(I,J) = 0.0
      RT(1,1) = CT(M)
      RT(2,2) = CT(M)
      RT(4,4) = CT(M)
      RT(5,5) = CT(M)
      RT(1,2) =  ST(M)
      RT(5,4) = -ST(M)
      RT(2,1) = -ST(M)
      RT(4,5) =  ST(M)
      RT(3,3) = 1.0
      RT(6,6) = 1.0
      DO 109 K=1, 6
      DO 109 N=1, 6
      RTT(K,N) = RT(N,K)
  109 CONTINUE
      RETURN
      END
```

SUBROUTINE FORMSM CDC 6400 FTN

```
      SUBROUTINE FORMSM(M,N)
      COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
      REAL IX
      DO 110 I=1, 6
      DO 110 J=1, 6
      SM(I,J) = 0.0
  110 CONTINUE
      AEL = AREA(N)*E/AL(M)
      EIL = E*IX(N)/AL(M)
      IF (IPIN(M).EQ.1) EIL = 0.0001
      SM(4,1) = -AEL
      SM(1,1) = AEL
      SM(2,2) = 12. * EIL / AL(M)**2
      SM(3,2) = 6.0 * EIL / AL(M)
      SM(3,3) = 4.0 * EIL
      SM(4,4) = AEL
      SM(5,2) = -12.0 * EIL / AL(M)**2
      SM(5,3) = -6.0 * EIL / AL(M)
      SM(6,2) =  6.0 * EIL / AL(M)
      SM(6,3) = 2.0 * EIL
      SM(5,5) = 12.0 * EIL / AL(M)**2
      SM(6,5) = -6.0 * EIL / AL(M)
      SM(6,6) = 4.0 * EIL
C  SM IS SYMMETRICAL
      DO 111 I=1, 6
      DO 111 J=1, 6
  111 SM(I,J) = SM(J,I)
      RETURN
      END
```

SUBROUTINE MULT CDC 6400 FTN

```
      SUBROUTINE MULT (A,MA,NA,B,MB,NB,C)
      DIMENSION A(6,6), B(6,6), C(6,6)
      DO 201 I=1, MA
      DO 201 J=1, NB
      SUM = 0.0
      DO 30 L=1, NA
   30 SUM = SUM + A(I,L) * B(L,J)
  201 C(I,J) = SUM
      RETURN
      END
```

SUBROUTINE INDEX CDC 6400 FTN

```
      SUBROUTINE INDEX(M)
      COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
C  INDEX COMPUTES INDICIES FOR USE IN DO LOOPS USED IN STORING SMR IN S,
C  FEA IN AJ, AND IN EXTRACTING D(M) FROM DJ.
      JJ = JC(M)
      KK = KC(M)
      J3 = 3 * JJ
      K3 = 3 * KK
      IN(3) = J3
      IN(2) = J3 - 1
      IN(1) = J3 - 2
      IN(6) = K3
```

```
      IN(5) = K3 - 1
      IN(4) = K3 - 2
      RETURN
      END
```

SUBROUTINE BNDSOLV CDC 6400 FTN

```
      SUBROUTINE BNDSOLV  (JN,IBAND)
      COMMON/SIMEQ/S(312,30), AJ(312), DJ(312), AJO(312)
      NUMNP = 3*JN
      NPM1 = NUMNP - 1
      DO 450 I=1, NPM1
      D = 1.0/S(I,1)
      JEND = NUMNP - I + 1
      IF (JEND.GT.IBAND) JEND = IBAND
      DO 440 J=2, JEND
      NPN = I + J - 1
      FAC = S(I,J)*D
      M=0
      DO 430 K=J, JEND
      M = M + 1
      S(NPN,M) = S(NPN,M) - S(I,K)*FAC
  430 CONTINUE
      AJ(NPN) = AJ(NPN) - AJ(I)*FAC
      S(I,J) = FAC
  440 CONTINUE
      AJ(I) = AJ(I)*D
  450 CONTINUE
      AJ(NUMNP) = AJ(NUMNP)/S(NUMNP,1)
      DJ(NUMNP) = AJ(NUMNP)
      DO 470 I=1, NPM1
      NPN = NUMNP - I
      JEND = NUMNP - NPN + 1
      IF (JEND.GT.IBAND) JEND = IBAND
      RHS = AJ(NPN)
      DO 460 J=2, JEND
      M = NPN + J - 1
      RHS = RHS - S(NPN,J)*DJ(M)
  460 CONTINUE
      DJ(NPN) = RHS
  470 CONTINUE
      RETURN
      END
```

SUBROUTINE FBCB CDC 6400 FTN

```
      SUBROUTINE FBCB ( CB, FY, L, RT, DAF, FBTRI, FB, KFLAG )
      REAL L, LRT1, LRT2, LRT
    0 IF ( L .LE. 0.0 ) GO TO 14
    1 LRT1 = SQRT ( ( 510000.0 * CB )/FY )
    2 LRT2 = SQRT ( ( 102000.0 * CB )/FY )
    3 LRT = L/RT
    4 IF ( LRT .LT. LRT2 ) GO TO 14
    5 IF ( LRT .GT. LRT1 ) GO TO 8
    6 FB1 = ( 2.0/3.0 - ( FY * LRT**2 )/( 1530000.0 * CB ) ) * FY
    7 GO TO 9
    8 FB1 = ( 170000.0 * CB )/( LRT**2 )
    9 FBTEST = ( 12000.0 * CB )/( L * DAF )
   10 IF ( FB1 .GT. FBTEST ) FBTEST = FB1
   11 IF ( FBTRI .LT. FBTEST ) FBTEST = FBTRI
```

```
   12 FB = FBTEST
   13 GO TO 15
   14 FB = FBTRI
   15 KFLAG = 1
      RETURN
      END
```

SUBROUTINE FIXEND CDC 6400 FTN

```
      SUBROUTINE FIXEND (M,I)
      COMMON/LOADS/MEM(80), JCODE(80), RLOAD(80),  P(80),  Q(80)
      COMMON/FRT/RT(6,6), CT(208),ST(208),RTT(6,6)
      COMMON/MISC/MSIZE(208), FF(6), AMF(1248),AM1(6)
      COMMON/FSM/SM(6,6), AX(213), E, ALEN(208), IZ(213), IPIN(208)
      REAL IZ
C     THIS SUBROUTINE COMPUTES THE FIXED END ACTIONS FOR LATERALLY LOADED MEMB
C     RLENGTH IS THE LENGTH OF THE MEMBER
      RLENGTH=ALEN(M)
      R = 1.0 - P(I)
      T = Q(I)
      DO 9 K=1,6
    9 FF(K) = 0.0
      K = MSIZE(M)
      IF(JCODE(I).EQ.0) GO TO 10
      GO TO (11,12,13,14,15,16,17,100,100,100,100) JCODE(I)
  100 WRITE(6,101)
  101 FORMAT (1H1,*  YOU HAVE TROUBLE IN THE INPUT OF A MEMBER LOADING.
     1JCODE IS GREATER THAN THE MAXIMUM PERMISSIBLE VALUE.*)
      GO TO 50
C
C       CONCENTRATED LOAD
C
   11 FF(1)=0.0           $ FF(4)=0.
      FF(2)=-RLOAD(I)*(1.-3.*R**2+2.*R**3)
      FF(3)=-RLOAD(I)*R*RLENGTH*(1.-R)**2
      FF(5)=-RLOAD(I)*(R**2)*(3.-2.*R)
      FF(6)= RLOAD(I)*R**2*RLENGTH*(1.-R)
      GO TO 50
C
C       UNIFORMLY DISTRIBUTED LOAD
   12 A = 1.0
      FF(1) = 0.0  $  FF(4) = 0.0
  112 FF(2) = -RLOAD(I)*A*R*RLENGTH*(18.0-14.0*R-R*R+3.0*R**3)/12.0+FF(2
     1)
      FF(3) = -RLOAD(I)*A*RLENGTH**2*R**2*(6.0-8.0*R+3.0*R*R)/12.0 + FF(
     13)
      FF(5) = -RLOAD(I)*A*R*RLENGTH*(-6.0+14.0*R+R*R-3.0*R*R)/12. + FF(5
     1)
      FF(6) = RLOAD(I)*A*RLENGTH**2*R**3*(4.0-3.0*R)/12.0 + FF(6)
      IF (A.NE.1.0) GO TO 50
      A = -1.0
      R = T
      GO TO 112
C
C     CONCENTRATED MOMENT-- COUNTER CLOCKWISE POSITIVE
   13 FF(1)=0.0           $ FF(4)=0.
      FF(2)=6.*RLOAD(I)*R*(1.-R)/RLENGTH
      FF(3)=RLOAD(I)*(1.-4.*R+3.*R*R)*(-1.)
      FF(5)=-FF(2)
      FF(6)=-RLOAD(I)*R*(2.-3.*R)
      GO TO 50
```

```
C
C        JOINT DISPLACEMENT--COUNTER CLOCKWISE POSITIVE
C
   14 FF(1)=0.0       $ FF(4)=0.
      FF(2)=-12.*IZ(M)*RLOAD(I)/(RLENGTH**3)
      FF(3)=-6.*IZ(M)*RLOAD(I)/(RLENGTH**2)
      FF(5)=-FF(2)
      FF(6)= FF(3)
      GO TO 50
C
C         TEMPERATURE CHANGE FOR STEEL ONLY
C
   15 FF(1)=AX(K)*.0000065*RLOAD(I)
      FF(2)=0.   $ FF(3)=0. $ FF(5)=0. $ FF(6)=0.
      FF(4)=-FF(1)
      GO TO 50
C
C        JOINT DISPLACEMENT IN X DIRECTION
C
   16 FF(1)=-RLOAD(I)*AX(K)/(RLENGTH)
      FF(2)=0.   $ FF(3)=0. $ FF(5)=0. $ FF(6)=0.
      FF(4)=-FF(1)
      GO TO 50
C     UNIFORMLY VARYING LOAD-- LARGE PART AT NEAR END OF MEMBER
C     RLOAD = MAXIMUM MAGNITUDE
   17 FF(1)=0.0   $ FF(4)=0.
      FF(2)=-RLOAD(I)*R*RLENGTH*(10.-5.*R**2+2.*R**3)/20.
      FF(3)=-RLOAD(I)*(R**2)*(RLENGTH**2)*(10.-10.*R+3.*R**2)/60.
      FF(5)=-0.5*R*RLENGTH*RLOAD(I)+FF(2)
      FF(6)=-RLOAD(I)*R*R*R*(RLENGTH**2)*(5.-3.*R)/60.
      GO TO 50
   10 CONTINUE
   50 IF (IPIN(M).NE.1) GO TO 65
      FF(3) = 0.0    $    FF(6) = 0.0
   65 RETURN
      END
```

SUBROUTINE EFLEN   CDC 6400 FTN

```
      SUBROUTINE EFLEN (M)
      COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
      COMMON/EFL/Y(208),     EK, NM, SWAY, PIE
      COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
      COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
      REAL IX
      GA=0.0    $    GB=0.0
      COLA=0.0   $   COLB=0.0
      BEAMA=0.0   $   BEAMB = 0.0
      NA = JC(M)
      NB = KC(M)
C  BEGIN LOOP TO CALCULATE GA, GB VALUES
      DO 10 I=1, NM
      IF (IPIN(I).EQ.1) GO TO 10
      JTEST = JC(I)
      KTEST = KC(I)
      IF (JTEST.NE.NA.AND.KTEST.NE.NA) GO TO 20
C  MEMBER DOES FRAME INTO JOINT NA
      IA = MSIZE(I)
       EIL = IX(IA)/AL(I)
```

```
C  COLUMN OR BEAM
      IF (IBC(I) .EQ.0) GO TO 25
      COLA = COLA + EIL
      GO TO 20
   25 CALL GIRMOD (TEST,KTEST,JTEST,NA)
      BEAMA = BEAMA + EIL * TEST
   20 IF (JTEST.NE.NB.AND.KTEST.NE.NB) GO TO 10
      IA = MSIZE(I)
      EIL = IX(IA)/AL(I)
      IF (IBC(I).EQ.0) GO TO 35
      COLB = COLB +EIL
      GO TO 10
   35 CALL GIRMOD (TEST,KTEST,JTEST,NB)
      BEAMB = BEAMB + EIL * TEST
   10 CONTINUE
      IF (BEAMA.NE.0.0) GO TO 11
      NA = 3*(NA-1) + 1
      IF (FIXYZ(NA).NE.0) GA=30.0
      IF (GA.EQ.30.0.AND.FIXYZ(NA+1).NE.0) GO TO 12
      GO TO 13
   12 GA = 10.0
      IF (FIXYZ(NA+2).NE.0) GA = 1.0
   13 IF (GA.EQ.0.0) GA = 1000.0
      GO TO 14
   11 GA = COLA/BEAMA
   14 IF (BEAMB.EQ.0.0) BEAMB = .0001
      GB= COLB/BEAMB
      NA = JC(M)
      IF (Y(NA).GT.SWAY.OR.Y(NB).GT.SWAY) GO TO 60
C  THERE IS SWAY BRACING PRESENT
      TEST = 1.0
      EK = 0.51
   50 EIL = PIE/EK
      BEAMA = TEST
      TEST = (GA*GB*0.25)*EIL**2 +((GA+GB)*0.5)*(1.0 - PIE/(EK * TAN(EIL
     1))) + 2.0* TAN(EIL*0.5)/(EIL)
      IF (EK.LT.0.97) GO TO 53
      EK = 1.0
      GO TO 65
   53 IF ((TEST-1.0)/BEAMA) 65, 52, 52
   52 EK = EK+ 0.025
      GO TO 50
C  NO SWAY BRACING
   60 EK = 1.98
      EIL = PIE/EK
      TEST = (TAN(EIL)*(GA * GB * EIL**2 - 36.0))/(6.0*(GA+GB)*EIL)
      BEAMB = 1.0
      IF (TEST.GT.0.0) GO TO 62
      BEAMB = -1.15
      EK=2.025
   62 EIL = PIE/EK
      TEST = (TAN(EIL)*(GA * GB * EIL**2 - 36.0))/(6.0*(GA+GB)*EIL)
      IF (TEST.LT.1.03) GO TO 65
      EK = EK -0.025*BEAMB
      GO TO 62
   65 RETURN
      END
```

```
          SUBROUTINE PVM (WT,AMXA)
          COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
          COMMON/FRT/RT(6,6), CT(208),ST(208),RTT(6,6)
          COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
          COMMON/LOADS/MEM(80), JCODE(80), PLOAD(80), RN(80), RF(80)
          COMMON/PV/M, MLOADS, AMX, PZ, MOM(9), Z(208),NINTEN
          REAL MOM
          WT = WT*0.001*0.08333333
          RL = ABS(AL(M))
C   ADD IN END MOMENTS
C   SET UP AMF
          JJ = 6*(M-1) + 3
          J3 = JJ + 3
          HOLD = (AMF(JJ) + AMF(J3))/RL
          DO 100 I=1, 9
          X= I
          X = X * 0.1 * RL
      100 MOM(I) = HOLD*X - AMF(JJ)
C   ADD IN MOMENTS FROM WT
          DO 101 I=1,9
          X = I
          X = X * 0.1 * RL
      101 MOM(I) = MOM(I) +((WT*X*(RL-X))/2.0) * ABS(CT(M))
C     WORK ON MEMBER LOADS
          DO 102 IA=1, MLOADS
          IF (MEM(IA).NE.M) GO TO 102
          A = RN(IA)*RL
          B = RF(IA)*RL
          F = (-1.0)*PLOAD(IA)
          GO TO (10,20,30,102,102,102,40,102,102,102,102)  JCODE(IA)
C     POINT LOADS
       10 HOLD = F*A
          DO 103 I=1,9
          X=I
          X = X * 0.1 * RL
          IF (X.GT.A) GO TO 104
          MOM(I) = MOM(I) + (B*X*F)/RL
          GO TO 103
      104 MOM(I) = MOM(I) + (HOLD*(-1.0)*X)/RL + HOLD
      103 CONTINUE
          GO TO 102
C     UNIFORM LOADS
       20 HOLD = (RL-A-B)
          R1 = F*HOLD*(2.0*B+HOLD)/(2.0*RL)
          R2 = F*HOLD*(2.0*A+HOLD)/(2.0*RL)
          DO 105 I =1, 9
          X=I
          X = X * 0.1 * RL
          IF (X-A) 106,106,107
      106 MOM(I) = MOM(I) + R1*X
          GO TO 105
      107 IF (X-(RL  - B)) 108,109,109
      108 MOM(I) = MOM(I) + ((R1*X) - (F*(X-A)**2)/2.0)
          GO TO 105
      109 MOM(I) = MOM(I) + R2*(RL-X)
```

```
  105 CONTINUE
      GO TO 102
C     CONCENTRATED MOMENTS
   30 R1 = F/RL
      F =(-1.0) * F
      DO 110 I=1,9
      X=I
      X = X*0.1*RL
      IF (X-A) 111,111,112
  111 MOM(I) = MOM(I) + R1*X
      GO TO 110
  112 MOM(I) = MOM(I) + (R1*X+F)
  110 CONTINUE
      GO TO 102
C     UNIFORMLY VARING LOAD
   40 DO 113 I=1, 9
      X=I
      X = (1.0 - X*0.1)*RL
      HOLD = F*RL/2.0
  113 MOM(I) = MOM(I) + HOLD*X*(RL*RL - X*X)/(3.0*RL*RL)
  102 CONTINUE
      WT = WT*1000.0*12.0
      AMX = ABS(AMF(JJ))
      IF (ABS(AMF(J3)).GT.AMX) AMX = ABS(AMF(J3))
      IF (NINTEN.NE.1) GO TO 115
      IF (IPIN(M).EQ.1)GO TO 115
C     JUGGLE MOMENTS FOR 9/10 FACTOR
      HOLD = MOM(1)
      DO 116 I=2, 9
  116 IF (MOM(I).GT.HOLD) HOLD = MOM(I)
      IF (HOLD.LT.0.0) GO TO 115
      IF (HOLD.GT.AMX) GO TO 115
      X = (ABS(AMF(JJ)) + ABS(AMF(J3)))/2.0
      DO 117 I=1, 10
      HOLD1 = HOLD
      AMX1 = AMX
      F = I
      F = F * 0.01
      AMX1 = AMX1 * (1.0 - F)
      HOLD1 = HOLD1 + F*X
      IF (HOLD1.LT.AMX1) GO TO 117
      GO TO 118
  117 CONTINUE
  118 AMX = AMX1
      AMXA = HOLD1
      GO TO 200
  115 AMXA = ABS(HOLD)
  200 PZ = ABS(AMF(JJ-2))
      RETURN
      END
```

SUBROUTINE STORE CDC 6400 FTN

```
      SUBROUTINE STORE (M)
      COMMON/SIMEQ/S(312,30), AJ(312), DJ(312), AJO(312)
      COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
      COMMON/FRT/RT(6,6), CT(208),ST(208),RTT(6,6)
      COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
      CALL FORMRT(M)
      CALL MULT (RTT,6,6,FEA,6,1,AM1)
```

SUBROUTINE TABLEC CDC 6400 FTN

```
      SUBROUTINE TABLEC (THETA)
      COMMON/FSM/SM(6,6), AREA(213), E, AL(208),IX(213), IPIN(208)
      COMMON/LOADS/MEM(80), JCODE(80), RLOAD(80), RN(80), RF(80)
      COMMON/PV/M, MLOADS, AMX, PZ, MOM(9), X(208),NINTEN
      THETA = 0.0
      DO 100 I=1, MLOADS
      IF (MEM(I).NE.M) GO TO 100
      GO TO(102,104,102,100,100,100,104) JCODE(I)
  102 THETA = -0.6
      IF (IPIN(M).EQ.1) THETA = -0.2
      GO TO 100
  104 THETA = -0.4
      IF (IPIN(M).EQ.1) THETA = -0.0
      GO TO 106
  100 CONTINUE
  106 RETURN
      END
```

SUBROUTINE GIRMOD CDC 6400 FTN

```
      SUBROUTINE GIRMOD (TEST,KTEST,JTEST,NC)
C  THIS SUBROUTINE MODIFIES GA AND GB IF THE OPPOSITE END CONDITION OF
C  A GIRGER IS PINNED OR FIXED.
      COMMON/REFS/JC(208),KC(208),IN(6), IBC(208),FIXYZ(312)
      COMMON/EFL/Y(208),    EK, NM, SWAY, PIE
      TEST = 1.0
      MA = JTEST
      IF (MA.NE.NC) MA = KTEST
      MB = 3*(MA-1) + 1
      IF (Y(MA).GT.SWAY) GO TO 100
      IF (FIXYZ(MB).NE.0.0.OR.FIXYZ(MB+1).NE.0.0) TEST = 1.5
      IF (FIXYZ(MB+2).NE.0.0.AND.TEST.EQ.1.5) TEST = 2.0
      GO TO 1000
  100 IF (FIXYZ(MB).NE.0.0.OR.FIXYZ(MB+1).NE.0.0) TEST = 0.5
 1000 RETURN
      END
```

```
      C  STORE -AM1 IN AJ
            CALL INDEX(M)
            DO 106 IA=1, 6
            MA = IN(IA)
        106 AJO(MA) = AJO(MA) - AM1(IA)
      C  STORE MEMBER FEA IN AMF
            MA = 6 * (M-1)
            DO 105 IA=1, 6
            MA = MA + 1
        105 AMF(MA) = AMF(MA) + FEA(IA)
            RETURN
            END
```

SUBROUTINE C15141 CDC 6400 FTN

```
            SUBROUTINE C15141
            COMMON/MISC/MSIZE(208),FEA(6), AMF(1248),AM1(6)
            COMMON/PV/M, MLOADS, AMX, PZ, MOM(9), X(208),NINTEN
            COMMON/COMP/FA, FYIELD(208),CDEP(213), CTW(213), CBF(213), CDAF(21
           113), BRCEX(208),CTF(213)
      C  THIS SUBROUTINE CHECKS 1.5.1.4.1 OF THE CODE
            J = MSIZE(M)
            SFY = SQRT(FYIELD(M))
      C  CHECK 1.5.1.4.1D
            ADOT = 412.0 * (1.0-2.33*FA/FYIELD(M))/SFY
            IF (ADOT.LT.257.0/SFY) ADOT = 257.0/SFY
            DOT = CDEP(J)/CTW(J)
            IF (DOT.GT.ADOT) GO TO 100
      C  CHECK 1.5.1.4.1E
            ADOT = 76.0*(CBF(J)/SFY)
            DOT = 20000.0/(FYIELD(M)*CDAF(J))
            IF (DOT.GT.ADOT) ADOT = DOT
            IF (BRCEX(M).GT.ADOT) GO TO 100
      C  CHECK 1.5.1.4.1B.C
            NINTEN = 2
            DOT = 0.5*CBF(J)/CTF(J)
            ADOT = 52.2/SFY
            IF (BRCEX(M).EQ.0.0) ADOT = 190.0/SFY
            IF (DOT.GT.ADOT) GO TO 100
            NINTEN = 1
        100 RETURN
            END
```

APPENDIX C

MEMBER SIZES AND DEFLECTED SHAPES FOR SELECTED FRAMES

Appendix C

MEMBER SIZES AND DEFLECTED SHAPES FOR SELECTED FRAMES

This appendix shows the chosen member sizes and deflected shapes for several of the frames designed. Figs. C.1 to C.4 list the member sizes selected for the rigid frame, the X braced vertical truss, the K braced rigid frame and K braced rigid frame with an outrigger truss on the top floor. Since the frames are symmetrical, only the member sizes for one exterior bay and the interior bay are given.

Figure C.5 shows the deflected shapes for the K braced vertical truss and the rigid frame. The vertical truss shows its relative increase in stiffness at the lower levels as opposed to the rigid frames decrease in stiffness in the same area. The upswing of the rigid frame's deflection curve at the top illustrates the result of the smaller wind moment distortions when gravity loads rather than wind loads governed the member stress.

Figure C.6 shows the deflected shapes for the K braced rigid frame, the outrigger trussed rigid frame and the multistory X braced rigid frame. The gradual S shape of the K braced rigid frame respresents the combination of the two curves of Fig. B.5. Since the outrigger truss is simply the K braced rigid frame with the external bays of the top floor X braced, the stiffening effect of the external bracing is clearly seen. It tends to exaggerate the S shape by increasing the stiffness at the top of the frame. The stiffness of the multistory X braced frame is also shown.

Floor	Exterior		Interior		Bracing
	Column	Beam	Column	Beam	
20	W14 x 43	W24 x 61	W14 x 38	W24 x 61	
19	W14 x 43	W24 x 84	W14 x 48	W24 x 84	
18	W14 x 48	W24 x 100	W14 x 61	W24 x 100	
17	W14 x 61	W30 x 172	W14 x 74	W30 x 172	
16	W14 x 68	W30 x 210	W14 x 84	W30 x 210	
15	W14 x 78	W30 X 210	W14 x 103	W30 x 210	
14	W14 x 87	W30 x 210	W14 x 119	W30 x 210	
13	W14 x 103	W30 x 210	W14 x 136	W30 x 210	
12	W14 x 111	W30 x 210	W14 x 142	W30 x 210	
11	W14 x 119	W30 x 210	W14 x 167	W30 x 210	
10	W14 x 142	W30 x 210	W14 x 176	W30 x 210	
9	W14 x 150	W30 x 210	W12 x 202	W30 x 210	
8	W14 x 176	W30 x 210	W14 x 219	W30 x 210	
7	W14 x 211	W30 x 210	W14 x 246	W30 x 210	
6	W14 x 211	W30 x 210	W14 x 246	W30 x 210	
5	W14 x 219	W30 x 210	W14 x 264	W30 x 210	
4	W14 x 219	W30 x 210	W14 x 264	W30 x 210	
3	W14 x 237	W30 x 210	W14 x 287	W30 x 210	
2	W14 x 237	W30 x 210	W14 x 320	W30 x 210	
1	W14 x 455	W36 x 300	W14 x 500	W36 x 300	

Fig. C.1 Member sizes for the rigid frame.

Floor	Exterior Column	Exterior Beam	Interior Column	Interior Beam	Bracing
20	W14 x 22	W18 x 40	W14 x 68	W18 x 40	W14 x 43
19	W14 x 22	W18 x 40	W14 x 167	W18 x 40	W14 x 43
18	W14 x 30	W18 x 40	W14 x 202	W18 x 40	W14 x 43
17	W14 x 30	W18 x 40	W14 x 228	W18 x 40	W14 x 43
16	W14 x 30	W18 x 40	W14 x 264	W18 x 40	W14 x 43
15	W14 x 34	W18 x 40	W14 x 370	W18 x 40	W14 x 48
14	W14 x 38	W18 x 40	W14 x 370	W18 x 40	W14 x 53
13	W14 x 43	W18 x 40	W14 x 370	W18 x 40	W14 x 61
12	W14 x 48	W18 x 40	W14 x 455	W18 x 40	W14 x 61
11	W14 x 48	W18 x 40	W14 x 455	W18 x 40	W14 x 61
10	W14 x 53	W18 x 40	W14 x 455	W18 x 40	W14 x 61
9	W14 x 53	W18 x 40	W14 x 455	W18 x 40	W14 x 61
8	W14 x 61	W18 x 40	W14 x 455	W18 x 40	W14 x 61
7	W14 x 61	W18 x 40	W14 x 500	W18 x 40	W14 x 61
6	W14 x 68	W18 x 40	W14 x 500	W18 x 40	W14 x 61
5	W14 x 68	W18 x 40	W14 x 500	W18 x 40	W14 x 68
4	W14 x 68	W18 x 40	W14 x 500	W18 x 40	W14 x 68
3	W14 x 74	W18 x 40	W14 x 500	W18 x 40	W14 x 68
2	W14 x 78	W18 x 40	W14 x 550	W18 x 40	W14 x 74
1	W14 x 87	W18 x 40	W14 x 550	W18 x 40	W14 x 78

Fig. C.2 Member sizes for the X braced vertical truss.

Floor	Exterior		Interior		Bracing
	Column	Beam	Column	Beam	
20	W14 x 38	W21 x 44	W14 x 38	W18 x 35	W14 x 22
19	W14 x 38	W21 x 49	W14 x 53	W18 x 35	W14 x 34
18	W14 x 43	W21 x 49	W14 x 87	W18 x 35	W14 x 34
17	W14 x 43	W21 x 49	W14 x 87	W21 x 44	W14 x 34
16	W14 x 48	W21 x 49	W14 x 95	W21 x 44	W14 x 34
15	W14 x 53	W21 x 49	W14 x 111	W21 x 44	W14 x 34
14	W14 x 61	W24 x 55	W14 x 111	W21 x 49	W14 x 34
13	W14 x 68	W24 x 55	W14 x 111	W21 x 49	W14 x 34
12	W14 x 68	W24 x 55	W14 x 127	W21 x 49	W14 x 48
11	W14 x 68	W24 x 55	W14 x 142	W21 x 49	W14 x 48
10	W14 x 74	W24 x 55	W14 x 150	W21 x 49	W14 x 48
9	W14 x 74	W24 x 55	W14 x 158	W21 x 49	W14 x 48
8	W14 x 78	W24 x 55	W14 x 176	W21 x 49	W14 x 48
7	W14 x 84	W24 x 55	W14 x 193	W21 x 49	W14 x 48
6	W14 x 84	W24 x 55	W14 x 219	W21 x 49	W14 x 48
5	W14 x 84	W24 x 55	W14 x 219	W21 x 49	W14 x 48
4	W14 x 87	W24 x 55	W14 x 237	W21 x 49	W14 x 48
3	W14 x 87	W24 x 55	W14 x 246	W21 x 49	W14 x 48
2	W14 x 95	W24 x 55	W14 x 246	W21 x 49	W14 x 53
1	W14 x 95	W24 x 55	W14 x 264	W21 x 49	W14 x 61

Fig. C.3 Member sizes for the K braced rigid frame.

Floor	Exterior		Interior		Bracing
	Column	Beam	Column	Beam	
20	W14 x 30	W14 x 34	W14 x 43	W14 x 30	W14 x 43E W14 x 34I
19	W14 x 38	W18 x 40	W14 x 53	W12 x 19	W14 x 22
18	W14 x 38	W21 x 44	W14 x 78	W12 x 16.5	W14 x 30
17	W14 x 43	W21 x 44	W14 x 78	W12 x 19	W14 x 30
16	W14 x 43	W21 x 55	W14 x 78	W12 x 19	W14 x 30
15	W14 x 53	W21 x 55	W14 x 84	W12 x 19	W14 x 30
14	W14 x 61	W21 x 55	W14 x 84	W12 x 19	W14 x 30
13	W14 x 61	W21 x 55	W14 x 87	W12 x 19	W14 x 30
12	W14 x 68	W21 x 55	W14 x 95	W12 x 22	W14 x 34
11	W14 x 68	W21 x 55	W14 x 95	W12 x 22	W14 x 34
10	W14 x 74	W21 x 55	W14 x 111	W14 x 26	W14 x 38
9	W14 x 74	W24 x 55	W14 x 127	W14 x 30	W14 x 43
8	W14 x 78	W24 x 55	W14 x 142	W14 x 30	W14 x 43
7	W14 x 78	W24 x 61	W14 x 158	W14 x 30	W14 x 43
6	W14 x 84	W24 x 61	W14 x 176	W14 x 30	W1r x 43
5	W14 x 84	W24 x 61	W14 x 193	W14 x 34	W14 x 43
4	W14 x 87	W24 x 61	W14 x 211	W14 x 34	W14 x 48
3	W14 x 95	W24 x 61	W14 x 237	W18 x 35	W14 x 48
2	W14 x 95	W24 x 68	W14 x 287	W16 x 40	W14 x 48
1	W14 x 95	W24 x 68	W14 x 320	W16 x 40	W14 x 61

Fig. C.4 Member sizes for the outrigger truss (20th floor).

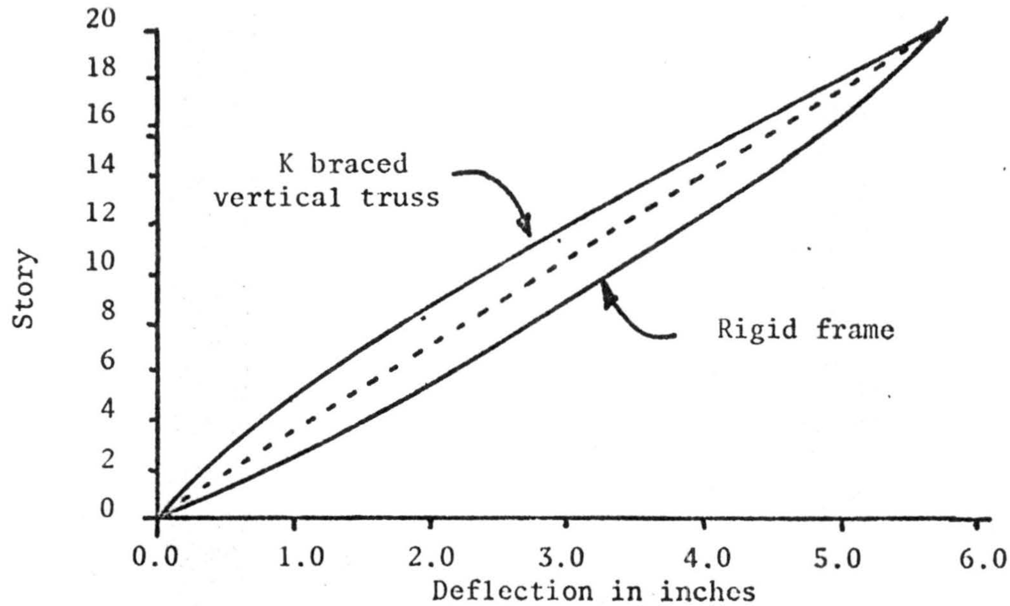

Fig. C.5 Deflected shapes of various frames, 3/4 (W+G) loading.

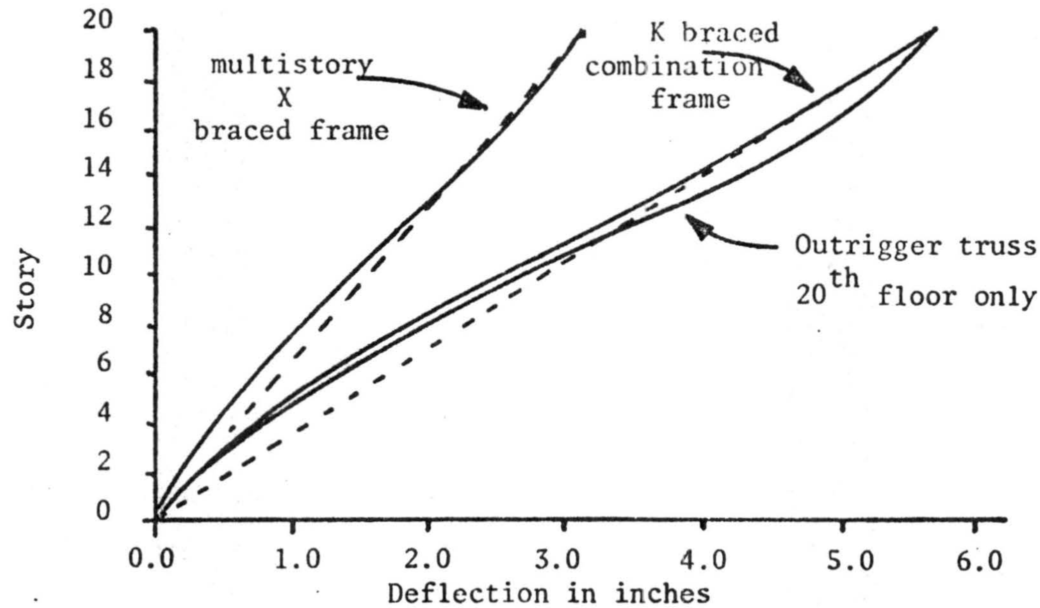

Fig. C.6 Deflected shapes of various frames, 3/4 (W+G) loading.